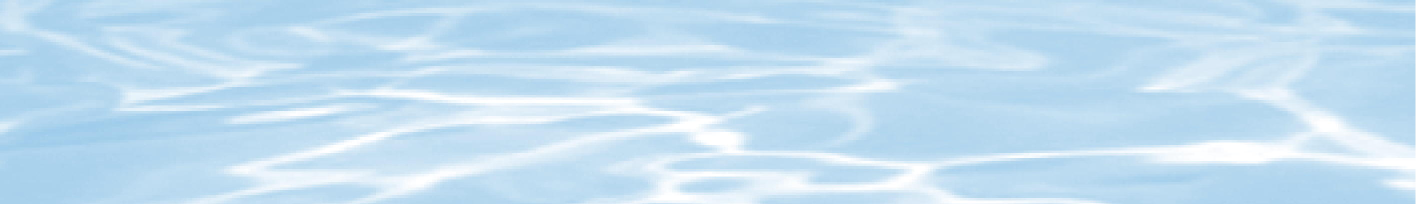
Awwa Research Foundation
Advancing the Science of Water®

Water Treatment Residuals Engineering

Subject Area:
Environmental Leadership



Water Treatment Residuals Engineering

Awwa
Research
Foundation

About the Awwa Research Foundation

The Awwa Research Foundation (AwwaRF) is a member-supported, international, nonprofit organization that sponsors research to enable water utilities, public health agencies, and other professionals to provide safe and affordable drinking water to consumers.

The Foundation's mission is to advance the science of water to improve the quality of life. To achieve this mission, the Foundation sponsors studies on all aspects of drinking water, including supply and resources, treatment, monitoring and analysis, distribution, management, and health effects. Funding for research is provided primarily by subscription payments from approximately 1,000 utilities, consulting firms, and manufacturers in North America and abroad. Additional funding comes from collaborative partnerships with other national and international organizations, allowing for resources to be leveraged, expertise to be shared, and broad-based knowledge to be developed and disseminated. Government funding serves as a third source of research dollars.

From its headquarters in Denver, Colorado, the Foundation's staff directs and supports the efforts of more than 800 volunteers who serve on the board of trustees and various committees. These volunteers represent many facets of the water industry, and contribute their expertise to select and monitor research studies that benefit the entire drinking water community.

The results of research are disseminated through a number of channels, including reports, the Web site, conferences, and periodicals.

For subscribers, the Foundation serves as a cooperative program in which water suppliers unite to pool their resources. By applying Foundation research findings, these water suppliers can save substantial costs and stay on the leading edge of drinking water science and technology. Since its inception, AwwaRF has supplied the water community with more than $300 million in applied research.

More information about the Foundation and how to become a subscriber is available on the Web at **www.awwarf.org**.

Water Treatment Residuals Engineering

Prepared by:
David A. Cornwell
Environmental Engineering & Technology, Inc.
712 Gum Rock Court
Newport News, VA 23606

Sponsored by:
Awwa Research Foundation
6666 West Quincy Avenue, Denver, CO 80235-3098

Published by:

Awwa Research Foundation

American Water Works Association

DISCLAIMER

This study was funded by the Awwa Research Foundation (AwwaRF). AwwaRF assumes no responsibility for the content of the research study reported in this publication or for the opinions or statements of fact expressed in the report. The mention of trade names for commercial products does not represent or imply the approval or endorsement of AwwaRF. This report is presented solely for informational purposes.

Printed in the U.S.A.

Printed on recycled paper

CONTENTS

TABLES

FIGURES

FOREWORD

The Awwa Research Foundation (AwwaRF) is a nonprofit corporation that is dedicated to the implementation of a research effort to help utilities respond to regulatory requirements and traditional high-priority concerns of the industry. The research agenda is developed through a process of consultation with subscribers and drinking water professionals. Under the umbrella of a Strategic Research Plan, the Research Advisory Council prioritizes the suggested projects based upon current and future needs, applicability, and past work; the recommendations are forwarded to the Board of Trustees for final selection. The foundation also sponsors research projects through an unsolicited proposal process; the Collaborative Research, Research Applications, and Tailored Collaboration programs; and various joint research efforts with organizations such as the U.S. Environmental Protection Agency, the U.S. Bureau of Reclamation, and the Association of California Water Agencies.

This publication is a result of one of these sponsored studies, and it is hoped that its findings will be applied in communities throughout the world. The following report serves not only as a means of communicating the results of the water industry's centralized research program but also as a tool to enlist the further support of the nonmember utilities and individuals.

Projects are managed closely from their inception to the final report by the foundation's staff and large cadre of volunteers who willingly contribute their time and expertise. The foundation serves a planning and management function and awards contracts to other institutions such as water utilities, universities, and engineering firms. The funding for this research effort comes primarily from the Subscription Program, through which water utilities subscribe to the research program and make an annual payment proportionate to the volume of water they deliver and consultants and manufacturers subscribe based on their annual billings. The program offers a cost-effective and fair method for funding research in the public interest.

A broad spectrum of water supply issues is addressed by the foundation's research agenda: resources, treatment and operations, distribution and storage, water quality and analysis, toxicology, economics, and management. The ultimate purpose of the coordinated effort is to assist water suppliers to provide the highest possible quality of water economically and reliably. The true benefits are realized when the results are implemented at the utility level. The foundation's trustees are pleased to offer this publication as a contribution toward that end.

Walter J. Bishop
Chair, Board of Trustees
Awwa Research Foundation

Robert C. Renner, P.E.
Executive Director
Awwa Research Foundation

ACKNOWLEDGMENTS

The author of this report is indebted to Nancy McTigue for the many hours spent in technical review and editing of the book, and also to Kimberly Bonilla for the work involved in processing the manuscript text and the many figures.

This text is a compilation of the work completed on several Awwa Research Foundation projects. It brings together the significant contributions of all those researchers so that the profession could have a central source on residuals management. I have tried to properly acknowledge all the researchers whose work was included.

Several utilities also assisted in providing needed information, particularly the Philadelphia Water Department, Philadelphia, Pa.; the Cleveland Division of Water, Cleveland, Ohio; the City of Newport News, Va.; Dayton Water Department, Dayton, Ohio; and Utilities Department of Lake County, Painesville, Ohio.

EXECUTIVE SUMMARY

Water treatment processes are utilized to remove contaminants from water or to alter the contaminant properties, generally through oxidation, in order to produce a potable water. All water treatment processes that remove contaminants produce a waste by-product. That by-product may be liquid, solid, a mixture of the two, or a gaseous vapor. These water treatment plant wastes are referred to as residuals. The names of individual waste streams are generally a function of how the residual is produced. There are five general types of water treatment processes that produce residuals. The first is produced at those plants that coagulate and oxidize a surface water to remove particles (both organic and inorganic) and dissolved contaminants such as color, organic carbon, iron, manganese, and occasionally trace metals. These coagulation plants produce two major residuals, sedimentation (or clarifier) sludge and spent filter backwash water (SFBW).

The second type of residual is produced at treatment plants that practice softening for the removal of calcium and magnesium. These plants may also remove trace metals, radioactivity, and particles if a surface water is being treated. These plants also produce a clarifier sludge and SFBW.

Ion exchange processes, as their name implies, are used to remove cation or anion contaminants such as calcium and magnesium, arsenic, nitrate, and barium. These processes produce a brine residual as well as spent rinse water. There are also adsorption processes that produce similar types of residuals. Some adsorption processes use “throw-away media” such that the residual produced is the spent adsorption material. Adsorption media may also need to be backwashed so that a spent backwash water is produced.

The fourth general category of treatment is when membranes are used to remove particulates or dissolved solids. In this case a concentrate is produced that consists of concentrated levels of the raw water contaminants rejected by the membrane as well as any additives that may have been used prior to membrane treatment.

Finally, gaseous residuals are produced by air stripping processes that release vapor to the atmosphere. These releases are primarily volatile organic compounds and radon.

This book is a revised and updated version of the original *Handbook of Water Treatment Plant Waste Management* published in 1987 by the Awwa Research Foundation (AwwaRF).

In rewriting this text, many of the AwwaRF projects that have been compiled on residuals over the last 15 years were included in an attempt at providing a single reference of information on residuals and providing the appropriate information sources for the readers to find additional detail.

CHAPTER 1
WATER TREATMENT PLANT RESIDUALS

Water treatment processes are utilized to remove contaminants from water or to alter the contaminant properties, generally through oxidation, in order to produce a potable water. All water treatment processes that remove contaminants produce a waste by-product. That by-product may be liquid, solid, a mixture of the two, or a gaseous vapor. These water treatment plant (WTP) wastes are referred to as residuals. The names of individual waste streams are generally a function of how the residual is produced. There are five general types of water treatment processes that produce residuals. The first type of residual is produced at those plants that coagulate and oxidize a surface water to remove particles (both organic and inorganic) and dissolved contaminants such as color, organic carbon, iron, manganese, and occasionally trace metals. These coagulation plants produce two major residuals, sedimentation (or clarifier) sludge and spent filter backwash water (SFBW).

The second type of residual is produced at treatment plants that practice softening for the removal of calcium and magnesium. These plants may also remove trace metals, radioactivity, and particles if a surface water is being treated. These plants also produce a clarifier sludge and SFBW.

Ion exchange processes, as the name implies, are used to remove cation or anion contaminants such as calcium and magnesium, arsenic, nitrate, and barium. These processes produce a brine residual as well as spent rinse water. There are also adsorption processes that produce similar types of residuals. Some adsorption processes use “throw-away media” such that the residual produced is the spent adsorption material. Adsorption media may also need to be backwashed so that a spent backwash water is produced.

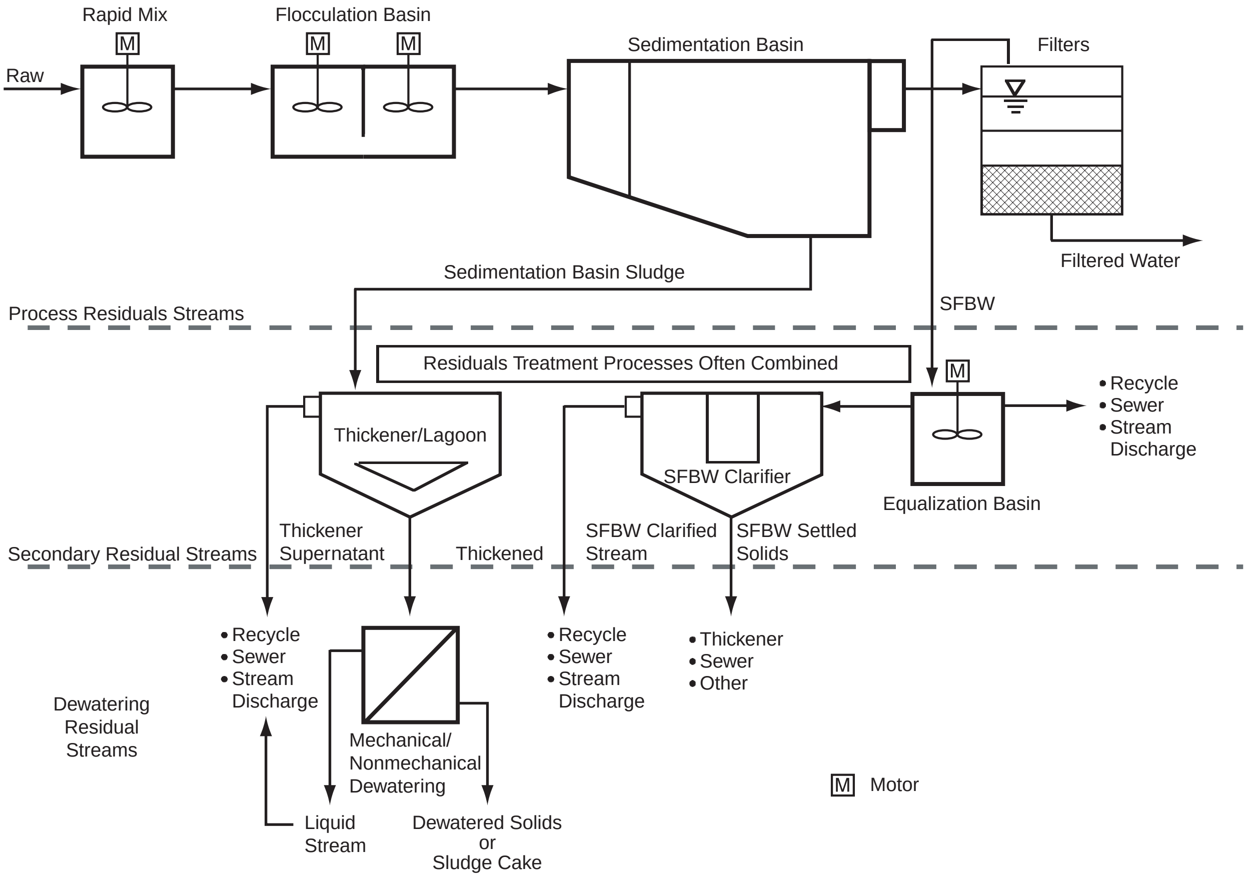
The fourth general category of treatment is when membranes are used to remove particulates or dissolved solids. In this case, a concentrate is produced that consists of concentrated levels of the raw water contaminants that the membrane has rejected as well as any additives that may have been used prior to membrane treatment.

Finally, gaseous residuals are produced by air stripping processes that release vapor to the atmosphere. These releases are primarily volatile organic compounds (VOCs) and radon.

COAGULATION WASTE STREAMS

Coagulation of surface waters is by far the most commonly used water supply treatment technology. These waste streams make up the majority of the water plant wastes produced by the water industry. They are also some of the more difficult wastes to treat. Figure 1.1 shows a schematic of a conventional coagulation treatment process showing the typical waste products. Some water plants have a presedimentation step. This is generally used only when the raw water source is high in settleable solids. Often no chemical is added prior to presedimentation, although an oxidant or small amount of polymer or coagulant may be added.

The coagulation process itself generates most of the waste solids. Generally a metal salt (aluminum or iron) is added as the primary coagulant. In addition to the coagulant, other solids producing chemicals such as powdered activated carbon (PAC), polymer, clay, lime, or activated silica may be used. These added chemicals will all produce waste solids. They are usually removed, along with the solids in the raw water, in a sedimentation tank or clarifier. These residuals are



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 1.1 Major residual streams commonly generated by conventional WTPs

referred to as sedimentation sludge or clarifier sludge. They are more specifically referred to by the type of coagulant used. For example, alum sludge is residuals produced from the use of an aluminum-based coagulant and iron sludge is residuals produced by the use of an iron-based coagulant. When dissolved air flotation (DAF) is used as the clarification step, the residuals are referred to as float. In areas with very good raw water quality, the clarification step is occasionally omitted and the solids are removed by filtration only. This process, commonly known as direct filtration, is usually used for water with low turbidity and requiring low levels of coagulant.

The residuals can be further treated (addressed in subsequent chapters), resulting in additional residuals streams. Much of the terminology is somewhat obvious. A thickener-treating clarifier sludge or clarifier sludge plus SFBW produces thickened sludge (which could be thickened alum or iron sludge) and thickener supernatant. A thickener that only treats SFBW will produce SFBW settled solids. A dewatering device will produce a sludge cake (also called dewatered solids) as well as a liquid stream. The liquid stream is referred to by the type of dewatering used such as filtrate, decant, centrate, and pressate.

The quantity of solids produced depends on the raw water quality and chemical addition. Chapter 2 includes a discussion of how to determine the amount of solids produced. The volume of sedimentation basin sludge produced depends on both the characteristics of the solids and the mechanism by which solids are removed from the basin. Many basins, particularly older ones, do not have mechanical means for solids removal and must be manually cleaned. In these basins the solids are stored for extended periods of time and are allowed to accumulate to a predetermined level. Periodically the basin is drained and is often washed out with a fire hose. Obviously, for these basins, the cleaning frequency is a function of the volume of sludge produced and the available storage volume in the basins. Manual cleaning results in batch production of sludge and makes subsequent sludge handling more difficult. In most situations it is desirable to retrofit the basins with continuous sludge removal equipment, which may be difficult to accomplish because of basin configurations. However, producing a fairly continuous and consistent flow of sludge to the sludge treatment process is often a critical factor in successful dewatering. Appropriate sludge removal in combination with flow equalization must be well planned.

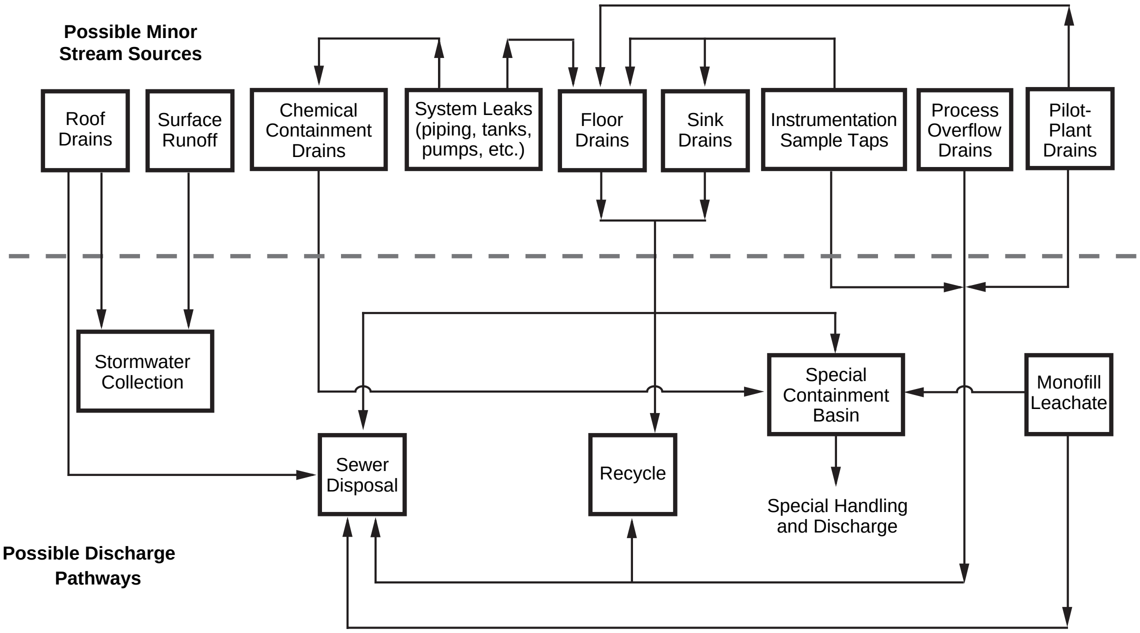
The second major residual produced is from the batch process of backwashing the filters, SFBW. Note that the residuals are not called backwash water; backwash water is used to clean the filters. The solids collected on the filters are those remaining after sedimentation, are caused by the addition of a filter aid, or are formed by oxidation of perhaps iron or manganese. In a direct filtration process, these are the only solids produced. The volume of SFBW produced is a function of the amount of water used for backwashing. This waste stream is produced at very high flow rates for short periods of time.

Another waste product that is occasionally produced in a coagulation-based plant is spent granular activated carbon (GAC). GAC is sometimes used in the filters or postfiltration. When its use is for taste and odor removal, the carbon is disposed of after its capacity is exhausted. When its use is for continuous low-level organics removal, the carbon is then usually regenerated onsite or through a vendor.

There can also be a wide variety of what are referred to as minor waste streams that are produced at WTPs and need to be addressed (Figures 1.2 and 1.3). Minor waste streams generated at WTPs include analysis waters, sample line and sink drainage, leakage from pipes and pump seals, pump packing water, floor and lab drains, roof drains, GAC transport water, leakage from chemical tanks, and process overflows. Analysis waters include spent water from online analyses conducted on water from various points in the treatment process. Sample lines from various points in the treatment process are typically routed to laboratory sinks, along with discarded analysis waters. Water normally flows continuously through sample lines at a relatively slow rate, approximately 0.5 gpm (0.03 L/sec), into a lab sink.

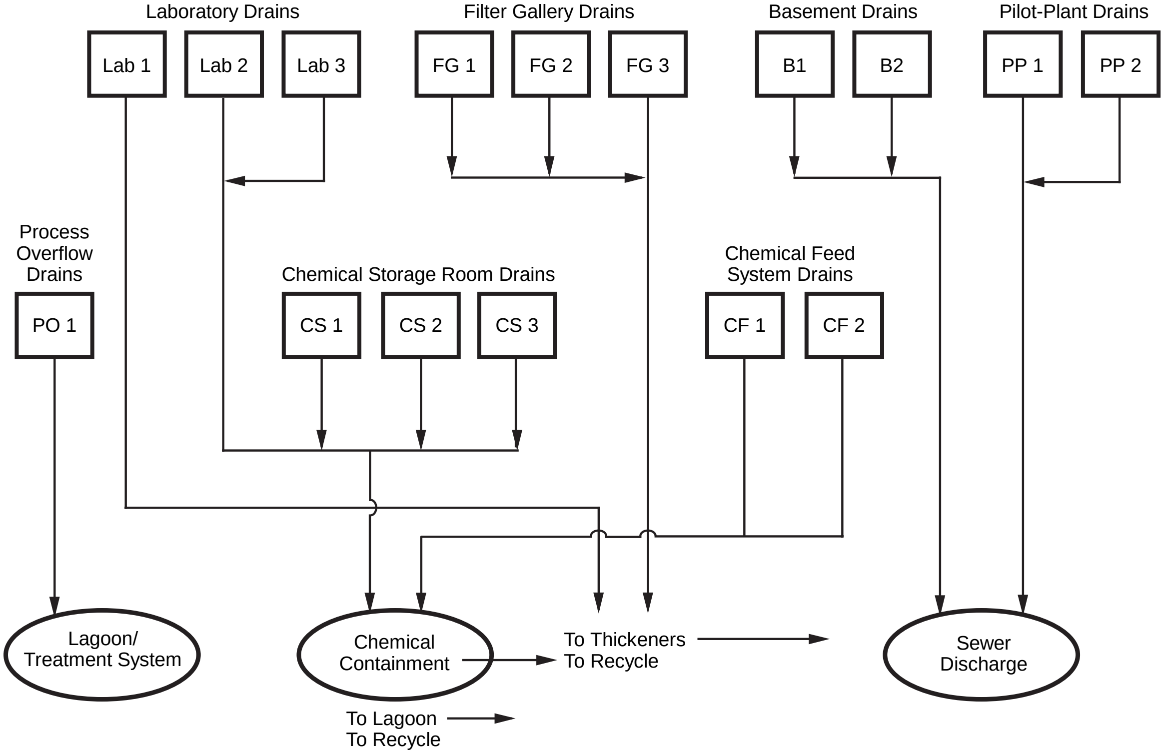
Another category of minor waste streams is process overflows. Unit process tanks (rapid mix, flocculation, sedimentation, etc.) are often designed with an overflow located above the normal operating water level. Thus, process overflow streams are generated only when water levels rise above design operating levels to the overflow elevation in a tank.

Analysis water quantities generated continuously could be on the order of several hundred gallons per day or more depending on the size of the WTP and its support facilities. For example, an online turbidimeter may flow at 500 mL/min, which generates about 190 gpd (720 L/d). Sample lines may produce a few thousand gallons per day, assuming four or five sample lines run continuously into a lab sink at a rate of 0.5 gpm (0.03 L/sec). Leakage amounts from pipes, etc., in a plant are related to the condition of equipment but would be minimal under ideal conditions. Volumes of water collected in roof drains would depend on rainfall quantities, and process overflows from water treatment unit processes would be zero under normal operating conditions.



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 1.2 Example of minor residual stream sources and discharge pathways



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 1.3 Example floor drain floor schematics

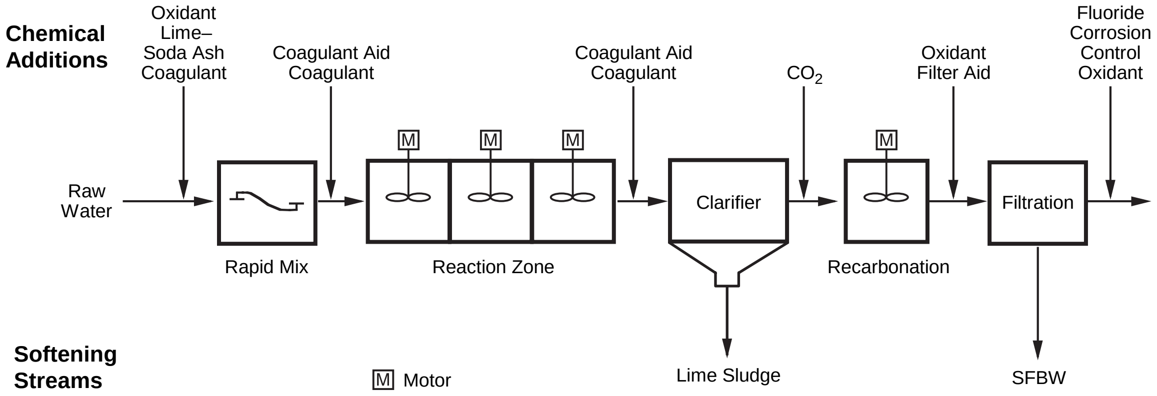


Figure 1.4 Residual-producing processes in softening plants

SOFTENING WASTE STREAMS

Wastes produced from softening plants represent the second major waste product produced by the water industry. Fortunately, they are generally more easily dewatered than are coagulant wastes, although the presence of some trace inorganics can make their proper disposal difficult. There are many variations of the softening process. Chemical addition, flow processes, and the subsequent waste quantities and characteristics are all dependent on the raw water hardness and alkalinity, and the desired finished water quality. Since softening is generally a process used to improve the chemical characteristics and aesthetics of the finished water rather than its potability, subjective decisions can be made as to the final desired quality. One of the factors that should enter into the decision process is the impact on sludge handling and costs.

Softening is accomplished either by chemical precipitation of the calcium and magnesium or by the use of ion exchange (IX) resins. The former, traditionally called lime–soda ash softening, is by far the most widely used softening process. In this method, lime is added for the removal of carbonate hardness supplemented with the use of soda ash for noncarbonate hardness removal if required. From the standpoint of sludge economics, it is desirable to leave as much magnesium hardness in the water as considered acceptable. Often the magnesium hardness can be allowed to remain around 40 mg/L as calcium carbonate ($CaCO_3$), or slightly higher, and not have an adverse effect on home water heaters. The less magnesium in the sludge, the easier it is to dewater.

Figure 1.4 is a rather simplified softening plant schematic. Several variations and configurations of Figure 1.4 are used to obtain the desired water quality and minimize costs. In softening plants there are usually two waste streams produced: the lime sludge from the clarifier and the backwash residual. Some plants will add a polymer or metal salt to aid in the removal of fine precipitates, color, or turbidity present in the original water. Again, from a sludge viewpoint, the addition of metal salts should be held to a minimum as the presence of metal hydroxides could significantly increase sludge treatment costs. The use of polymers and slurry recirculation can help minimize the use of these coagulants.

In many plants, the reaction zone and clarifier are combined into a single solids contact unit. In these plants, sludge can be fairly uniformly withdrawn from the sludge blanket, and a consistent suspended solids concentration and flow rate can be maintained. Plants that have separate clarifiers are often equipped with scrapers for sludge removal. Although not quite as easy to control as the sludge blanket units, the separate clarifiers can produce a fairly consistent sludge. As with coagulation plants, SFBW is produced at high flow rates for short periods of time.

ION EXCHANGE AND ADSORPTION

In water softening by IX, the water containing the hardness is passed through a column containing the IX material. The hardness in the water exchanges with an ion from the IX resin. Generally, the ion exchanged with the hardness is sodium:

$$Ca\,(HCO_3)_2 + 2NaR = CaR_2 + 2NaHCO_3 \tag{1.1}$$

where R represents the solid IX material. By the above reaction, calcium (or magnesium) has been 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 essentially 100 percent removal of the hardness from the water until the exchange capacity of the IX material is reached. When the IX resin becomes saturated, “breakthrough” is said to have occurred because the hardness is no longer removed. At this point, the IX material is regenerated. During regeneration, the hardness is removed from the material by passing water containing a large amount of Na^+ through the column. The mass action of having so much Na^+ in the water will cause the hardness of the IX material to enter the water and exchange with the sodium:

$$CaR_2 + 2NaCl = 2NaR + CaCl_2 \tag{1.2}$$

The IX material can now be used to remove more hardness.

This spent regenerant or brine is the residual stream that requires disposal. It contains the excess or leftover NaCl, and the ions removed—calcium chloride ($CaCl_2$) and magnesium chloride ($MgCl_2$). Economics dictate that a readily available location be used for disposal of this brine. Therefore, most large plants that utilize IX softening are located in coastal communities so that ocean brine disposal is practiced. Ion exchange has been used in small water supply systems in other parts of the country, and wastes have most often been discharged to municipal wastewater systems, or to receiving streams. Two additional waste streams are also produced in conjunction with IX. Prior to the use of the regenerant, the column is usually backwashed in an upflow mode to remove any suspended material. After regeneration, the column is rinsed, which will produce a waste stream also high in dissolved solids.

Removal of trace inorganic substances by IX or adsorption, such as arsenic, barium, cadmium, chromium, fluoride, lead, nitrate, mercury, selenium, silver, radium, and uranium, is becoming more of a concern in the drinking water industry. Often hardest hit with removal of these constituents is the very small utility, with limited capital but with no other water source. The proper handling and disposal of resulting wastes can be critical to overall economic success.

MEMBRANE RESIDUALS

Membranes can be used to remove a variety of contaminants. The size of contaminant removed depends upon the type of membrane selected and its associated pore size.

As membrane systems are increasingly used for water utility applications, the management of their residuals has become a growing challenge. Membrane technology uses a driving force (e.g., electrical, pressure/vacuum, etc.) to separate contaminants from the water. Pressure-driven membranes include microfiltration (MF), ultrafiltration (UF), nanofiltration (NF), and reverse osmosis (RO). Electrically-driven membranes include electrodialysis (ED) and its variant, electrodialysis reversal (EDR). Whereas MF and UF membranes are designed for particle removal and use low pressure, NF, RO, and ED/EDR are designed for desalination (and softening).

MF and UF processes are low-pressure membranes that primarily remove particles. The particles build up on the membranes and are backwashed off to clean the membrane. This membrane backwash residual contains the particles that were removed from the source water. Occasionally a coagulant or PAC is added to the raw water prior to the membrane and that substance will also be in the membrane backwash. These residuals can be referred to as membrane backwash but are also simply referred to as concentrates.

High-pressure membranes such as RO and NF primarily remove dissolved ions. These membranes produce a fairly continuous residual that contains the concentrated ions that substance the membrane rejects. The waste streams are referred to as concentrate and occasionally brine.

ADDITIONAL RESIDUALS

There are several additional residuals that are produced because of specific contaminant removal. Many of these are addressed in this book and include specific contaminants such as arsenic removal and radionuclide removal, which have significant regulatory issues associated with their proper management.

REFERENCE

Cornwell, D.A., M.J. MacPhee, and R.N. Mutter. 2003. *Self Assessment of Recycle Practices*. Washington, D.C.: AWWA.

CHAPTER 2
COAGULANT AND LIME SOFTENING RESIDUALS

OVERVIEW

It has been said that there is no such thing as a typical WTP. There is likewise no such thing as a typical water plant residual. A detailed analysis of residual characteristics should be conducted for each site. However, there are common characteristics among particular types of water plant residuals, and there are certainly common test methods that can be used to evaluate residuals.

In beginning to characterize water plant wastes, there are four important areas that need to be addressed:

- Type of residual generated
- Quantity of residual generated
- Classification by physical properties and dewatering characteristics
- Specific constituents in the residual streams, particularly as they may relate to proper disposal or beneficial use

In this chapter, sufficient guidance has been provided to enable the utility or design engineer to characterize their specific residuals. Methods are presented to estimate residual quantities (for coagulant and softening plant residuals) and suspended or dissolved components as may be appropriate. Typical values obtained from a cross section of plants as were available in the literature or produced from direct contact have been included. The Residual Physical Characteristics section is primarily directed at test methods to evaluate conditioning and dewaterability of solid–liquid phase residual streams. The Chemical Characteristics section presents example chemical analyses of residuals resulting from these processes. As appropriate, methods to estimate the chemical composition are presented.

SOLID–LIQUID RESIDUALS

Hydrolyzing metal salts or synthetic organic polymers are added in the water treatment process to coagulate suspended and dissolved contaminants and yield relatively clean water that is suitable for filtration. Most of these coagulants and the impurities they remove settle to the bottom of the settling basin where they become part of the sludge. These sludges are referred to as alum, iron, or polymeric sludge according to which primary coagulant is used. The sludges produced in treatment plants where water softening is practiced using lime or lime and soda ash account for the additional major source of sludge. These residuals are referred to as lime sludge. It is therefore apparent that most of the waste generation involves WTPs using coagulation or softening processes. The above residuals are referred to as solid–liquid residuals in that the liquid residual (water) contains suspended solids. Other solid–liquid residuals produced in the water industry include all polymer coagulation wastes, residuals from iron or manganese removal plants, spent GAC, spent precoat filter media, and residuals from slow sand filter plants.

QUANTITIES

Coagulant Solids

The quantity of solid–liquid residuals (which are commonly referred to as sludge) generated from WTPs depends on the raw water quality, dosage of chemicals used, the performance of the treatment process, and the method of sludge removal.

One of the most difficult tasks facing the utility or engineer in planning and designing a residuals treatment process is determining the amount of residual to be handled. The residual quantity is usually determined as an annual average for a given design year and is a function of flow projections. Information about seasonal and monthly variations is as important as the average production values. It is not unusual for order-of-magnitude differences in sludge production to exist for different months of the year. Because determining quantities of waste produced requires a history of data compilation, it is wise for a utility to begin collecting these data even if there are no immediate plans to implement a new waste management program. Such a data analysis process can even provide insights to options to ease burdens on existing systems and methods to increase efficiency of the water treatment process itself.

There are three methods used for determining sludge quantities. None are completely accurate; therefore, it is advisable to use and cross-check one against another. These methods are calculations, coagulant mass balance analysis, and field determination. Each is discussed in the paragraphs that follow.

Using the calculation method, the amount of alum (or iron) sludge generated can be calculated fairly closely by considering the reactions of alum or iron in the coagulation process. Using an empirical relation to account for the sludge contribution from turbidity will improve the estimate, and the contribution from other sources can be added as required.

When alum is added to water as aluminum sulfate, the reaction is typically represented by the simplified equation:

$$Al_2(SO_4)_3{\cdot}14H_2O + 6HCO_3^- = 2Al(OH)_3 + 6CO_2 + 14H_2O + 3SO_4^{-2} \tag{2.1}$$

If adequate alkalinity is not present, then lime, sodium hydroxide, or soda ash is normally added to maintain the proper pH. If equilibrium was achieved, the aluminum hydroxide, as shown, would be the predominant solid product. However, equilibrium is not normally obtained, and a complex polymerized compound containing, on average, three to four water molecules bound to the aluminum hydroxide is formed as the precipitate. This chemically bound water increases the sludge quantity and the sludge volume and also makes it more difficult to dewater since the chemically bound water can not be removed by normal mechanical methods. The resulting aluminum hydroxide species is closer to the form $Al(OH)_3{\cdot}3H_2O$ which has a molecular weight (MW) of 132, and, therefore, 1 mg/L of alum added to water will produce approximately 0.44 mg/L of inorganic aluminum solids, as shown in Equation 2.2.

The molecular weight of alum used in WTPs is 594, and each mole of alum added produces 2 moles of aluminum hydroxide with three waters of hydration. Therefore, the weight of coagulant solids produced per coagulant added can be found as

$$\frac{2(\text{MW solids})}{\text{MW alum}} = \frac{2(132)}{594} = 0.44\frac{\text{mg solids produced}}{\text{mg alum added}} \tag{2.2}$$

Suspended solids present in the raw water produce an equivalent weight of sludge solids since they are nonreactive. It can be assumed that other additives such as polymer and PAC produce sludge on a one-to-one basis. The amount of sludge produced in an alum coagulation plant for the removal of turbidity is then

$$S = 8.34\ Q\ (0.44\ \text{Al} + SS + A) \tag{2.3}$$

where S = sludge produced (lb/d dry weight)
Q = plant flow (mgd)
Al = alum dose as 17 percent Al_2O_3 (mg/L) (MW = 594)
SS = raw water suspended solids (mg/L)
A = additional chemicals added such as polymer, clay, or activated carbon (mg/L)

If iron is used as the coagulant, then the equivalent product is $Fe(OH)_3{\cdot}3H_2O$ with a molecular weight of 161. The solids production equation becomes:

$$S = 8.34\ Q\ (2.9\ \text{Fe} + SS + A) \tag{2.4}$$

where the iron dose is expressed as milligrams per liter of Fe^{3+} added or produced via Fe^{2+} oxidation (note that significant Fe^{2+} in the raw water will also produce sludge at a factor of 2.9 if it is oxidized). The above two equations should not be interpreted to suggest that iron produces several times the amount of sludge that alum produces. In the case of aluminum, the dose is expressed as alum with a molecular weight of 594; in the case of iron, the dose is expressed as iron with a molecular weight of 55.8. In reality, 1 mole of coagulating equivalent of iron produces about 20 to 25 percent more dry weight sludge than 1 mole of aluminum.

When iron is purchased as ferric chloride ($FeCl_3$), the coagulant dose is usually reported as equivalent dry weight of chemical without waters of hydration (although this should be confirmed with the manufacturer) and, thus, the coagulant has a molecular weight of 162.3. This results in the production of 1.0 mg of solids produced for each milligram of ferric chloride added. Ferric sulfate is reported by different manufacturers with different waters of hydration, and the individual products need to be referenced.

Polyaluminum chloride (PACl) is the third major coagulant used. Care especially needs to be used when converting PACl dose to solids production, because each manufacturer may use different strengths and utilities report these doses differently. Some utilities report PACl dose as a neat solution, some as aluminum oxide (Al_2O_3), and some as PACl product. A “typical” manufactured PACl liquid contains about 30 to 35 percent PACl and around 10 percent Al_2O_3. One manufactured product contains 33.3 percent PACl and 10.3 percent Al_2O_3 or, in this case, the PACl itself contains 30.9 percent Al_2O_3. This is equivalent to 16.4 percent aluminum, and therefore 1 mg of PACl (as PACl) will produce 0.8 mg of solid product (0.164×4.89).

The above equations can then be used to track yearly or even daily changes in sludge weight produced. The difficulty in applying the relationships is that most plants do not routinely analyze raw water suspended solids concentrations. The logical correlation is to equate a turbidity

(Tu) unit to a suspended solids unit. Unfortunately, the relationship is generally not necessarily one to one:

$$SS\ (\text{mg/L}) = \text{b} \cdot \text{Tu} \tag{2.5}$$

The value of b for low color, predominantly turbidity removal plants, can vary from 0.7 to 2.2. It may vary seasonally for the same raw water supplies; therefore, a utility can either continually measure suspended solids or it may be possible to develop a site-specific correlation between turbidity and suspended solids. Ideally, this correlation should be done weekly until information is learned as to seasonal variations in the suspended solids turbidity relationship, whereupon a monthly correlation may be sufficient. Care needs to be used when conducting raw water suspended solids analysis. Any filterable solids should be removed by coagulation; therefore, it is recommended that 0.45-µm paper be used for the suspended solids analysis. In many raw waters, <5 mg of weight may be in 1 L of water; therefore, several liters of water may need to be filtered for accurate results. Each individual utility will need to determine its exact procedure for accuracy. Oven temperature is also important. For raw water suspended solids analysis, the standard temperature of 103°C is generally sufficient.

Another complication exists for raw water sources that contain a significant amount of color. Color, perhaps indicated by total organic carbon (TOC), can be a large contributor to the sludge production. For many waters, color and nondissolved TOC is filterable on a 0.45-µm filter and will therefore be measured in a suspended solids analysis. In this case, TOC is accounted for in the suspended solids analysis. Values of b for colored raw waters can be as high as 20, but unless turbidity and color vary together, a correlation between SS and Tu will not exist. For waters that a high, nonfilterable TOC (called dissolved organic carbon [DOC]) is removed, that DOC removed should be added to the equation.

Overall, therefore, the equations might be generalized as

$$S = 8.34\ Q\ (aC + \text{bTu} + DOC_R + A) \tag{2.6}$$

where a = solids produced/coagulant addition
C = coagulant addition (mg/L)
b = SS/Tu ratio
Tu = raw water turbidity (ntu)
DOC_R = nonfilterable raw water TOC removed (mg/L)
A = additional solids-producing chemicals added

The second method that can be used to estimate total sludge weight produced for coagulant sludge is to conduct a conservation of coagulant mass balance analysis. This method is based on conservation of the coagulant; that is, whatever is added in the coagulation process ends up in the sedimentation basin solids, backwash solids, or finished water. The first step is to analyze the aluminum (or iron) content of the coagulant. As an approximation, it can be assumed that dry weight alum is 9.1 percent aluminum. Because iron coagulants vary, the iron content must be determined or obtained from the manufacturer. A series of sludge, SFBW, and finished water samples are then collected and analyzed for aluminum (or iron). The pH of the solids is lowered to 1 and maintained for 10 to 15 minutes. The sample is then filtered and the filtrate analyzed for aluminum (or iron). This method will tend to solubilize the aluminum hydroxide but not solubilize the aluminum in the clays that may be present. A second aliquot of unacidified sludge is analyzed for suspended solids concentration. When conducting suspended solids tests on hydroxide sludges, an oven temperature of 140°C should be used, rather than the normal 103°C. Use of 140°C will drive off the sorbed water (not the 3 moles of bound water). Sorbed water is not part of the solids production. Care should be used to be certain the weight has stabilized. The total amount of sludge can then be computed.

For example, if a 10-mgd (38-ML/d) plant uses 30 mg/L alum, and the sedimentation basin sludge is analyzed to contain 2,000 mg/L Al at 2.5 percent solids concentration, how much sludge is produced per day? Ignore SFBW and finished water aluminum for this example.

$$\begin{aligned}\text{total Al added/day} &= 10 \text{ mgd} \times 8.34 \times 9.1 \text{ percent} \times 30 \text{ mg/L}\\ &= 228 \text{ lb (103 kg)}\end{aligned}$$

Because the sludge contains 2,000 mg/L Al, there must be 51,756 L of sludge ($1.03 \times 10^8/2{,}000$) (or 13,800 gpd [52 m^3/d]). At 2.5 percent solids concentration, there would be 2,877 lb (1,305 kg) of dry weight sludge produced per day (13,800 × 8.34 × 2.5 percent). By collecting a series of sludge samples from various locations and depths from within a basin, it is possible to formulate an accurate estimate of the amount of sludge produced. A complete mass balance should be done to include aluminum in the backwash solids and the finished water.

The third method for determining sludge quantities is through field determination. This is probably the most difficult method to obtain accurate results, unless the utility already has continuous sludge collection equipment with a monitoring system. If the utility already has a mechanical dewatering system, then truck tare weights can be obtained, and, in conjunction with carefully conducted solids analyses, the amount of sludge produced can be determined. However, a utility usually needs to determine the amount of sludge produced before the installation of facilities, and often the plant has manually cleaned sedimentation basins or does not have a way to continually measure sludge flow and dry weight.

In order to conduct an estimate by field determination, all basins should be cleaned. Then a specific period of time should be allowed for sludge to collect in the basins. Next, using a "sludge judge," a cross section of sludge depth can be made in each basin.* The sludge judge can also be used to collect a composite depth of sludge from several locations in the basins for suspended solids analyses. With the results of suspended solids analysis and the calculation of sludge volume in the basins, a very rough projection of sludge production can be determined. However, it is highly recommended that this procedure be supplemented by use of the other two procedures. Another method that can be employed is to measure the suspended solids in the flocculation compartment. Several samples are needed, as uniform mixing often does not occur in flocculators.

The City of Philadelphia, Pa. (Philadelphia Water Department, or PWD) conducted a study to determine the residuals production of its three WTPs using both the calculation and field determination methods.

Table 2.1 summarizes specific information for each water treatment facility that would be relevant to the overall planning of residuals management facilities. The flocculation–sedimentation basin routine and annual cleaning volumes shown for each plant were derived from data generated

---

* A sludge judge is a clear acrylic tube with a foot valve to allow collection of a cross-sectional sample.

Table 2.1
Overview of existing residuals-generating processes at PWD

Parameter	Baxter WTP	Queen Lane WTP	Belmont WTP
Treatment capacity (mgd)			
Rated	282	120	80
Average	200	107	65
Raw water basin			
No. of basins	1	1	2
Volume, each (MG)	176	214	43
Flocculation–sedimentation basins			
No. of basins	4	4	4
Volume, each (MG)	9.5	10	3.0 to 4.1
Residuals collection type	Circular	Chain and flight	Circular
Flocculation–sedimentation residuals volume (gal)			
Routine, average	270,000	961,000	40,000
Routine, maximum	360,000	1,300,000	42,000
Cleanings, average	320,000	157,000	245,000
Cleanings, maximum	736,000	372,000	537,000
Filtration			
No. of filters	94	40	26
Filter area, each (ft^2)	1,044	1,044	1,044
Filter to waste	None	None	None
Filters washed per day			
Average	48	48	19
Maximum	116	95	46
Wash-water volume (MG)			
Per backwash	0.12	0.15	0.12
Average day	5.7	7.3	2.4
Maximum day	16.5	17.5	8.3

NOTES: mgd × 3.785 = ML/d; MG × 3.785 = ML; $ft^2 \times 0.93 = m^2$.

by PWD. Filter backwash frequencies and volumes shown in Table 2.1 were obtained from plant operating records.

The following paragraphs provide an overview of each WTP and a description of the existing residuals management practices.

The Baxter WTP is a conventional surface WTP with a rated capacity of 282 mgd (1,070 ML/d) and a current average water production rate of 200 mgd (757 ML/d). The raw water turbidity ranges seasonally from 4 to 13 ntu, but can be significantly higher following rainstorm events. Ferric chloride is used as a primary coagulant, and lime is added for pH adjustment. Raw water is drawn by gravity from the Delaware River into the raw water basin.

- Flocculation–sedimentation basins—The plant has four flocculation–sedimentation basins, each equipped with circular collectors to provide for daily residuals removal.

Residuals from each flocculation–sedimentation basin are discharged for 3 hours each day, one basin at a time, at an estimated rate of 500 gpm (135 m^3/hr). Two basins are routinely cleaned in the morning and two in the afternoon. Flocculation–sedimentation basin residuals can flow by gravity to the raw water basin, but the flocculation–sedimentation must be pumped to the sewer. When all four flocculation–sedimentation basins are in service, approximately 360,000 gal (1.36×10^6 L) of flocculation–sedimentation basin residuals are discharged per day.

Each basin is taken out of service for cleaning about once per year. This operation consists of draining each basin (9.5 MG [36 ML]) over an 8- to 16-hour period, followed by 4 to 6 weeks of removing the accumulated solids with fire hoses. The quantity of solids removed during each basin cleaning is estimated by PWD to average 400 DT (3.6×10^5 kg).

- Filters—The Baxter plant has 94 filters (67 with sand media and 27 with dual media). The design rate is 2.0 gpm/ft^2 (4.9 m/hr), and the filter surface area is 1,044 ft^2 (97 ML) per filter. The filter backwash rate ranges from 15,000 and 22,000 gpm for approximately 8 minutes in the winter and summer, respectively. The daily filter backwash water usage ranges from 5.7 to 16.5 MG (21 to 62 ML). This corresponds to 2.9 to 8.2 percent of the average daily treatment capacity. Currently, the elevated backwash tanks are out of service and the filters are washed directly with the backwash tank fill pumps. Two filters, one on each side of the plant, can be backwashed simultaneously. The spent backwash water flows by gravity to the raw water basin and is ultimately returned to the treatment plant.

The Queen Lane WTP has a rated capacity of 120 mgd (450 ML/d) with an average water production rate of 90 mgd (340 ML/d). Raw water is pumped from the Schuylkill River and settled in the raw water basin. The raw water turbidity ranges seasonally from 2 to 18 ntu, but can be significantly higher after rainstorm events. Both alum and ferric chloride can be used as a primary coagulant, although ferric chloride is preferred. Lime is added for pH adjustment.

- Flocculation–sedimentation basin—The plant has four 10-MG (38-ML) flocculation–sedimentation basins, each with an upper and lower level. The upper levels are equipped with chain and flight collectors. Operation of the collectors is continuous 24 hours per day as is the routine discharge of the flocculation–sedimentation residuals. The residuals are pumped from each basin at approximately 225 gpm (14 L/sec) for a total flow of 900 gpm (57 L/sec) or 1.3 mgd (5.0 ML/d) for all four basins.

  The flocculation–sedimentation basins are taken out of service at least once each year for annual cleaning. This occurs in the spring and/or the fall and removes about 200 tons (1.8×10^5 kg) of solids per basin per cleaning. A cleaning event consists of draining the upper level over a period of 12 hours. The lower level, which does not have collectors, drains over a 3-day period.
- Filters—The Queen Lane WTP has 40 filters (22 with sand media and 18 with dual media). The design filtration rate is 2.0 gpm/ft^2 (4.9 m/hr), and the filter surface area is 1,044 ft^2 (97 m^2) per filter. The filter backwash rate is 22,000 gpm (1,320 L/sec) for the sand filters and 18,500 gpm (1,110 L/sec) for the dual-media filters. The backwash duration is approximately 7 minutes. The daily filter backwash water usage ranges

from 7.3 to 17.5 MG (28 to 66 ML). This is equivalent to 6.8 to 14.9 percent of the average water treatment rate.

The Belmont WTP has a rated capacity of 80 mgd (300 ML/d) with an average water production rate of 65 mgd (246 ML/d). Raw water is obtained from the Schuylkill River. The raw water turbidity ranges from 3 to 28 ntu, and ferric chloride is used as the primary coagulant.

- Flocculation–sedimentation basins—The plant has four flocculation–sedimentation basins (two at 4.1 MG [16 ML] and two at 3.0 MG [111/L]), each equipped with circular collectors to allow routine removal of the flocculation–sedimentation basin residuals. Routine discharges occur for 3 minutes per basin per shift at an estimated discharge rate of 2.6 cfs or 1,167 gpm (70 L/sec). When all four basins are in service, there is an estimated 3,500 gal (1.3×10^4 L) sedimentation basin discharge every 2 hours for a total of 42,000 gpd (160 m^3/d). Each flocculation–sedimentation basin is cleaned annually either in the spring or fall. A cleaning event consists of draining the basin and then hosing out the solids accumulated in the corners and floc compartments. This cleaning operation takes about 10 days, including a gradual draining of the basins. The volume discharged is estimated by multiplying the total time of hosing each day by an estimated hose flow rate of 86 gpm (5 L/sec).
- Filters—The Belmont WTP has 26 filters, each with a filter surface area of 1,044 ft^2 (97 m^2). Filters are backwashed at 22,000 gpm (1,320 L/sec) and 14,000 gpm (840 L/sec) in the summer and winter, respectively, for 8 minutes. Two filters must be taken out of service at the same time for backwashing, but only one filter is backwashed at a time. Backwashes are kept to one filter each half-hour to minimize power consumption and costs. The daily backwash water usage ranges from 2.4 to 8.3 MG (9 to 31 ML). This corresponds to 3.6 to 11.7 percent of the average treated water rate. Under worst-case conditions (which occur in the summer), up to 26 filter backwashes have been experienced per day, with two thirds of the filters being backwashed on the night shift.

Operating records for the Baxter, Queen Lane, and Belmont WTPs were analyzed to quantify historical residuals production. Monthly residuals production rates were determined by incorporating average monthly raw water basin effluent flow and turbidity values along with ferric chloride, lime, alum, and PAC dosages in the following empirical equation:

$$S = 8.34 \times Q \times (\mathrm{bTu} + 1.0\ \mathrm{Fe} + 0.1\ \mathrm{lime} + 0.44\ \mathrm{Al} + \mathrm{PAC}) \tag{2.7}$$

where S = residuals production (lb/d)
Q = plant flow (mgd)
b = ratio of total suspended solids (TSS) to turbidity
Tu = raw water turbidity (ntu) after presedimentation
Fe = ferric chloride dose (mg/L)
lime = lime dose (mg/L)
Al = alum dose (mg/L)
PAC = powdered activated carbon dose (mg/L)

Lime was included at a 10 percent factor to account for impurities.

Table 2.2
Mass balance sampling results for PWD over 1 year

Parameter	Baxter WTP	Queen Lane WTP	Belmont WTP
Raw water basin effluent			
Turbidity (ntu)	7.2	6.1	5.1
TSS (mg/L)	9.9	9.7	6.3
TSS/turbidity	1.4	1.5	1.5
Sedimentation basin efficiency			
Influent TSS (mg/L)	28.8	23.3	11.9
Removal (% of total)			
Average	93	88	77
Maximum	98	98	96
Minimum	61	59	41

A mass balance sampling program was initiated to determine the b-factor for each of PWD's WTPs. Data are summarized in Table 2.2 and reveal that the ratio of raw water basin effluent TSS to turbidity is 1.4 for the Baxter plant, 1.5 for Queen Lane, and 1.5 for Belmont.

To calculate the historical residuals production for the WTPs, pertinent monthly operating data for the Baxter, Queen Lane, and Belmont WTPs were obtained. Key historical operating data and residuals production for each plant are summarized in Table 2.3. The residuals production is shown as average dry pounds per day and pounds of solids produced per million gallons of water treated (lb/MG). The latter characterizes variations due to raw water quality. The 90th percentile value represents about a maximum month condition. This value does not include basin cleaning operations which would result in a higher monthly residuals production when the basins are cleaned.

The historical trends for the three PWD plants are as follows:

- The average residuals generation rate for the Baxter WTP is estimated at 140 lb/MG (17 kg/ML). Seasonal variations (spring and fall) in residuals production range from 100 to 200 lb/MG (12 to 24 kg/ML). A peak residuals generation of 391 lb/MG (47 kg/ML) occurred in February 1994 because of high turbidity (20 ntu) coming out of the raw water basin.
- The average residuals generation rate for the Queen Lane WTP is 189 lb/MG (2 kg/ML), with typical variations between 150 and 350 lb/MG (18 and 42 kg/ML). The variations are primarily due to raw water quality changes.
- The average residuals generation rate for the Belmont WTP is estimated at 179 lb/MG (22 kg/ML), with typical variations between 90 and 350 lb/MG (11 and 42 kg/ML).

PWD has measured residuals discharge quantities from the Baxter, Queen Lane, and Belmont WTPs by estimating the discharge volumes and applying the results of routine solids analyses to these volumes. The measured residuals discharge quantities are summarized in Table 2.4.

Table 2.3
Summary of 4-year historical operating data and calculated residuals production for PWD

Parameter	Baxter WTP	Queen Lane WTP	Belmont WTP
Average raw water basin effluent conditions			
Flow (mgd)	199	107	65
b-factor	1.4	1.5	1.5
Turbidity (ntu)	4.5	6.6	7.2
Ferric chloride dose (mg/L)	11.5	14.8	11.7
Lime dose (mg/L)	12.9	5.2	6.1
Alum dose (mg/L)	0.0	1.0	1.3
PAC dose (mg/L)	0.0	0.0	0.0
Residuals quantities			
Average daily production (lb/d)	27,636	20,327	11,608
Average residuals generation (lb/MG)	140	189	179
90th percentile (lb/d)	37,915	29,755	17,503

NOTES: lb/d × 0.45 = kg/d; lb/MG × 0.12 = kg/ML; mgd × 3.785 = ML/d.

A comparison of 3 years and average annual calculated residuals production to the measured residuals quantities is shown in Figure 2.1. A good correlation exists at each plant for the 3-year running average. At the Baxter plant, the calculated quantities are within 5 percent of the measured quantities (32,234 versus 30,707 lb/d [14,620 versus 13,930 kg/d]). At the Queen Lane plant, the measured quantities are 26,903 lb/d (12,200 kg/d) versus 22,030 lb/d (9,990 kg/d) for the calculated quantities. At Belmont, the difference is about 0.5 percent (13,087 lb/d [5,940 kg/d] calculated versus 13,159 lb/d [5,970 kg/d] measured).

Lime Residuals

Through similar theoretical considerations, a general equation has been developed for plants that use a softening process with or without the use of alum, iron, or polymer:

$$S = 8.34\,Q(2\ \mathrm{CaCH} + 2.6\ \mathrm{MgCH} + \mathrm{CaNCH} + 1.6\ \mathrm{MgNCH} + \mathrm{CO_2} + 0.44\ \mathrm{Al} + 2.9\ \mathrm{Fe} + SS + A) \tag{2.8}$$

where S = sludge production (lb/d)
CaCH = calcium carbonate hardness removed as $CaCO_3$ (mg/L)
MgCH = magnesium carbonate hardness removed as $CaCO_3$ (mg/L)
Fe = iron dose as Fe (mg/L)
Al = alum dose as 17.1 percent Al_2O_3 (mg/L)
Q = plant flow, mgd
SS = raw water suspended solids (mg/L)
A = other additives (mg/L)

Table 2.4
Summary of historical measured residuals production at PWD

	Quantity (lb/d)		
Parameter	Baxter WTP	Queen Lane WTP	Belmont WTP
Routine flocculation–sedimentation basin discharges			
Average annual	15,790	16,160	9,478
Maximum month	28,822	31,700	19,526
Maximum day	ND*	72,507	28,189
Flocculation–sedimentation basin cleanings			
Average annual	8,800	3,350	1,073
Maximum month	57,142	21,898	6,583
Maximum day	ND	113,487	46,101
Total flocculation–sedimentation basin solids			
Average annual	24,590	19,511	10,543
Maximum month	ND	42,537	19,526
Maximum day	ND	151,106	55,063
Backwash water solids			
Average annual	5,631	7,481	2,447
Maximum month	9,247	12,310	4,586
Total WTP residuals			
Average annual	30,221	26,992	12,990
Maximum month	ND	46,982	24,112
Maximum day	ND	155,551	57,307

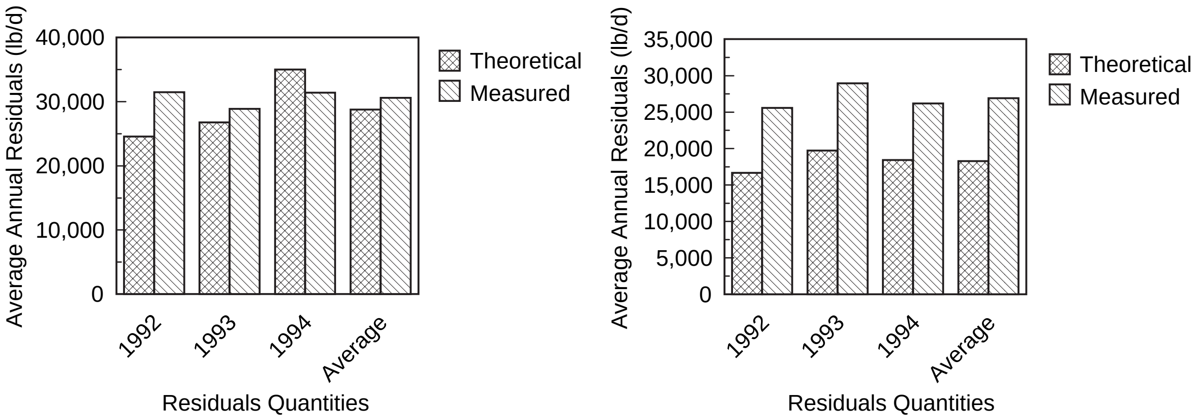
NOTES: lb/d × 4.536×10^{-1} = kg/d.
*ND = not detected.

CaNCH = noncarbonate calcium hardness removed as $CaCO_3$ (mg/L)
MgNCH = noncarbonate magnesium hardness removed as $CaCO_3$ (mg/L)
CO_2 = carbon dioxide removed by lime addition, as $CaCO_3$

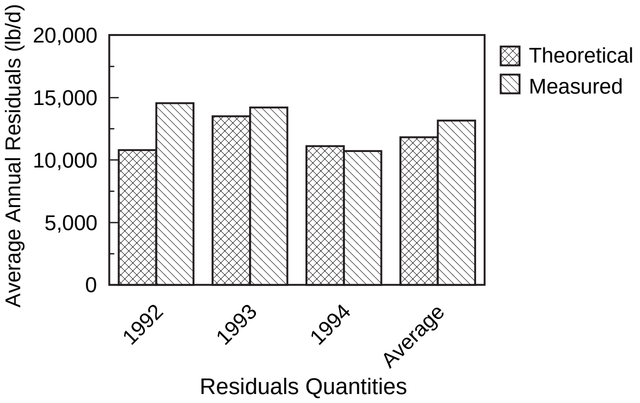
Equation 2.8 will allow estimation of the dry weight of sludge produced. Impurities associated with lime can be added to this.

RESIDUAL PHYSICAL CHARACTERISTICS

The physical properties of WTP residuals affects the ultimate requirements for managing the residual stream or solid. The most important initial characteristic is whether the residual is predominately a liquid, that is, essentially water-containing solid particles, or whether the residual is essentially a solid or semisolid material. Physical properties of predominately liquid residuals that contain solids are used to define the ease of solid separation from the water. The characteristic tests are useful for comparing different treatment or conditioning techniques on a laboratory scale to predict full-scale performance. Physical characteristics testing on solid or semisolid residuals are useful for understanding handling and compaction for the material.



(a) Baxter WTP **(b) Queen Lane WTP**



(c) Belmont WTP

Figure 2.1 Comparison of theoretical and measured residuals quantities at PWD

The major constituent of any of the water plant residuals is water. In the case of suspended solids in the water, the treatment objective is generally one of solid–water separation. When the waste contains predominantly dissolved solids in the water, either a phase change is required to remove the total dissolved solids (TDS) or the water and TDS are jointly disposed of in some acceptable manner.

Those wastes in which the water contains suspended solids are traditionally referred to as sludges. Vesilind (1988) classified the water content of wastewater sludges into four categories:

1. Free water is not held to sludge solids and can be removed by simple gravitational settling
2. Floc water is trapped within sludge flocs and can be removed by mechanical dewatering
3. Capillary water is held to sludge solids by surface tension and attractive forces and can be removed only by compaction and deformation of the sludge flocs
4. Bound water is chemically bound to the individual floc particle and cannot be removed

For chemical residuals, Cornwell (1978) has proposed three water classifications:

1. Free water can be removed by drainage or low-pressure mechanical methods
2. Hydrogen-bound water is attracted to the floc particle through hydrogen binding. The force of attraction of the water to the chemical floc is in the range of 0.13 kcal/mole.
3. Chemically bound water is bound through covalent bonds directly to the chemical floc

The practical difference between a liquid and solid residual in water treatment is usually visually obvious. The predominant liquid streams are free flowing. However, there can be a transition phase between a free-flowing residual and a solid residual that can be harder to observe. In soil science a test referred to as the Atterberg limit test quantitatively defines these differences, and the test has been found to be useful in describing WTP residuals.

The Atterberg limit test consists of five "limits" proposed by Atterberg in 1911 to quantitatively describe the effect of varying the water content on the consistency of fine-grained soils. The five limit tests are cohesion, sticky, shrinkage, plastic, and liquid. The liquid and plastic limits have been used primarily for soil identification and classification. Clay soils, or sludges in this case, can be classified as exhibiting either solid, plastic, or liquid behavior, depending on their solids content. The plastic limit identifies the solids concentration at which a sludge transitions from a semisolid to a plastic stage. The consistency of a material in the plastic stage could be described as ranging from that of soft butter to stiff putty. The liquid limit is the solids concentration below which the sludge exhibits viscous behavior; the consistency could be described as ranging from soft butter to a pea soup–type slurry.

From a practical sense with WTP residuals, the liquid limit obtained by this test is somewhat higher than practical for pumping or handling as a true liquid. Cornwell et al. (1992) found the liquid limit of two alum residuals to range from 15.4 to 19.1 percent solids concentration and 48.1 percent for a lime residual. These solids concentrations are in the range of typical for residuals that have been treated by mechanical means for water and solid separation. The liquid limit test in the laboratory can be useful in estimating the achievable level by mechanical dewatering.

Several tests are available to characterize a predominately liquid residual, and several are useful for a predominately solid residual. Each are discussed in the next sections.

Characteristics of Predominantly Liquid Sludge

Knocke and Wakeland (1983) have divided physical properties of sludges into "macroproperties" and "microproperties." Macroproperties are such parameters as specific resistance, settling rates, and cake solids concentrations. Microproperties would include particle size distribution and density. Knocke has evaluated several of these microproperties and their effect on dewatering of alum and lime sludges. In general he found alum residuals that are predominantly aluminum hydroxide (low-turbidity raw waters) and had densities of 1.002 to 1.008 g/cm^3 and correspondingly dewatered to a 7 to 19 percent solids concentration by laboratory vacuum filtration. Alum sludges from higher-turbidity raw waters had densities of 1.008 to 1.018 g/cm^3 and dewatered in the lab to a 20 to 34 percent solids concentration. The lime sludge evaluated had a density of 1.292 g/cm^3. An evaluation of floc shape for alum sludges showed the particles to be elliptical, with a ratio of minor to major axes between 0.5 and 0.75. The axis length for the major axes was in the range of 5 to 50 μm for unconditioned alum sludges and 5 to 100 μm for polymer conditioned sludges. Although a wide scatter in the data existed, Knocke found that, generally, the larger the floc particle (longer axis length), the lower the specific resistance value. Polymer addition had the effect of increasing the particle size, resulting in a decrease in sludge specific resistance.

Tests that define macroproperties of sludges can be used to assist in the selection of dewatering aids and to determine relative ease of dewatering. They can be useful as an operating tool to determine conditioning doses on a routine basis.

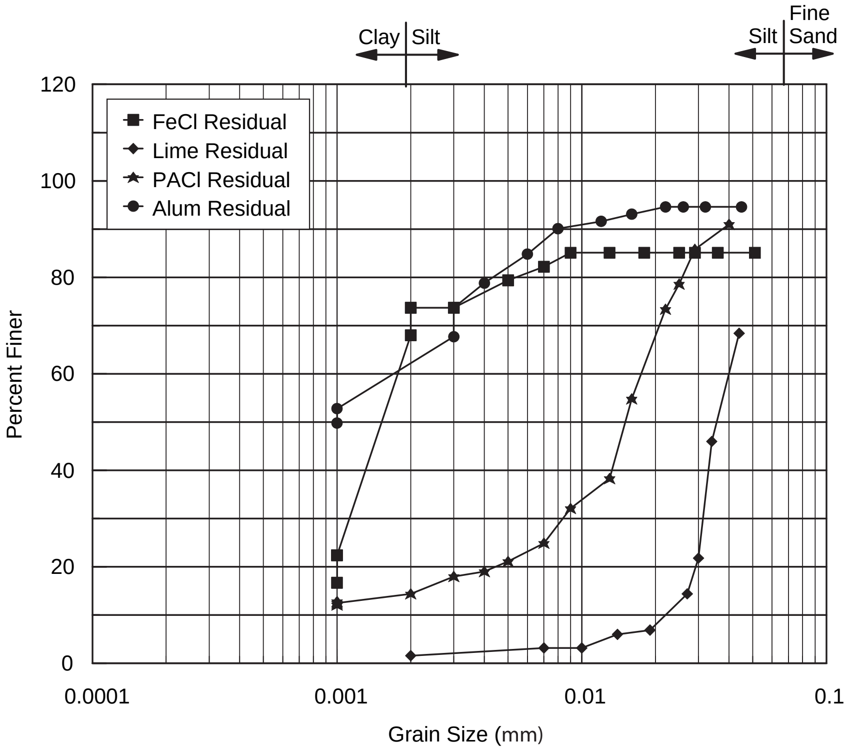
Microproperty testing can include grain size analysis, density, and specific gravity. Many methods exist to count and size the particles in a sample. Most commonly used in the water industry are analysis by microscope, resistance-based particle size analyzers and laser light blockage–based particle size analyzers. For residuals analysis, some researchers have used laser light blockage–based particle size analyzers, but these instruments have limitations that make them difficult to use in this application. The instruments have an upper concentration limit of around 15,000 particles/mL. Residual samples have more particles than these limits. In order to analyze residuals samples on these instruments, the sample must be diluted to an extent that may impact the reliability of the result. In addition, the samples are often pumped through instruments, which could cause particle shearing.

Particle size distribution has been successfully analyzed using a modification of a test commonly used in soils research, the hydrometer test (American Society for Testing and Materials [ASTM] D422-63; ASTM 1993) (Vandermeyden and Cornwell 1998). The test measures particles smaller than 0.075 mm, including silt and clay particles (0.05 to 0.001 mm). Since particles of this size are predominant in water plant residuals, the test is applicable for analyzing residual samples.

The test is based on the theory that solid particles dispersed in suspension settle at different rates based on the size and specific gravity of the particles. In the standard hydrometer test for soils, a 50-g sample of soil material is dried and weighed prior to analysis. The dried sample is then mixed with a deflocculating agent to prevent flocculation of small particles through Brownian motion during the test. Vandermeyden and Cornwell (1998) found that the drying portion of this test was impractical for residuals analysis since, when dried, the samples became strongly bonded and could not be separated into a suspension, even after prolonged soaking in the deflocculating agent. Further, a 50-g sample did not allow the hydrometer to sink into the suspension. Therefore, the ASTM method was modified by reducing the sample size to approximately 10 g and by eliminating the drying step. Instead, the sample was added directly to the deflocculating agent, and the amount of solids in the suspension was determined after the experiment.

After mixing the sample with the deflocculating agent, the mixture was placed in a graduated cylinder with the hydrometer. The hydrometer is used to measure the amount of material that has settled past the zone of measurement during a given period of time. Elapsed time and hydrometer readings are recorded during the test. Settling velocity can then be calculated as the quotient of distance and time. Particle diameter is then determined from the velocity data through an application of Stoke’s law.

Results of the hydrometer test are usually plotted on semilogarithmic paper. As in Figure 2.2, percent finer is plotted versus particle diameter, with particle diameter, or grain size, plotted on the logarithmic scale. With the information presented in this format, percentages of fine sand-, silt-, and clay-sized particles as well as median diameter, d_{50}, are easily obtained. The amount of each fraction is plotted as a cumulative distribution requiring some interpretation to obtain each fraction. The amount of fine sand, silt, clay, and the d_{50} can be determined as follows:



Source: Vandermeyden and Cornwell 1998.

Figure 2.2 Typical grain size analysis using the hydrometer method

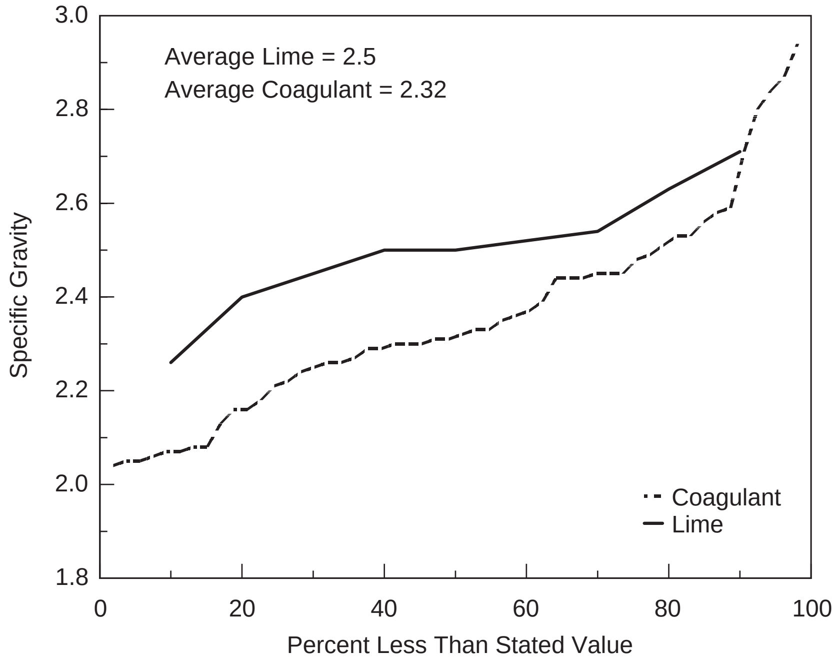
Classification	Diameter (mm)
Fine sand	>0.075
Silt	Between 0.075 and 0.002
Clay	<0.002

The specific gravity of the solid material contained within the liquid matrix can also be determined during the hydrometer test. Vandermeyden and Cornwell (1998) reported specific gravities of between 2.0 and 3.0, with an average of 2.39. Figure 2.3 shows the distribution of specific gravity found by the researchers in coagulant and lime produced solids.

Several physical tests are available to classify the dewatering or drainage properties of the liquid containing solids materials, including the capillary suction time (CST) test, the specific resistance (SR) test, and the time to filter (TTF) test.

The SR test has been used to optimize sludge dewatering performance. The test is most valuable when evaluating chemical conditioning of sludge for full-scale applications.

A simple Buchner funnel apparatus can be used to determine the specific resistance of water plant sludges. Typically 100-mL portions of the sludge are added to the Buchner funnel and



Source: Vandermeyden and Cornwell 1998.

Figure 2.3 Specific gravity distribution for coagulant and lime residuals

the volume of filtrate generated at various times is recorded. Based on the Carmen–Kozeny equation for flow through porous media, an equation can be developed to describe the flow through a sludge cake and associated support media. The equation is written as follows:

$$\frac{t}{V} = \frac{\mu WSR}{2PA^2}V + \frac{\mu SR_r}{AP} \tag{2.9}$$

where: t = time of filtration (sec)
V = volume of filtrate (mL)

which is of the form:

$$\frac{t}{V} = bV + a \tag{2.10}$$

Therefore, a plot of t/V versus V should give a straight line with a slope “b” and intercept “a.” Since the slope of the line b equals

$$\mathrm{b} = \frac{\mu WSR}{2PA^2} \tag{2.11}$$

then the specific resistance is

$$SR = \frac{2bPA^2}{\mu W} \tag{2.12}$$

where SR = specific resistance (m/kg) of sludge
b = slope of line
P = vacuum applied (N/m^2)
A = filter area (m^2)
μ = filtrate viscosity (N•sec/m^2)
W = dry weight of solids per volume of filtrate (kg/m^3)
SR_r = specific resistance (m/kg) of filter media

Note that these are the proper Système International (SI) units. It had been traditional to report SR as sec^2/g, which are not SI units. One can convert from sec^2/g to m/kg by multiplying 9.81 by 10^3.

By repeating the SR test for untreated residuals and residuals receiving various chemical conditioning, it is possible to determine the optimum chemical treatment. However, this is optimum only at the residuals solids concentration used in the experiment. The SR test is dependent upon the suspended solids concentration and, therefore, it is difficult to compare various researchers' results. Table 2.5 shows specific resistance values for several water plant sludges (Calkins and Novak 1973).

While specific resistance data could theoretically be used to size full-scale equipment, this is not a recommended practice. The test can be useful for conditioning studies, however.

Most sludges are compressible, and the degree of compression appears to depend on the vacuum applied and the geometry of the dewatering system. An empirical expression has been found to adequately relate specific resistance and the vacuum level. This expression states that

$$SR = CP^S \tag{2.13}$$

where SR = specific resistance
C = cake constant
P = vacuum applied
S = coefficient of compressibility

Both the cake constant, C, and the coefficient of compressibility, *S*, can be determined from a logarithmic plot of specific resistance versus the vacuum level. The coefficient of compressibility, *S*, is the slope of the straight line generated while the cake constant, C, is the intercept where P = 1. The coefficient of compressibility equals zero for an incompressible sludge. Hamon (1986) found that the coefficient of compressibility varied from 0.6 to 0.8 for aluminum hydroxide sludges and 0.71 to 0.83 for ferric hydroxide sludges. Knocke and Wakeland (1983) reported values of 0.97 for alum sludges and 0.8 for lime sludges.

Figure 2.4 shows results of specific resistance as a function of polymer dose for alum sludge at Durham, N.C. The test was conducted using a 100-mL and a 50-mL sample. The 100-mL sample gave better results, primarily because more accurate volume-versus-time data can be collected during the test procedure.

Table 2.5
Specific resistance for various chemical sludges

Sludge	Location*	Specific resistance 10^6 (sec^2/g)†
Lime and iron	Jefferson City	2.11
Lime and iron	Jefferson City	4.3
High magnesium softening sludge	Kansas City	5.49
Lime and alum	Boonville	5.83
Excess lime and alum backwash	Boonville	5.98
Lime and iron	Jefferson City	6.12
Lime and iron	Jefferson City	6.79
Lime and iron	Jefferson City	7.0
Softening	Kansas City	11.57
Excess lime and alum backwash	Boonville	13.2
Cationic flocculent	St. Joseph	14.1
Lime and iron	St. Louis	21.2
High magnesium softening sludge	Kansas City	25.1
Iron	St. Louis	40.8
Lime and alum	Boonville	53.4
Iron backwash	St. Louis Co.	76.8
Iron	St. Louis Co.	77.6
Cationic flocculent backwash	St. Joseph	80.1
Iron backwash	St. Louis	121.8
Iron	St. Louis Co.	148.5
Alum	Moberly	164.3

Source: Calkins and Novak 1973.
*All locations are in Missouri.
†$S^2/g \times 9.81 \times 10^3$ = m/kg.

A simplification of the SR test is the TTF test. This test is set up with the same Buchner funnel apparatus as the SR test. The only data collected are the times for one half of the volume to filter. This test is much faster to run and analyze the data than the SR test and can provide useful information on the effects of conditioning procedures. Sample size can be very important in obtaining good differentiation of results. Figure 2.5 shows test results on the TTF test for the same Durham sludge as the SR test results discussed previously. It can be seen that the TTF test did predict the same optimal polymer dose as the SR test.

The CST technique is one of the fastest and simplest tests to perform on the dewatering characteristics of sludges. The results are very useful for comparing conditioning methods or for use as an operator tool in determining polymer dose for full-scale dewatering devices.

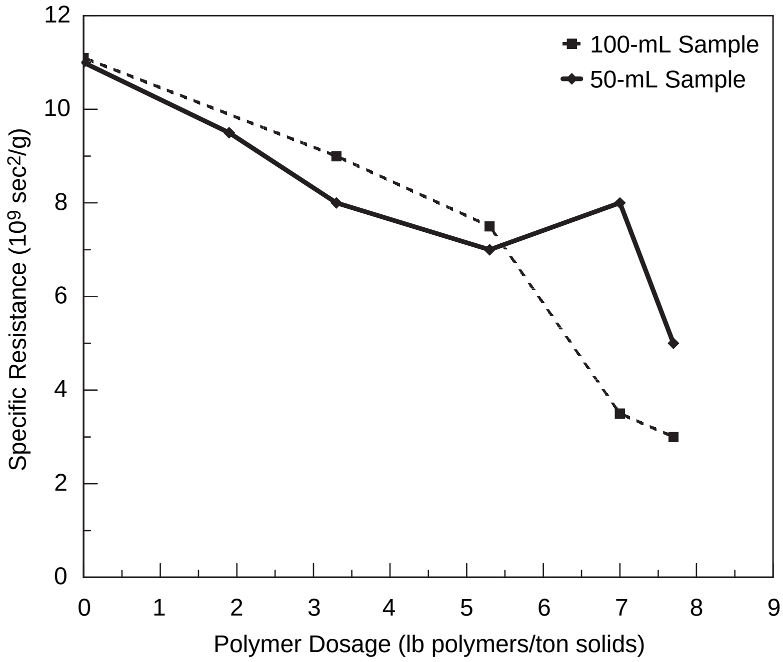


Figure 2.4 Example of specific resistance versus polymer dosage for an alum residual

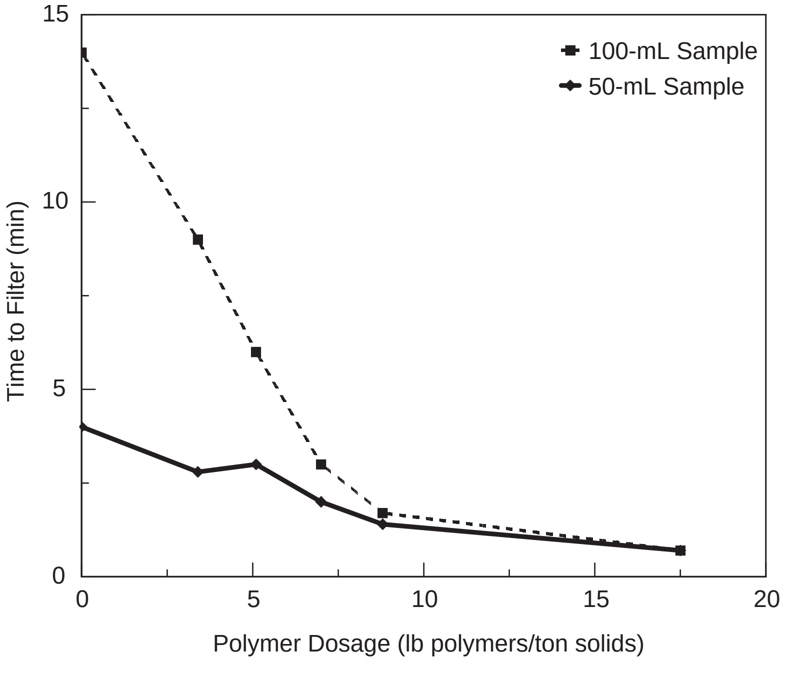


Figure 2.5 Example of TTF versus polymer dosage for an alum residual

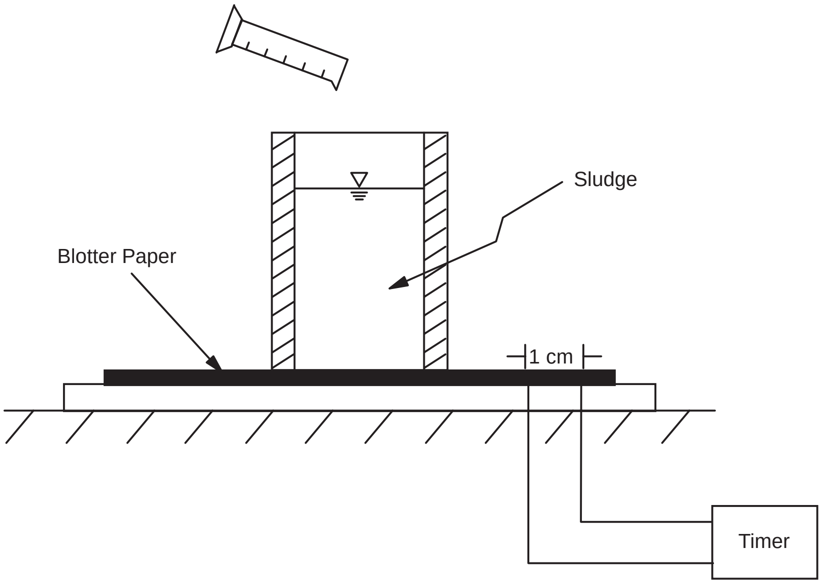


Figure 2.6 CST apparatus

The CST test is run on the apparatus shown in Figure 2.6. A representative residual sample is placed into the cup shown. As the residuals sample is dewatered, the liquid flows outward through a special blotter or chromatography paper. As the liquid flows by the first probe, it activates a timer, and the timer stops when the flow reaches the second set of probes, which are normally 1 cm away. Samples are conditioned by mixing a known concentration of polymer (or other conditioner) to the sludge. The sample is then poured into the tube of the CST apparatus. Figure 2.7 shows the CST results for the Durham sludge.

The two curves in Figure 2.7 also reflect the different tubes available from the manufacturer. For this residuals sample, it appeared that the narrow tube diameter (taller tube) gave better-defined results. Again, the optimal polymer dose was about the same as predicted by the SR and TTF tests.

Vesilind (1988) presented a model for what occurs during the performance of a CST test. He proposed that the rate at which water is released from the sludge material into the chromatography paper is a function of two distinct and separate processes. The first is adsorption associated with the test instrument, and the second is water release associated with the sludge material. The adsorption associated with the test instrument can be quantified as a function of the test apparatus and the chromatography paper. In terms of the test apparatus, the flow of free water from the solids is a function of the bottom diameter area of the stainless-steel reservoir, the permeability of the chromatography paper used, and viscosity. The values and effects of each of these parameters can be evaluated and determined through simple measurements conducted on the test instrument. Viscosity is a function of temperature, so its value must be determined for each test conducted.

This instrument constant accounts for the change in diameter between the first and second sets of electrodes used to measure the CST. It also quantifies the permeability of the chromatography paper and the effects on dewatering associated with the reservoir. This instrument constant is labeled Φ, and is assumed at 0.012 unless measured.

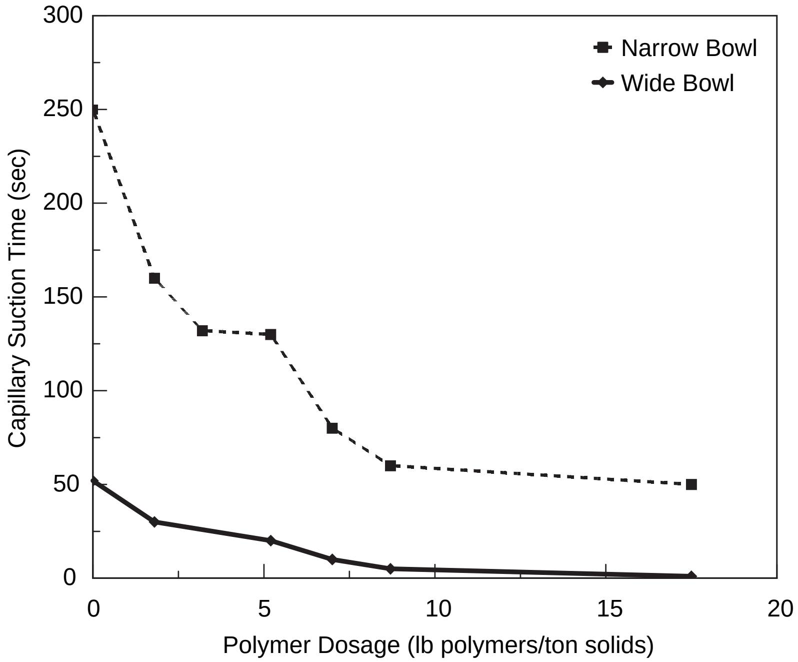


Figure 2.7 Example of CST versus polymer dosage for an alum residual

Vesilind (1988) further proposed that the water released from the sludge material is a function of solids concentration and viscosity. It has long been recognized that solids concentration has an effect on CST. The sludge concentration is directly proportional to the filterability constant. The filterability constant can be determined as follows:

$$X = \Phi\left[\frac{\mu SS}{\mathrm{CST}}\right] \tag{2.14}$$

where X = filterability constant ($\mathrm{kg^2/sec^2{\cdot}m^4}$)
Φ = dimensionless instrument constant
μ = viscosity (centipoise)
SS = suspended solids concentration (mg/L)

Vandermeyden and Cornwell (1998) analyzed drainage characteristics on about 50 different types of WTP residuals. Table 2.6 presents summary statistics for each of the measured drainage parameters. The table presents the data by residual type and gives the number of samples (n), mean value (mean), and sample standard deviation (SD) for each of the drainage parameters.

Interestingly, the general results from all the tests were similar. For example, lime residuals had the lowest CST, TTF, and SR. The tests would predict the ease of dewatering order to be lime, ferric iron, alum, and PACl.

Table 2.6
Summary statistics for drainage parameters

	CST (sec)			TTF (sec)			SR (10^{13} m/kg)		
Sludge type	n	mean	SD	n	mean	SD	n	mean	SD
Alum	38	194.1	195.4	38	319.5	412.6	38	15.8	21.3
Ferric iron	9	103.0	64.5	9	104.7	79.5	9	6.4	8.1
PACl	5	289.8	258.8	5	410.9	562.5	5	13.8	11.0
Lime	9	70.0	34.5	9	34.3	20.4	9	0.5	0.8

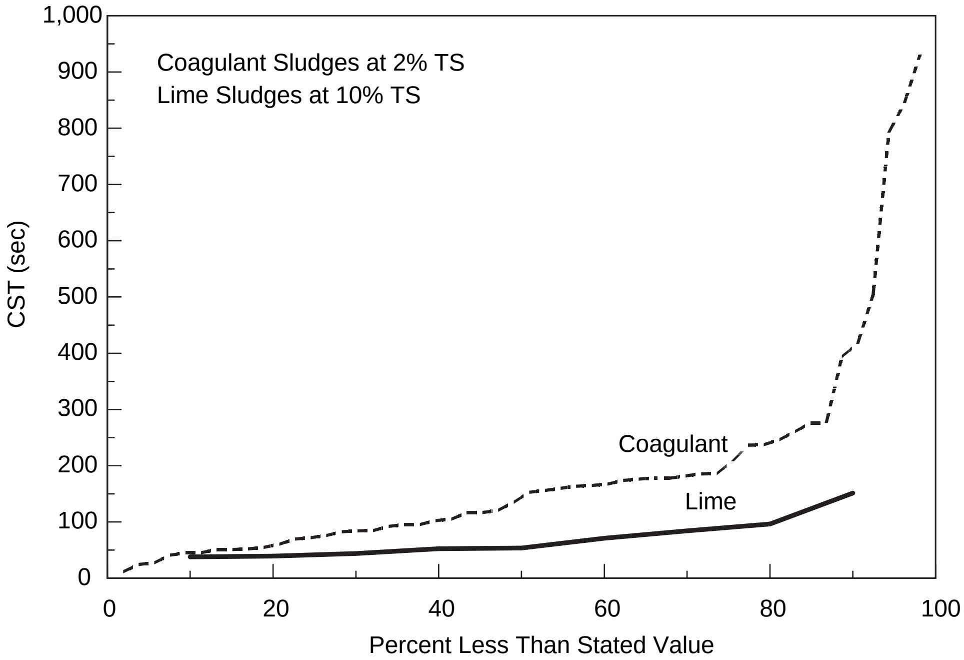
Source: Vandermeyden and Cornwell 1998.

Probability plots for each of the above-mentioned parameters were also developed. This type of plot provides a means of comparison between a particular sample and the entire database. The data were divided into either a coagulant residual (alum, ferric iron, PACl) or a lime residual. Figures 2.8 to 2.10 show the results for CST, TTF, and SR. As an example in using the plots, 90 percent of all the coagulant residuals samples had a CST of less than about 400 sec^{-1}. Therefore, if utility personnel ran a CST test on their residual and got a value of 700 sec^{-1} at 2 percent total solids (TS), they might expect that their residual would be difficult to dewater compared to other utilities. In order to compare results, the same TS need to be utilized.

Vandermeyden and Cornwell (1998) also conducted hydrometer analyses of each of the characterization samples and determined the percentages of sand-, silt-, and clay-sized particles contained in each sample. As a result of analyzing the characterization samples, frequency of occurrence plots of each of these fractions were constructed. Figure 2.11 shows the distribution of average particle diameters for the coagulant and lime residuals. The results are presented as percentages of the total number of samples collected.

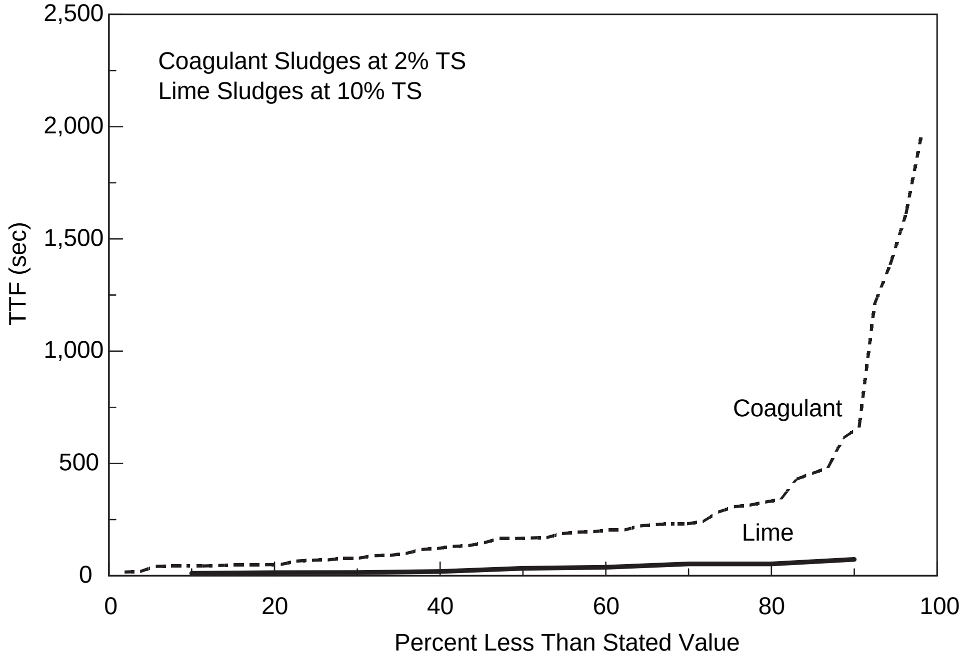
There also appears to be a great variance associated with the sand, silt, and clay data. Standard deviations are quite large for the measurements. Each of the samples collected represent a variety of both raw water characteristics and treatment processes. Because of large variations in both source water quality and treatment process, it seems reasonable that this amount of variation would be encountered.

Volumes and suspended solids concentrations of sludges leaving the sedimentation basins or clarifiers are a function of raw water quality, treatment, and the sludge removal method. Sludges that are allowed to build up in basins being cleaned only periodically by manual procedures tend to compact and thicken in the bottoms of the basins. There is often a stratification of solids with the heavier particles settling to the bottom and the hydroxide or lighter particles at the top. This is one reason why it is very difficult to obtain representative sludge samples from within a basin. There will also be a distribution of particle types longitudinally. However, the actual concentration produced will depend on the amount of water used to flush the solids out of the basin during cleaning. With increasing finished water quality standards, there will be a trend to remove the solids as quickly as possible, generally with continuous collection equipment. In this



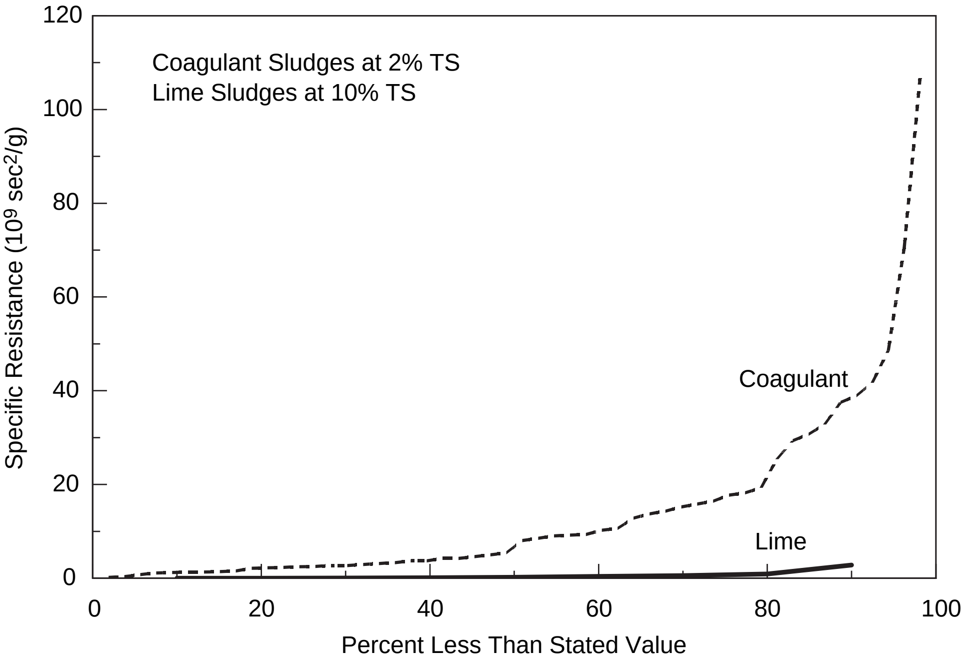
Source: Vandermeyden and Cornwell 1998.

Figure 2.8 CST distribution for coagulant and lime residuals



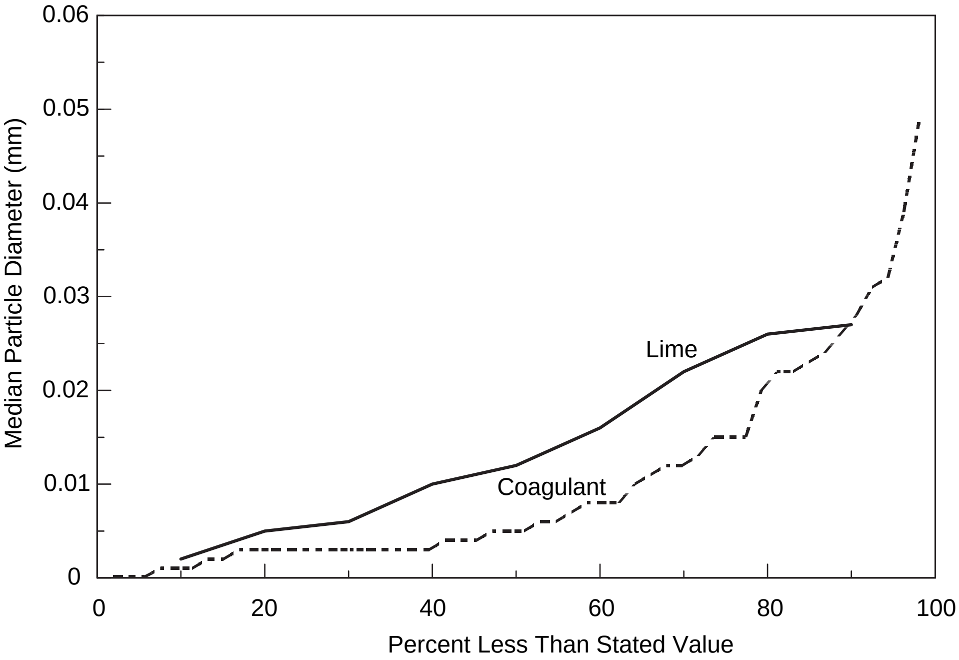
Source: Vandermeyden and Cornwell 1998.

Figure 2.9 TTF distribution for coagulant and lime residuals



Source: Vandermeyden and Cornwell 1998.

Figure 2.10 SR distribution for coagulant and lime residuals



Source: Vandermeyden and Cornwell 1998.

Figure 2.11 Average particle diameter distribution of coagulant and lime solids

case the solids concentrations will be lower because compaction height and time has been less. Solids concentrations for sludges produced with alum or iron coagulants and for low- to moderate-turbidity raw waters will be about 0.1 to 1.0 percent. The higher the coagulant-to-raw–water-solids ratio, the lower the solids concentration and the higher the sludge volume. Coagulant sludges from highly turbid raw waters may be in the 2 to 4 percent range and occasionally higher. Sludge volumes tend to be 0.1 to 3 percent of the raw water flow, with one survey finding an average of 0.6 percent. Softening sludges will be more concentrated, usually as a function of the $CaCO_3/Mg(OH)_2$ ratio and the type of clarifier. Conventional sedimentation basins may only produce solids concentrations of 2 to 4 percent, whereas sludge blanket clarifiers can produce solids concentrations of up to 15 percent. Sludge volumes will correspondingly vary considerably, from 0.5 to 5 percent of the water plant flow.

Characteristics of Predominantly Solid Sludge

Tests for characterization of more solid samples—those above the liquid limit—are generally completed to evaluate compaction and shear. A compaction test finds the optimal density that is necessary for predicting the volume of residuals that require disposal. The shear tests relate to the slope stability and ability to carry the load of earthmoving equipment, and so forth.

Density, Specific Gravity, and Solids Concentration

Many relationships in soils engineering utilize the moisture content of the material in question rather than the solids content. Moisture or water content is expressed in soil engineering as a percentage of the mass of the dry material. In other words, at 100 percent moisture content, the mass of water and the mass of solids are equal and the corresponding solids concentration is 50 percent. It should also be noted that moisture concentrations can thus exceed 100 percent, in which case there is more water present than solids. The exact relationship between moisture content and solids concentration is expressed as follows:

$$\text{solids content, \%} = \left[\frac{1}{1+\dfrac{\text{water content, \%}}{100}}\right]\times 100 \tag{2.15}$$

$$\text{water content, \%} = \left[\frac{100}{\text{solids content, \%}} - 1\right]\times 100 \tag{2.16}$$

These relationships are useful to refer to, particularly if a utility elects to conduct physical characterization tests on its sludge, because geotechnical laboratories typically express all test results in terms of moisture or water content. The term most familiar in the water profession is *solids content*.

Various sludge density measurements include loose wet and dry densities and compacted dry density. The wet unit weight of a cohesive material such as sludge is determined according to a standardized procedure per ASTM D2937-71 (ASTM 1993). In this test, a tubular specimen is weighed and placed in a container of a known volume. The container is filled with water and the volume of the sample is subsequently determined. The sample is then weighed again and oven dried to determine the solids or water content. Wet unit weight can be calculated as the weight of the sample divided by the volume of the sample in water. The dry unit weight is determined with the relationships shown below:

$$\text{dry density} = \frac{\text{wet density}}{1 + \dfrac{\text{water content, \%}}{100}} \tag{2.17}$$

$$\text{dry density} = \text{wet density} \times \left[\frac{\text{solids content, \%}}{100}\right] \tag{2.18}$$

Both relationships are useful in data interpretation and in converting laboratory data to more workable terms for the planning of sludge handling facilities.

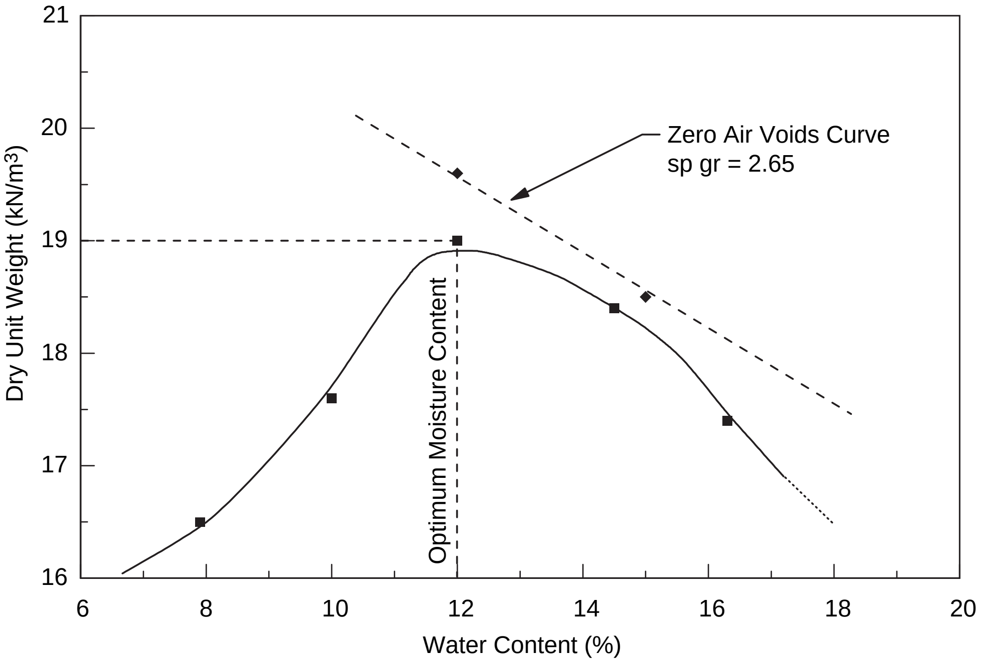
Compaction tests or moisture–density relationship tests (ASTM D698-70 [ASTM 1993]) determine the optimum moisture content and corresponding dry unit weight. This information provides data that are important to establish the degree of compaction of the sludge necessary to increase stability, decrease permeability, and enhance resistance to erosion. In the compaction test (standard Proctor type), the sludge is compacted in a 4-in. (10.2 cm) diameter cylindrical mold. The sludge is placed in the mold in three equal layers, each layer compacted by 25 blows of a standardized metal tamper. The tamper (a 24.5-N compaction hammer) drops 1 ft (0.305 m) onto the sludge sample, delivering a compaction energy of 12,400 ft-lb/ft^3 (593.7 kN/m^3). The amount of energy applied by the tamper was established by Proctor as the amount that would yield the maximum density in the laboratory and would approximately equal a density feasible to achieve with light rollers or very thorough tamping in thin layers. A typical compaction test generates several dry density concentrations versus water content from which the optimum moisture content can be selected to yield the greatest dry unit weight. An example for a soil material is shown in Figure 2.12.

It is important to note that density obtained from the compaction curve is the maximum density achievable at the particular solids concentration. No amount of overcompaction will be sufficient to reach a higher density unless the sludge is dried further. Overcompaction could also rework the sludge and actually cause a reduction in shear strength.

The cone penetration test (ASTM D3441 [ASTM 1993]) consists of a laboratory method to directly measure the shear strength of the sludge at various solids concentrations. In this test, a weighted, conical, pointed object is allowed to drop into a prepared sample while the testing apparatus measures the degree of penetration (or cone resistance). Through standardized tables, the amount of shear strength exhibited by the sample under a confined condition can be determined.

Sludge typically exhibits strong rheotropic (decreasing shear strength due to disturbance) and thixotropic (increasing shear strength with time after disturbance) characteristics. These phenomena can also be noted with the cone penetration test when conducting a series of tests at different sample curing times.

A soil's shear strength or its resistance to sliding along internal surfaces is one of its most important engineering properties. Shear strength enables a soil to maintain equilibrium on a sloping



Source: Cornwell et al. 1992.

Figure 2.12 Typical soil compaction curve

surface such as an embankment or a natural hillside. The shear strength of the soil also influences the bearing capacity. All these properties are directly applicable to sludges and their ability to support heavy equipment in a landfilling operation under different surface slope conditions.

The triaxial compression test is considered the most reliable of a host of different types of shear tests. In the triaxial compression test, a cylinder of sludge is placed in a rubber membrane and mounted in a pressure vessel. In the vessel, a pressurized liquid provides a confining pressure uniformly distributed around the sample. This pressure is kept constant during the test. An axial load is then applied to both ends of the sample until the sample fails because of shear on its internal surfaces.

Calkins and Novak (1973) reported various physical characteristics of sludge including viscosity and shear strength. Shear strength was measured on air-dried sludge samples at different solids concentrations with a Torvane shear tester. In this study, it was noted that the sludges became adequately dewatered for transportation and handling with earth-moving equipment at a shear strength of approximately 60 to 80 lb/ft^2 (2.87 to 3.83 kN/m^2). There was no specific solids concentration noted in this work that correlated to the 60 to 80 lb/ft^2 (2.87 to 3.83 kN/m^2) shear strength. For three alum sludges tested in works by Calkins and Novak (1973), the solids concentration corresponding to 80 lb/ft^2 (3.83 kN/m^2) ranged from approximately 20 percent to 28 percent, and two lime sludges required a 40 to 45 percent solids concentration.

Knocke and Wakeland (1983) conducted further research in the area of sludge handling characteristics by investigating the use of the Atterberg liquid limit test to estimate the sludge handling characteristics. As previously described, the liquid limit test deals with the shear resistance of a material. The test is based on the hypothesis that each blow with the standardized test apparatus equates to a shear resistance of 1 g/m^2. Knocke and Wakeland's research determined that the shear resistance recommended by Calkins and Novak (1973) for handling sludge, 60 to 80 lb/ft^2 or 0.42 to 0.56 psi (2.87 to 3.83 kN/m^2) corresponded to 30 blows of the standard liquid limit apparatus. By evaluating various sludges, it was concluded that alum sludges achieved the proper level of shear resistance at 17 to 20 percent solids concentration. Lime sludges, on the other hand, needed to be dewatered to beyond 60 percent solids concentration.

Cornwell and Koppers (1990) summarized research conducted in Europe to define the handleability and stability of WTP sludge. This work utilized a motorvane test apparatus to measure the undrained shear strength of various sludges. West Germany and the Netherlands have adopted a preliminary standard of 209 lb/ft^2 or 1.45 psi (10 kN/m^2) as a minimum sludge shear strength to define handleability and stability to support heavy equipment. Test data from 14 different utilities indicated a high degree of variability between solids concentration and shear strength. Even at 35 percent solids concentration, only approximately 25 percent of the sludges tested passed the 209-lb/ft^2 or 1.45-psi (10-kN/m^2) standard. Most of the sludges tested were iron sludges from either groundwater or surface water sources.

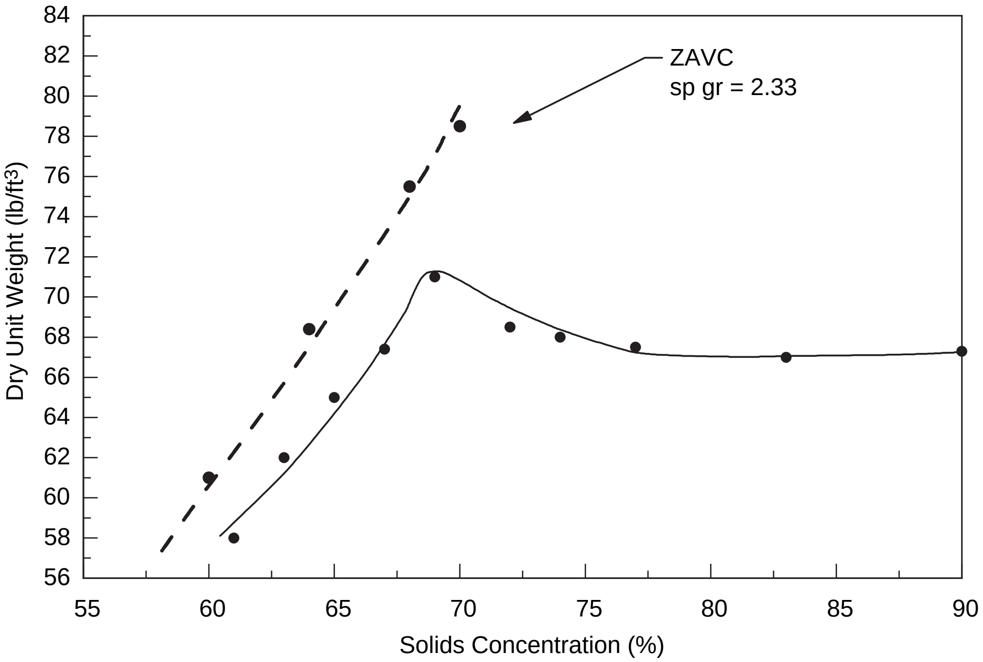
It is worth comparing the European work with the research conducted in the United States by Knocke and Wakeland (1983) and Calkins and Novak (1973). The European adopted standard of 209 lb/ft^2 or 1.45 psi (10 kN/m^2) is approximately three times greater than the shear strength reported as necessary by the American researchers. It should be pointed out, however, that the European value was preliminarily adopted to provide not only a handleable sludge but also a sludge that would be stable enough to support heavy equipment. The American work considered only handleability.

Cornwell et al. (1992) conducted compaction and shear tests on three WTP residuals; two were produced from a conventional plant using alum and one using ferric iron as the coagulant. For each sludge, the zero air voids curve (ZAVC) represents what the dry density of the sludge would be if its entire volume consisted of water and solids only. This curve is determined theoretically by Equation 2.19. No matter how much the sludge is compacted, it is impossible to fill all the air voids with water. As a result, the compaction curve for a particular sludge cannot cross its ZAVC.

$$\text{dry unit weight} = \frac{\text{specific gravity} \times \text{unit weight water}}{1 + \text{specific gravity} \times \left(\dfrac{\text{water content, \%}}{100}\right)} \tag{2.19}$$

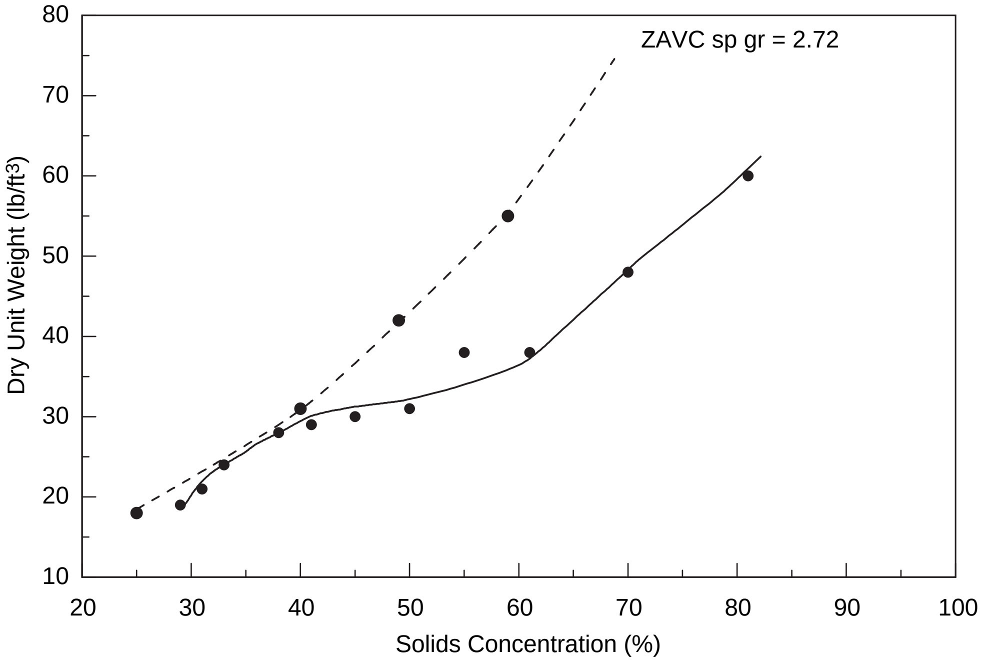
The compaction curve for ferric sludge (Figure 2.13) exhibits the general bell shape typically seen in a compaction curve for natural soils. For this ferric sludge, the maximum dry density occurred at 72 lb/ft^3 (1,224 kg/m^3) at a solids concentration of 69 percent. The compaction curves for alum sludges 1 and 2 (Figures 2.14 and 2.15) are notably different from the ferric sludge in that there is no peak formation. For these two sludges, the maximum dry density occurs at the highest possible solids concentration. Allowing the sludges to air dry at room temperature for 30 days yielded solids concentrations of 80 and 81 percent for alum sludge 1 and 2, respectively. The corresponding dry densities were 61 and 57 lb/ft^3 (1,037 and 969 kg/m^3), respectively.

Hsieh and Raghu (1997) conducted compaction tests on several different WTP sludges. They also found that for some residuals the density continually increased with increasing solids



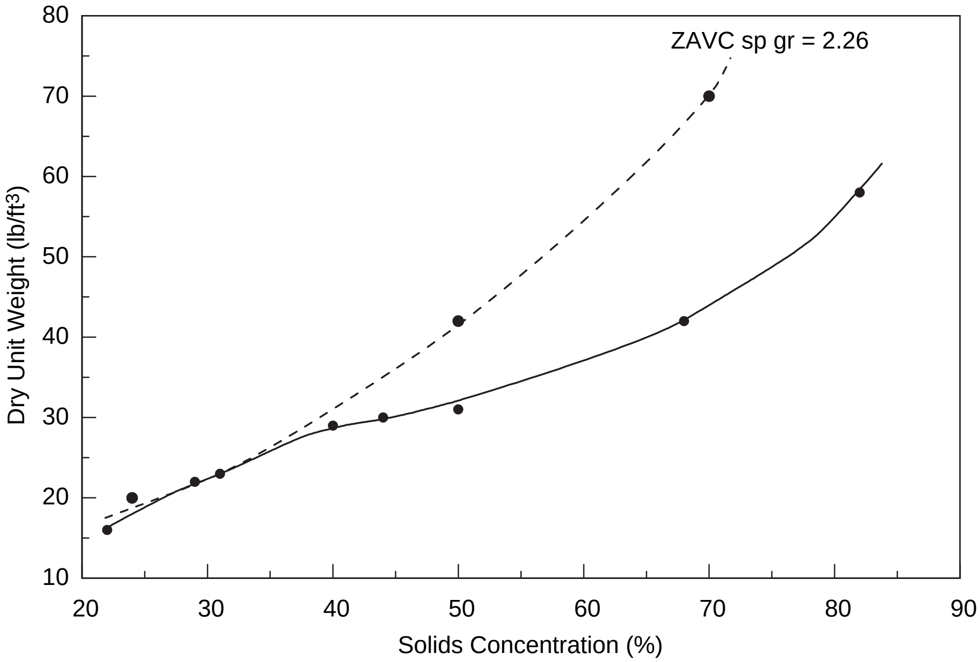
Source: Cornwell et al. 1992.

Figure 2.13 Compaction curve for ferric sludge



Source: Cornwell et al. 1992.

Figure 2.14 Compaction curve for alum sludge no. 1



Source: Cornwell et al. 1992.

Figure 2.15 Compaction curve for alum sludge no. 2

concentration while others exhibited a more traditional bell-shaped or one-hump curve. They found that maximum dry densities were in the range of 34 to 84 lb/ft^3 (580 to 1,420 kg/m^3) at optimal solids concentrations ranging from 50 to 80 percent. Although these are in the same range of the work by Cornwell et al. (1992), the wide variation in results is evident. This is a site-specific parameter that should be individually tested to determine volume requirements for various handling alternatives.

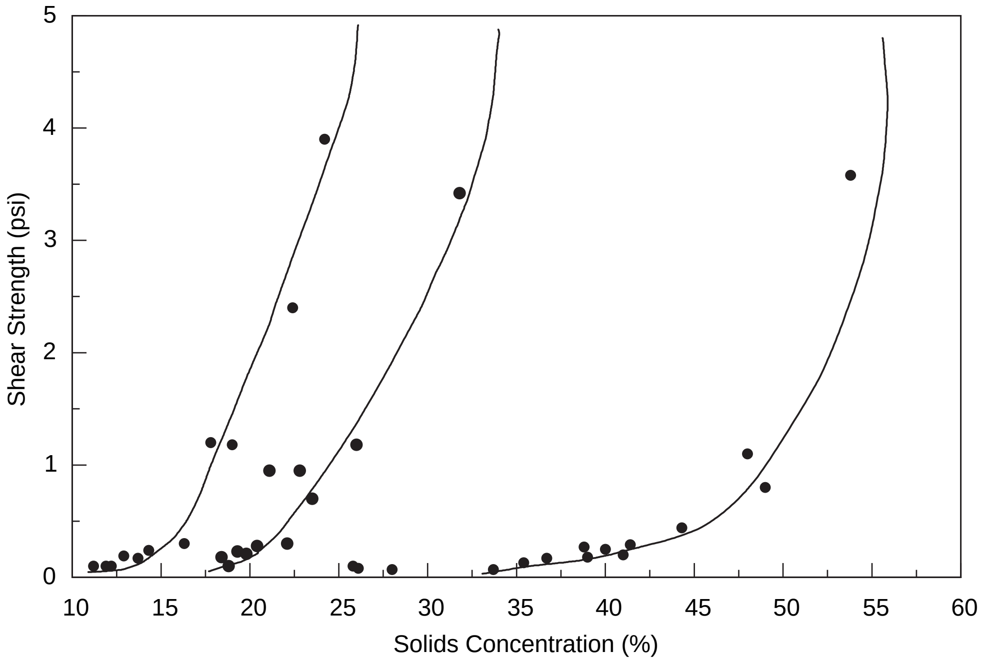
Cornwell et al. (1992) used the cone penetration test and the triaxial compression test to evaluate the shear strength for three different WTP sludges. They also evaluated sludge age and the addition of conditioning agents (lime, fly ash, soil) on the shear properties. The cone penetration test resulted in shear strength values in the low range and the triaxial compression tests in the higher range (Figures 2.16 and 2.17). The two results were combined to give a general trend, as shown in Figure 2.18.

Hsieh and Raghu (1997) showed shear test results for 11 different residuals. Solids concentrations required to reach 10 psi (70 kN/m^2) ranged from about 20 percent up to 75 percent—the same range of variation found by Cornwell et al. (1992). No general trend by sludge type was found, although the sludges were of various ages and produced by different dewatering methods. Generally the lime sludges required the highest percent solids for a given shear value.

All of these properties are useful for designing disposal options as addressed in chapter 5.

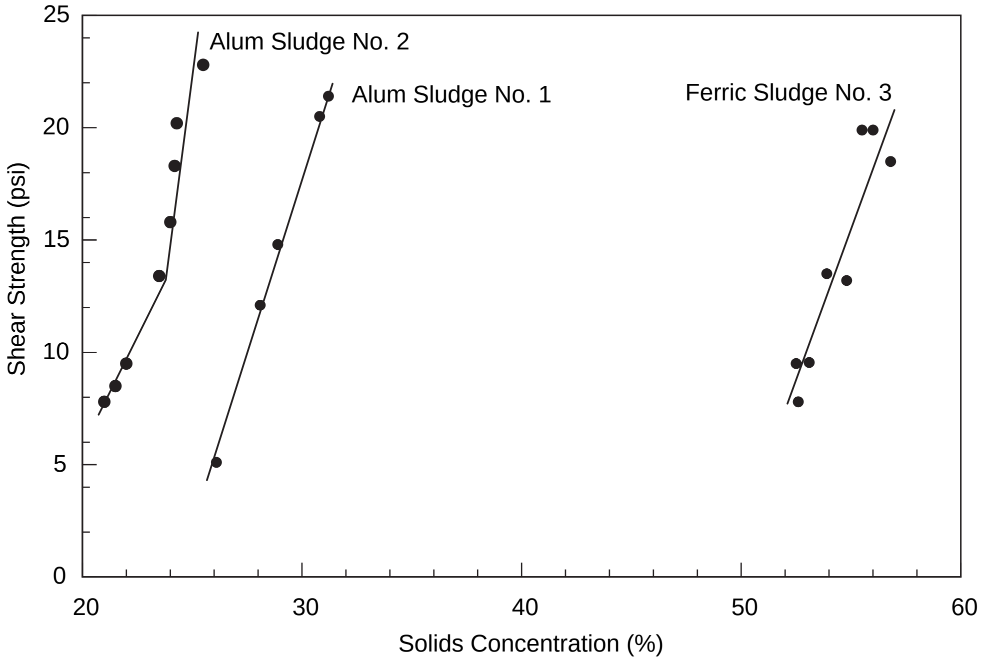
Chemical Characteristics

Chemical characterization of these residuals has previously only been of interest to researchers in the field or to those utilities considering chemical recovery. However, disposal



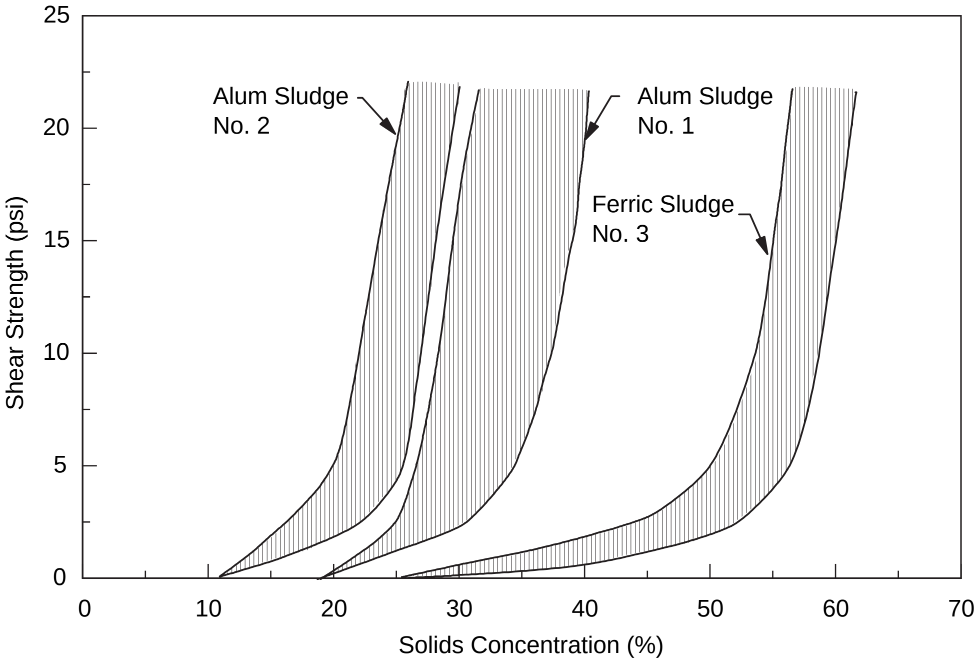
Source: Cornwell et al. 1992.

Figure 2.16 Shear strength versus solids concentration, cone penetration method



Source: Cornwell et al. 1992.

Figure 2.17 Shear strength versus solids concentration, triaxial compression method



Source: Cornwell et al. 1992.

Figure 2.18 Shear strength versus solids concentration, cone penetration, and triaxial compression tests

concerns and beneficial use opportunities have increased the awareness of constituents of the chemical sludges. As drinking water quality standards increase, so will the chemical levels in the waste streams. Analysis of wastes may need to be conducted based on the total concentration of chemical present or based on extraction procedures such as the toxicity characteristic leaching procedure (TCLP) or California Waste Extraction Test (CALWET). For liquid wastes, units of milligrams per liter are appropriate, and for solid wastes (dewatered sludges), units of milligrams per kilogram dry weight sludge would be appropriate. When trying to report results for comparative purposes, it is useful to present all sludge data (dilute or concentrated) in terms of milligrams per kilogram dry weight.

Given and Spink (1984) conducted a literature review and summarized their findings on alum sludge characteristics, which is shown in Table 2.7. Some of the very first detailed analysis of alum sludge was reported by Schmitt and Hall (1975) on alum sludge obtained from the Oak Ridge, Tennessee, Water Treatment Plant, as shown in Table 2.8. This sludge is characterized by a high percentage of nonhydrated metal species. Aluminum is 17,000 mg/kg, or as the precipitated species, about 8 percent of the solids are aluminum hydroxide. It is interesting to note that the rare earth elements were detected in the solids. In general, the rare earth elements of even atomic number were found in greater abundance than those of odd atomic number. This is in agreement with the Haskins rule regarding the relative order of abundance of rare earth elements in the lithosphere. The analysis also shows, as is typical of many sludges, the presence of lead, zinc, and manganese. This particular waste also showed 2.1 mg/kg of U-238.

Cornwell et al. (1992) reported total metal concentrations in graphical and tabular format for six different sludges, five alum and one iron. The Connecticut section of the American Water

Table 2.7
Reported alum residuals characteristics

Parameter	Concentration
TS	0.1% to 27% by weight
Volatile solids	10% to 35% of total solids
Suspended solids	75% to 99% of total solids
pH	5.5 to 7.5
Biochemical oxygen demand (BOD)	30 to 6,000 mg/L
Chemical oxygen demand (COD)	500 to 27,000 mg/L
Aluminum	4% to 11% of TS as Al (limited data)
Iron	6.5% of TS (one sample)
Manganese	<0.005% to 5% of TS
Arsenic	<0.04% of TS
Cadmium	<0.005% of TS
Individual heavy metals	<0.03% of TS
Total Kjeldahl nitrogen	0.7 to 1,200 mg/L as N
Phosphate	0.3 to 300 mg/L as P
Total plate count	30 to >300,000 per mL

Source: Adapted from Given and Spink 1984.

Works Association (AWWA) reported results from nine of its water utilities. MacPhee, Cornwell, and Brown (2002) showed results of total metal analysis from residuals in Philadelphia, Pa., and Denver, Colo. A summary of these results is presented in Table 2.9. The Cornwell et al. (1992) work went on to evaluate the amount of the total metal that leached under rainfall conditions. The Connecticut section AWWA work compared the levels to typical solids in different regulatory levels, while MacPhee, Cornwell, and Brown (2002) looked at this source of the metal in the residuals, be it raw water or from the coagulant addition.

Other sections of this book deal with specific constituents present in lime residuals such as arsenic or radionuclides. Some general analyses lime sludges were reported by Shannon et al. (1997). The concentrations of acid-soluble species were determined by dissolving the residuals samples in hydrochloric acid (HCl) solution and analyzing the dissolved liquid by inductively-coupled plasma (ICP) spectroscopy. The results of these analyses are shown in Table 2.10.

Hazardous Properties

In order to classify a water plant waste as “hazardous”, the term must be defined. From an academic standpoint, Davis and Cornwell (1998) have defined a hazardous waste as “any waste, or combination of wastes, that poses a substantial danger, now or in the future, to human, plant, or animal life and which therefore cannot be handled or disposed of without special precautions.” The US Environmental Protection Agency (USEPA) has developed a more usable definition by

Table 2.8
Elemental analysis of alum residuals from Oak Ridge, Tenn.

Atomic No.	Element	SFBW (mg/kg)	Sedimentation basin solids (mg/kg)	Blank
4	Be	0.29	<0.03	<0.02
5	B	1,020	27	1
9	F	20	19	1
11	Na	39	10	4
12	Mg	833	1,183	10
13	Al	Matrix	13,500	6
14	Si	12,500	17,833	40
15	P	200	183	10
16	S	340	167	10
17	Cl	567	267	30
19	K	Matrix	1,650	50
20	Ca	Matrix	3,333	300
21	Sc	2	5	0.3
22	Ti	8,500	6,833	7
23	V	273	30	0.4
24	Cr	7,260	200	4
25	Mn	1,800	983	1
26	Fe	42,733	ND*	40
27	Co	138	3	0.4
28	Ni	103	140	2
29	Cu	120	7	6
30	Zn	467	167	5
31	Ga	3	8	<0.3
32	Ge	0.9	2	<0.5
33	As	15	13	3
34	Se	1	1	<0.4
35	Br	6	13	<0.4
37	Rb	6	20	<0.3
38	Sr	200	93	0.4
39	Y	7	3	<0.2
40	Zr	80	40	<0.4
41	Nb	2	5	<0.2
42	Mo	4	<1	<0.9
44	Rw	<0.7	<1	<0.7
45	Rh	0.5	<3	<0.2
46	Pd	3	2	<0.9
47	Ag	ND	ND	ND

(continued)

Table 2.8 (Continued)

Atomic No.	Element	SFBW (mg/kg)	Sedimentation basin solids (mg/kg)	Blank
48	Cd	1	<1	<0.9
49	In	0.1	<0.4	<0.3
50	Sn	1	<1	<0.8
51	Sb	0.3	2	<0.5
52	Te	<0.9	<1	<0.9
53	I	1.9	6	<0.3
55	Cs	0.9	1	<0.3
56	Ba	367	333	<0.4
57	La	90	8	<0.3
58	Ce	97	28	<0.4
59	Pr	5	6	<0.3
60	Nd	113	18	<1
62	Sm	9	15	<1
63	Eu	4	2	<0.7
64	Gd	10	7	<2
65	Tb	0.8	1	<0.4
66	Dy	3	3	<1
67	Ho	0.6	1	<0.4
68	Er	1	<2	<1
69	Tm	0.6	<0.6	<0.4
70	Yb	2	<2	<1
71	Lu	0.6	<0.6	<0.4
72	Hf	3	<2	<2
73	Ta	9	17	20
74	W	5	<2	4
75	Re	<0.7	<0.9	<0.7
76	Os	<1	<1	<1
77	Ir	<0.7	<0.9	<0.7
78	Pt	2	<2	<1
79	Au	1	2	1
80	Hg	<2	<2	<2
81	Tl	2	<0.9	<0.7
82	Pb	50	47	1
83	Bi	0.6	<0.8	<0.5
90	Th	4	3	0.8
92	U	8	7	2

Source: Adapted from Schmitt and Hall 1975.
*ND = no data.

Table 2.9
Summary of total metal concentration in coagulant residuals (in mg/kg)

	Sludge*	SFBW*	Sludge*	Sludge†			Sludge‡	
	PWD-Fe	PWD-Fe	Denver Water-Al	Alum sludge no. 1	Alum sludge no. 2	Ferric sludge no. 3	Low	High
Aluminum	11,000	3,625	181,000	107,000	123,000	28,600	131,000	150,000
Antimony	<45	<3	<46	NA§	NA	NA	ND**	11.8
Arsenic	<45	6	<45	25.0	32.0	9.2	ND	16
Barium	320	142	240	30	<30	230	19	323
Beryllium	<0.01	<32	NA	NA	NA	NA	ND	49.2
Bromide	0.151	605	NA	NA	NA	NA	NA	NA
Cadmium	4.2	3	<4	1	1	2	ND	6.72
Calcium	51,000	122,330	10,000	NA	NA	NA	1,005	5,740
Chromium	160	<3	13	120	130	50	1.8	592
Cobalt	34	10	7	NA	NA	NA	ND	7
Copper	43	32	<5	168	16	52	2.9	412
Iron	311,000	72,168	14,000	48,500	15,200	79,500	119	20,300
Lead	<42	32	120	11	9	40	ND	115
Magnesium	8,300	25,350	3,300	NA	NA	NA	241	1,870
Manganese	3,300	3,790	6,300	1,180	233	4,800	403	6,520
Mercury	<47	NA	<47	0.1	<0.1	0.2	ND	0.33
Molybdenum	44	10	<14	NA	NA	NA	ND	7.9
Nickel	37	13	33	24	23	131	ND	40.1
Phosphorus	1,300	2,515	2,300	NA	NA	NA	NA	NA
Potassium	8,200	8,058	3,700	NA	NA	NA	363	702
Selenium	<42	3	<1,000	<2	<2	<2	ND	NA
Silicon	61,000	3,262	59,000	NA	NA	NA	NA	NA
Silver	<45	<3	<45	<2	<2	<2	ND	3.04
Sodium	1,000	57,929	2,600	NA	NA	NA	ND	2,800
Strontium	160	NA	160	NA	NA	NA	20.2	39.6
Sulfur	3,600	NA	1,100	NA	NA	NA	NA	NA
Thallium	<0.001	<3	NA	NA	NA	NA	ND	16.5
Tin	51	NA	<46	NA	NA	NA	NA	NA
Titanium	12,000	2,010	700	NA	NA	NA	160	456
Vanadium	1,600	362	25	NA	NA	NA	22.8	70.5
Yttrium	<9	NA	34	NA	NA	NA	NA	NA
Zinc	580	2,942	220	92	393	781	3.5	2,300
Zirconium	4,400	NA	16	NA	NA	NA	NA	NA

*Data from MacPhee, Cornwell, and Brown 2002.

†Data from Connecticut section AWWA.

‡Data from Vandermeyden and Cornwell 1998.

§NA = not analyzed.

**ND = not detected.

Table 2.10
Results of ICP analyses for acid-soluble species in lime residuals samples

Species	Water District No. 1 of Johnson County (mg/kg)	City of Lawrence (mg/kg)	City of Topeka (mg/kg)
Aluminum	3,210	4,950	1,030
Antimony	ND*	ND	ND
Arsenic	7.62	4.35	5.41
Barium	200	261	231
Beryllium	0.0622	0.181	0.0669
Boron	41.2	5.66	16.7
Cadmium	0.164	0.0847	0.0639
Calcium	334,000	327,000	364,000
Chromium	1.62	6.82	2.33
Cobalt	1.16	1.16	0.570
Copper	2.88	5.97	2.19
Iron	708	2,140	1,390
Lead	1.27	1.88	0.261
Magnesium	20,800	12,700	8,550
Manganese	463	1,550	88.2
Molybdenum	0.246	0.164	0.464
Nickel	4.73	6.84	3.87
Potassium	259†	324†	336†
Selenium	0.291	0.737	ND
Silicon	5,400	2,790	2,630
Silver	ND	ND	ND
Sodium	309	108	166
Strontium	932	588	763
Thallium	ND	ND	ND
Vanadium	6.95	6.26	4.75
Zinc	16.5*	13.5*	10.3*

Source: Shannon et al. 1997.
*ND = not detected.
†Analyte detected in bank.

stating the ways in which a waste can be classified as hazardous: (1) by its presence on the USEPA developed lists, or (2) by evidence that the waste exhibits ignitable, corrosive, reactive, or toxic characteristics. The regulations governing these definitions and the subsequent handling requirements are known as the Resource Conservation and Recovery Act of 1976, or RCRA. The RCRA concerns the handling of wastes at currently operating facilities (such as water plants) and at facilities yet to be constructed. It was designed to meet disposal needs resulting from the Clean Water Act (CWA) and the Clean Air Act (CAA). Those statutes require the removal of hazardous substances from air emissions and water discharges. Neither of these other statutes (CWA and CAA), however, ensured that the disposal of the waste materials generated would be environmentally sound. The RCRA was intended to provide that assurance. The five major elements of the RCRA are

1. Federal classification of hazardous wastes
2. Cradle-to-grave manifest system
3. Federal standards to be followed by generators, treaters, disposers, and storers of hazardous wastes
4. Enforcement of federal standards
5. Authorization of states to obtain primacy for implementation of the regulations

So the major question is, are water plant wastes hazardous according to the USEPA definitions? Water plant wastes are not on the developed list of specifically identified hazardous wastes, so that part of the definition does not apply. That leaves the properties of ignitability, reactivity, corrosivity, or toxicity as a means of defining the waste material as hazardous. It is highly unlikely that water plant wastes will fall under either of the first two properties. A waste is classified as corrosive it if has a pH ≤2 or ≥12.5. It is possible that coagulant recovery side streams, perhaps filtrate from lime conditioning of sludge in a filter press, and brines from acid regeneration of IX resins would fall outside these limits. The pH can be adjusted with appropriate neutralization.

Extraction tests are designated by USEPA in the *Federal Register* as the method to be used in identifying the characteristic of toxicity in a solid waste. The presence in the extract from a representative waste sample of any number of contaminants at or above a specified regulatory level constitutes failure of the test and furthermore makes the waste subject to regulation as a hazardous waste per Subtitle C of the RCRA. The TCLP is used as the indicator of toxicity in a waste.

The TCLP is designed to determine the mobility of both organic and inorganic analytes present in liquid, solid, and multiphasic wastes (40 CFR Part 261 [USEPA 1983]). A representative waste sample having a solids content of at least 0.5 percent is first separated into solid and liquid (if any) phases by subjecting it to pressure filtration. If a waste sample contains less than 0.5 percent solids, the waste itself is used as the extract in the procedure, after being filtered through a glass fiber filter.

Extraction of the solid phase, which involves the addition to the solid of an amount of extraction fluid equal to 20 times its weight followed by their combination in an extraction vessel, is preceded by a reduction in solid particle size (if necessary to conform to standard requirements). When obtaining an extract to be used for analysis of nonvolatiles (metals, etc.) only, a minimum waste amount of 100 g is specified, primarily to ensure that an adequate volume of extract is obtained for analysis. However, when an extract is to be used to evaluate the mobility of volatiles, the maximum amount of solid that can be accommodated by the zero-headspace extraction (ZHE) vessel is 25 g because of the volume of the vessel and the amount of extraction fluid required. After an 18-hour extraction period, the liquid extract is separated from the solid material through filtration using a glass fiber filter. Assuming compatibility (i.e., combination will not result in formation of multiple phases), the liquid extract is added to the liquid (if any) from the original separation for analysis; otherwise, liquid fractions are analyzed separately and volume-weighted average concentration is determined (40 CFR Part 261 [USEPA 1983]).

Some notable details regarding the extraction procedure include the specification that a ZHE vessel be employed when testing for the mobility of volatiles (zero headspace in the vessel precludes the escape of volatiles into the air) and the selection of the type of extraction fluid based on waste alkalinity in analysis of nonvolatile compounds. Two different extraction fluids, one having a pH of approximately 5 and the other with a pH of about 3, are used in the TCLP test. In TCLP extractions to be used to analyze for volatile constituents, and in extractions for nonvolatile analytes in which the pH of the sample after being mixed with water and stirred vigorously is less than 5, extraction fluid No. 1 (pH 4.93 ± 0.05) is specified. If the pH of the waste sample after being stirred is greater than 5, HCl is added to the mixture, which is subsequently heated to 50°C, held at that temperature for 10 minutes, and allowed to cool. Only if the pH is less than 5 after completion of this test should fluid No.1 be used for the extraction; otherwise extraction fluid No. 2 (pH 2.88 ± 0.05) should be used.

The results of TCLP tests reported by Cornwell et al. (1992) for five different sludges are shown in Table 2.11, along with the threshold level for each constituent regulated. None of the regulatory limits were exceeded. Although detection limits for each constituent varied from test to test, results were similar; the vast majority of regulated constituents were not detectable in either of the sludge samples. Constituents that were detected were present in amounts well below regulatory levels.

The researchers went on to compare the maximum amount of each regulated metal present in each of the three sludges, determined through a total metals analysis (Cornwell et al. 1992). This was compared to the TCLP regulatory threshold level. As shown in Table 2.12, no constituent is out of compliance with the limit. It should be noted that the data in the second, third, and fourth columns of the table indicate the total amount of metals that would be in the extract if 100 percent of the metal was extracted. The formula used to estimate the maximum amount of a particular constituent that could be extracted from the sludge is presented in Equation 2.20, along with a sample calculation for arsenic (Equation 2.21).

$$\frac{\begin{pmatrix}\text{sludge total metal}\\ \text{concentration}\\ \text{(mg/kb)}\end{pmatrix}\begin{pmatrix}\text{TCLP sample}\\ \text{size}\\ \text{(g)}\end{pmatrix}\begin{pmatrix}\text{solids content}\\ \text{of sludge}\\ \text{(\%)}\end{pmatrix}}{\text{extraction fluid volume (L)}} = \text{maximum leachable quantity (mg/L)} \tag{2.20}$$

$$\frac{\text{arsenic}}{\text{alum sludge no. 1}} : \frac{(25\ \text{mg/kg})(1\ \text{kg}/1{,}000\ \text{g})(25\ \text{g})(0.57)}{0.5\ \text{L}} = 0.72\ \text{mg/L} \tag{2.21}$$

Performing such a calculation, along with a total metals analysis, could eliminate the need for further, more expensive analysis according to the federal TCLP.

A direct comparison of potential and actual leaching serves to indicate metals that could not possibly exceed TCLP allowable limits. The significance of this comparison from the standpoint of economics is derived from the following statement: “If a total analysis of the waste

Table 2.11
TCLP results for five WTP coagulant sludges

Contaminant	Regulatory threshold level (mg/L)	Contra Costa Water District, Calif., alum sludge (mg/L)	Phoenix, Ariz., alum sludge (mg/L)	Alum sludge no. 1	Alum sludge no. 2	Alum sludge no. 3
Arsenic	5.0	0.04	<0.3	0.0088	0.0023	0.0006
Barium	100.0	1.1	1.1	1	<1	<2
Benzene	0.5	<0.001	<0.003	<0.005	<0.005	<0.005
Cadmium	1.0	<0.05	<0.02	<0.02	<0.02	0.02
Carbon tetrachloride	0.5	<0.001	<0.001	<0.005	<0.005	<0.005
Chlordane	0.03	NA*	<0.0003	<0.005	<0.005	<0.005
Chlorobenzene	100	<0.001	<0.001	<0.005	<0.005	<0.005
Chloroform	6.0	<0.001	0.004	<0.005	<0.005	<0.005
Chromium	5.0	0.06	<0.04	<0.1	<0.1	<0.1
Cresol	200	NA	NA	<0.005	<0.005	<0.005
m-Cresol	200	NA	NA	<0.500	<0.500	<0.500
o-Cresol	200	<0.05	<0.01	<0.500	<0.500	<0.500
p-Cresol	200	<0.1	<0.01	<0.500	<0.500	<0.500
2,4-D	10	NA	<3	NA	NA	NA
1,4-Dichlorobenzene	7.5	<0.02	<0.001	<0.005	<0.005	<0.005
1,2-Dichloroethane	0.5	<0.001	<0.001	<0.005	<0.005	<0.005
1,1-Dichloroethylene	0.7	<0.001	<0.001	<0.005	<0.005	<0.005
2,4-Dinitrotoluene	0.13	<0.2	<0.01	<0.01	<0.01	<0.01
Endrin	0.02	NA	<0.00007	<0.005	<0.005	<0.005
Heptachlor (and its expoxide)	0.008	NA	<0.00004	<0.005	<0.005	<0.005
Hexachlorobenzene	0.13	NA	<0.01	<0.005	<0.005	<0.005
Hexachlorobutadiene	0.5	<0.05	<0.01	<0.01	<0.01	<0.01
Hexachloroethane	3.0	<0.1	<0.01	<0.005	<0.005	<0.005
Lead	5.0	<0.2	<0.5	<0.1	<0.1	<0.1
Lindane	0.4	NA	<0.00004	<0.005	<0.005	<0.005
Mercury	0.2	<0.001	<0.01	<0.002	<0.002	<0.002
Methoxychlor	10	NA	<0.0002	<0.005	<0.005	<0.005
Methyl ethyl ketone	200	0.37	<0.01	<0.100	<0.100	<0.100
Nitrobenzene	2.0	<0.02	<0.01	<0.500	<0.500	<0.500
Pentachlorophenol	100	<0.2	<0.01	<0.01	<0.01	<0.01
Pyridine	5.0	NA	<0.05	<0.500	<0.500	<0.500
Selenium	1.0	<0.02	<1	<1	<0.002	<0.002
Silver	5.0	<0.05	<0.01	<0.1	<0.1	<0.1
Tetrachloroethylene (PCE)	0.7	<0.001	<0.001	<0.005	0.005	<0.005
Toxaphene	0.5	NA	<0.0005	<0.005	<0.005	<0.005
2,4,5-TP	1.0	NA	<0.5	NA	NA	NA
Trichloroethylene (TCE)	0.5	<0.001	<0.001	<0.01	<0.01	<0.01
2,4,5-Trichlorophenol	400	<0.1	<0.01	<0.01	<0.01	<0.01
2,4,6-Trichlorophenol	2.0	<0.1	<0.01	<0.01	<0.01	<0.01
Vinyl chloride	0.2	<0.001	<0.001	<0.005	<0.005	<0.005

Source: Cornwell et al. 1992.
*NA = not analyzed.

Table 2.12
Total possible TCLP versus actual TCLP for three sludges

	Maximum possible concentration in TCLP based on total metals analysis (mg/L)				Actual TCLP analysis results (mg/L)		
Metal	Alum sludge no. 1	Alum sludge no. 2	Ferric sludge no. 3	TCLP regulatory limit (mg/L)	Alum sludge no. 1	Alum sludge no. 2	Ferric sludge no. 3
Arsenic	0.72	0.35	0.27	5.0	0.008	0.0023	0.0006
Barium	0.86	0.33	6.60	100.0	1.0	<1.0	2.0
Cadmium	0.03	0.011	0.06	1.0	<0.02	<0.02	0.02
Chromium	3.43	1.43	1.44	5.0	<0.1	<0.1	<0.1
Lead	0.32	0.10	1.15	5.0	<0.1	<0.1	<0.1
Mercury	0.0029	0.0006	0.0057	0.2	<0.002	<0.002	<0.002
Selenium	<0.1	<0.1	<0.1	1.0	<0.002	<0.002	<0.002
Silver	<0.1	<0.1	<0.1	5.0	<0.1	<0.1	<0.1

Source: Cornwell et al. 1992.

demonstrates that individual analytes are not present, or that they are present at such low concentrations that appropriate regulatory levels could not possibly be exceeded, the TCLP need not be run" (40 CFR Part 261 [USEPA 1983]. It appears that a relatively inexpensive standard metal analysis would be sufficient for a typical WTP coagulant sludge to prove that regulatory TCLP metals limits could not be exceeded. Table 2.13 indicates the total metal concentration (for each regulated metal) that would have to be present in sludges with solids contents of 57 and 22 percent to reach threshold levels (assuming 100 percent of the total is extracted), along with actual concentrations determined analytically through total metals analyses performed on alum sludges No. 1 and No. 2. In the examples shown, the regulatory limits could not be reached.

In these examples, actually running the TCLP test would be unnecessary because the sludge could not have failed even if 100 percent of the metal was extracted.

The European Waste Catalogue

It is generally acknowledged that the quality of data in the waste sector is poor throughout the European Union (EU) and that uniform methodologies for compiling statistics on waste management need to be developed and applied. In an effort to standardize the management of information in the waste sector, Council Directive 91/156/EEC, among other things, required the Commission to draw up a list of wastes. This list, commonly referred to as the European Waste Catalogue (EWC), was published as Commission Decision 94/3/EEC. It is a harmonized, nonexhaustive list of wastes that will be periodically reviewed and, if necessary, revised. It is intended that the EWC will provide a common terminology throughout the EU with the purpose of improving the collection and management of statistics on waste and, by so doing, improve the

Table 2.13
Total sludge metal concentrations required to fail TCLP test compared to actual data

	Minimum concentration to exceed TCLP limit (mg/kg)		Total concentrations determined through total metals analyses (mg/kg)	
Constituent	22% solids	57% solids	Alum sludge no. 2 (22% solids)	Alum sludge no. 1 (57% solids)
Arsenic	455	175	32	25
Barium	9,090	3,510	<30	30
Cadmium	90	35	1	1
Chromium	455	175	130	120
Lead	455	175	9	11
Mercury	18	7	<0.1	0.1
Selenium	90	35	<2	<2
Silver	455	175	<2	<2

Source: Cornwell et al. 1992.

efficiency of waste management activities. The EWC will constitute the basic reference for the EU program on waste statistics.

After consultations with member states, the EWC 1994 was subseqently updated and combined to form a revised European Waste Catalogue (EWC 2002, Commission Decision 200/532/EC). The United Kingdom, like other member states, is fulfilling a requirement to integrate this catalog into its own domestic legislation.

REFERENCES

ASTM (American Society for Testing and Materials). 1993. *ASTM Book of Standards*. Vol. 04.08. West Conshohocken, Pa.: ASTM.

Calkins, R., and J. Novak. 1973. Characterization of Chemical Sludges. *Jour. AWWA*, 65(6):423.

Cornwell, D. 1978. Alum Sludge Characteristics. In *Proc. of the Joint Meeting of the Connecticut Water Works Association and the Connecticut Section.* New Haven, Conn.: Connecticut Water Works Association.

Cornwell, D.A., and H. Koppers, eds. 1990. *Slib, Schlamm, Sludge*. Denver, Colo.: AwwaRF, AWWA, and KIWA.

Cornwell, D.A., C. Vandermeyden, G. Dillow, and M. Wang. 1992. *Landfilling of Water Treatment Plant Coagulant Sludges.* Denver, Colo.: AwwaRF and AWWA.

Davis, M., and D. Cornwell. 1998. *Introduction to Environmental Engineering*. Boston, Mass.: McGraw-Hill.

Given, P., and D. Spink. 1984. Alum Sludge: Treatment Disposal and Characterization. In *Proc. of the 36th Annual Convention Western Canada and Sewer Conference*.

Hamon, J., Jr. 1986. The Effect of Treatment Process Variations on the Thickening and Dewatering Characteristics of Water Plant Sludges. Master's thesis. Virginia Polytechnic Institute and State University, Blacksburg.

Hsieh, H.N., and D. Raghu. 1997. *Criteria Development for Water Treatment Plant Residual Monofills*. Denver, Colo.: AwwaRF and AWWA.

Knocke, W., and D. Wakeland. 1983. Fundamental Characteristics of Water Treatment Plant Sludges. *Jour. AWWA*, 75(10):516.

MacPhee, M., D. Cornwell, and R. Brown. 2002. *Trace Contaminants in Water Treatment Chemicals*. Denver, Colo.: AwwaRF and AWWA.

Schmitt, C., and J. Hall. 1975. Analytical Characterization of Water Treatment Plant Sludges. *Jour. AWWA*, 67(1):40.

Shannon, L.D., C.H. Yu, R. Wieklinski, J. Jarvis, and D. DeKraker. 1997. *Beneficial Reuse of Lime Softening Residuals for Flue Gas Desulfurization*. Denver, Colo.: AwwaRF and AWWA.

USEPA (U.S. Environmental Protection Agency). 1983. 40 CFR Part 261. Identification and Listing of Hazardous Waste. April 1, 1983. *Fed. Reg*. 48:14293. Washington, D.C.: USEPA.

Vandermeyden, C., and D.A. Cornwell. 1998. *Nonmechanical Dewatering of Water Plant Residuals*. Denver, Colo.: AwwaRF and AWWA.

Vesilind, P. 1988. Capillary Suction Time as Fundamental Measure of Sludge Dewaterability. *Jour. WPCF*, 60(2):215–219.

CHAPTER 3
SPENT FILTER BACKWASH WATER

A major residual stream produced in terms of volume is from the batch process of backwashing filters. The solids collected on filters are those remaining after sedimentation, caused by the addition of a filter aid, or formed by oxidation of perhaps iron or manganese. In a direct filtration process, these are the only solids produced. The volume is a function of the amount of water used for backwashing. This residual stream is produced at very high flow rates for short periods of time.

REGULATIONS

Discharging SFBW, either treated or directly as produced, to receiving streams or wastewater treatment plants (WWTPs) is addressed in chapter 5. The other regulation important to SFBW is the filter backwash recycle regulation. In June 2001, USEPA finalized the Filter Backwash Recycling Rule (FBRR) to address a statutory requirement of the Safe Drinking Water Act (SDWA) Amendments of 1996 to promulgate a regulation that "governs" the recycling of filter backwash water within the treatment process of public water systems. The purpose of the FBRR is to require public water systems to review their recycle practices and, where appropriate, work with the state primacy agency to make any necessary changes to recycle practices that may compromise microbial control.

Prior to the FBRR, there was no federal regulation governing filter backwash recycling. Regulations within the United States regarding recycle had been established by individual states, if at all, and the filter backwash recycle regulatory approach varied by state. Currently, several states have some type of policy or regulation on recycle (Table 3.1). These policies range from detailed regulations to general philosophical statements.

According to USEPA, the FBRR applies to all public water systems that

- Use surface water or groundwater under the direct influence of surface water (GWUDI);
- Utilize direct or conventional filtration processes; and
- Recycle SFBW, sludge thickener supernatant, or liquids from dewatering processes.

The FBRR requires that recycled SFBW, sludge thickener supernatant, and liquids from dewatering processes be returned to a location such that all processes of a system's conventional or direct filtration plant, including coagulation, flocculation, sedimentation (conventional filtration only), and filtration, are employed. Systems may apply to the state for approval to recycle at an alternative location.

The FBRR also requires that systems notify the state in writing that they practice recycle. When notifying the state, systems must also provide the following information:

- A plant schematic showing the origin of all recycle flows, the hydraulic conveyance used to transport them, and the location where they are recycled back into the plant
- Typical recycle flow, highest observed plant flow experienced in the previous year, design flow for the treatment plant (all in gallons per minute); and the state-approved operating capacity for the plant where the state has made such determinations

Table 3.1
Example state recycle provisions

State	Policy or regulation
California	New facilities must remove 80% of solids before recycle; recycle flow <10% of plant flow
Illinois	Equalization for two filter backwashes
Maryland	New facilities must treat; current facilities should treat; recycle flow <5% of plant flow; treat using sedimentation + polymer
Nevada	New facilities must treat
North Carolina	No recycle of untreated water; recycle flow <10% of plant flow; need alternative discharge
Ohio	Recommends treatment; recycle flow 10% of instantaneous raw flow
Pennsylvania	Evaluation required; alternative discharge required
Vermont	Under certain conditions, recycle not allowed
Ten State Standards	Recycle <10% raw water flow; equalization basin size criteria; limits when recycle is acceptable

Finally, systems must collect and maintain the following information for review by the state, which may require a system to modify their recycle location or recycle practices:

- Copy of the recycle notification and information submitted to the state
- List of all recycle flows and the frequency with which they are returned
- Average and maximum backwash flow rate through the filters and average and maximum duration of the filter backwash process (in minutes)
- Typical filter run length and a written summary of how filter run length is determined (head loss, turbidity, time, etc.)
- Type of treatment provided for the recycle flow
- Data on the physical dimensions of the equalization and/or treatment units, typical and maximum hydraulic loading rates, type of treatment chemicals used and average dose and frequency of use, and frequency at which solids are removed where such units are used

Systems must notify the state with the appropriate information no later than 30 months after promulgation of the rule. Systems must comply with the recycle return location requirements of the FBRR by 36 months following promulgation of the rule. If a system requires capital improvements to modify the location of their recycle return, they must complete all improvements no later than 60 months after promulgation of the rule (USEPA 2004).

CHARACTERISTICS

Untreated SFBW

All water treatment facilities that have granular media filters (e.g., conventional, softening, iron removal, and direct filtration) generate SFBW during media cleaning operations. Untreated SFBW consists of the wash water and solids carried directly off the top of a filter during backwash operations. This recycle stream may or may not undergo any treatment before being recycled to the water treatment process or discharged to a stream or WWTP. Some characteristic data are contained in chapter 2, whereas this chapter has additional specific SFBW data.

The volume of SFBW generated at a WTP is dependent on several factors including backwash duration, wash-water rate, backwashing frequency (filter run length), and number of filters. Filters are typically backwashed every 1 to 4 days, depending on treatment effectiveness, water quality, and season. Many facilities stagger backwashes, so that even when filter run lengths are long, one or more filters are backwashed on a daily basis. Backwash duration and rate also vary with treatment process, water quality, and season.

Cornwell and Lee (1993) found that without removal of solids from SFBW, total trihalomethane formation potential (THMFP) and TOC levels in SFBW can be appreciable. The presence of preformed trihalomethanes (THMs) and haloacetic acids (HAAs) in SFBW at facilities that use chlorinated wash water could result in elevated disinfection by-product (DBP) concentrations in the combined influent when recycle is practical. TOC levels could also be increased.

The presence of chemical contaminants (as well as turbidity or solids) in SFBW can impact treatment and, potentially, finished water quality (Cornwell et al. 2001). For example, recycle of organic precursors or TOC could affect the ability of a plant to meet Stage 1 and 2 Disinfectants/Disinfection By-Products Rule (D/DBP) requirements. Cornwell et al. (2001) found that TOC levels in SFBW were on average higher by threefold compared to corresponding raw water levels. Observed increases in total trihalomethanes (TTHM) and the sum of six haloacetic acids (HAA6) were 92- and 24-fold, respectively, in SFBW. Average aluminum and iron levels in SFBW were 7- and 20-fold higher, while manganese levels were increased by nearly 13-fold. Table 3.2 summarizes the results of chemical monitoring. Corresponding average increases in aluminum and iron concentrations for plants using alum and ferric coagulants were somewhat higher (22-fold and 12-fold, respectively), while the average increase in aluminum levels for plants using polyaluminum chloride coagulants was much higher (55-fold).

Edzwald et al. (2001) showed that DOC, dissolved metals, and UV-254 were not elevated in untreated SFBW relative to raw water levels. However, that work did support the Cornwell et al. (2001) findings that total metals levels and TOC were elevated compared to the corresponding raw water.

Cincinnati Water Works (Hartman, Fredericksen, and Kashinkunti 1997) investigated the potential to return parasites in the untreated SFBW recycle stream. They detected *Giardia* cysts in 14 percent of the SFBW samples analyzed but found no detectable *Cryptosporidium* oocysts. Mass balance calculations showed that recycling the SFBW stream would not cause an increase in raw water parasite levels, even though particle counts, TSS, and turbidity levels in the SFBW were higher.

A database of pathogens and other chemical contaminants was developed (Cornwell et al. 2001, through investigation by LeChevallier) using 25 WTPs across the United States and

Table 3.2
Comparison of contaminant levels in raw water and SFBW

	Raw water		SFBW		Multiple increase
Parameter	Range	Average	Range	Average	
Al (mg/L)	ND[*]–30	0.72	ND–145.8	14.7	20.4
Br (mg/L)	ND–0.68	0.038	ND–0.46	0.033	–0.1
Fe (mg/L)	ND–56.6	1.2	ND–132	8.7	7.3
HAA6 (μg/L)	ND–21.5	1.9	ND–211	46.1	24.3
Mn (mg/L)	0.01–5.5	0.11	0.01–17.9	1.4	12.7
TTHM (μg/L)	ND–21.8	0.6	ND–198	55.0	91.7
TOC (mg/L)	0.7–5.4	2.4	0.8–191	8.0	3.3
Zn (mg/L)	ND–0.5	0.03	ND–1.0	0.1	3.3

Source: Cornwell et al. 2001.
*ND = not detected.

Canada. Raw water and SFBW at each of these water systems were sampled six times over a 1-year period.

Of the 146 raw water samples collected, *Giardia* and *Cryptosporidium* were detected in 30 and 11 percent of the samples, respectively. The observed geometric mean levels of *Giardia* and *Cryptosporidium* in the raw water samples for the detections were 89 and 108 per 100 L, respectively. For the 148 SFBW samples, 8 and 5 percent were positive for *Giardia* and *Cryptosporidium*, respectively. The geometric mean levels of *Giardia* and *Cryptosporidium* in the SFBW samples with detections were 203 and 175 per 100 L, respectively. All of the data were analyzed by means of the immunofluorescence assay (IFA) method.

The observed recovery efficiencies of *Giardia* and *Cryptosporidium* by the IFA method were 21 and 23 percent in the raw water samples, compared to 6.4 and 5.6 percent in the SFBW samples. The SFBW protozoan geometric mean levels in the positive samples (i.e., only where a detection was measured) were about seven times higher than the raw water levels after adjusting for recovery efficiencies.

When all of the samples were considered (i.e., with positive and nondetected results), the raw water geometric mean was nearly the same at 100 and 94 per 100 L for *Giardia* and *Cryptosporidium*, respectively. The SFBW geometric mean levels for all samples were about 2.5 times higher than for the positive samples at 470 and 450 per 100 L, respectively, for *Giardia* and *Cryptosporidium*. After adjusting for recovery efficiency, *Giardia* and *Cryptosporidium* in all of the SFBW samples were 16 and 21 times higher than those in the corresponding raw water samples.

The cell culture–polymerase chain reaction method was used after immunomagnetic separation to identify live *Cryptosporidium*. For this study, 122 raw water and 121 SFBW samples were analyzed. Live *Cryptosporidium* were observed in six raw water (4.9 percent) and nine

SFBW samples (7.4 percent). Occurrence of live *Cryptosporidium* in the SFBW samples suggests that viable oocysts can survive within the treatment process.

Overall, the data showed that live oocysts were found in a small percentage of SFBW samples. The total concentration of oocysts was on average seven times higher in the SFBW than in the raw water.

SFBW After Solids Removal

Treated SFBW is defined as the wash water remaining after some amount of the solids are removed through gravity settling or by other means. Removal of solids from SFBW results in only a minor reduction in volume compared to the original quantity of SFBW generated since its starting solids content is relatively low (well below 1 percent).

Cornwell and Lee (1993) found in two plants studied that although sedimentation with no chemical treatment was effective in reducing particles (and pathogen levels) in SFBW, very low overflow rates (<0.05 gpm/ft^2 [1.2 m/hr]) were required to achieve 70 to 80 percent particle removal in the *Giardia* cyst size range (5 to 15 µm). A nonionic polymer was effective in increasing particle removals to more than 90 percent at overflow rates of 0.2 gpm/ft^2 to 0.3 gpm/ft^2 (0.5 to 0.7 m/hr).

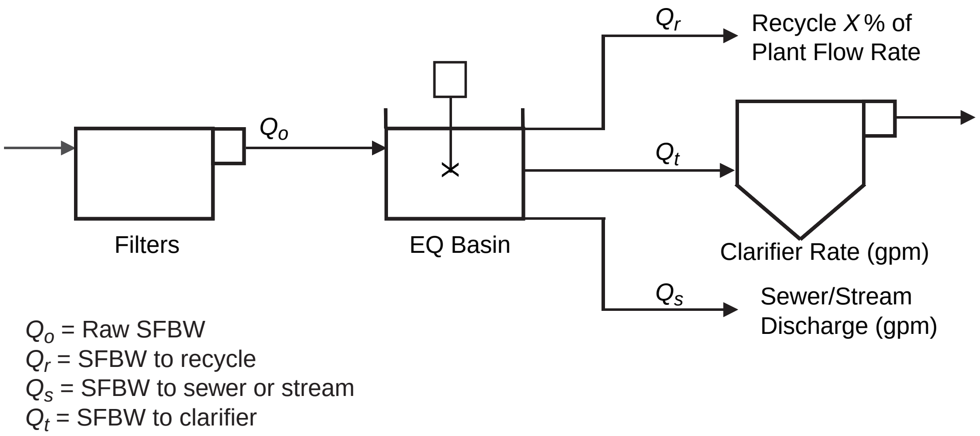
Research completed by Cornwell et al. (2001) on treatment options for SFBW yielded similar results, and also evaluated the effectiveness of more advanced treatment including DAF, clarification plus granular media filtration, and membrane MF on removal of particles. In fact, results demonstrated that when paired with polymer conditioning, both sedimentation and DAF resulted in particle removals of greater than 90 percent.

In a pilot spiking study published by the Foundation for Water Research in 1994 (Hall, Pressdee, and Carrington 1994), roughly 2 percent of oocysts present in SFBW remained in treated (settled) supernatant, indicating that gravity settling significantly reduced the pathogen concentrations. Reportedly, reliable *Giardia* cyst counts were difficult to determine because of the relatively high coagulant solids. The SFBW study by Hall, Pressdee, and Carrington (1994) suggests backwash water supernatant presents low *Cryptosporidium* levels, provided that it is well settled.

Research by Cornwell et al. (2001) indicated that treatment of SFBW to achieve at least 1 log of *Cryptosporidium* removal would reduce the cyst concentration in the SFBW to at or below raw water background levels, alleviating concerns over recycle or discharge to the environment. Other benefits of a 1-log treatment process could include reduction of contaminants such as TOC, Fe, Mn, and solids. Research also indicated that gravity treatment processes should also include the use of a polymer to aid solids settling. The type of polymer applied is important for achieving successful treatment of the recycle stream.

EQUALIZATION

SFBW is produced over a relatively short time period (around 15 minutes per filter) but at a high instantaneous flow rate. Equalization (EQ) is a method whereby the SFBW is captured in a storage device and released over a longer period of time and, therefore, at a lower flow rate. For example, capturing the volume of one backwash and releasing it over the next hour will reduce the flow to about 25 percent of the instantaneous rate. Reduction of the flow rate through EQ can be useful in sizing downstream treatment devices, reducing impacts to the main treatment plant if recycle is used, reducing impacts at wastewater plants or on sewer line size requirements, or for



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 3.1 SFBW EQ release criteria

reducing flow rates into the receiving stream. The allowed rate of release from the EQ basin is established by the downstream control (e.g., the sewer line capacity, regulatory or treatment limitations on recycle, stream flow discharge limits, etc., as depicted in Figure 3.1). The size of the EQ storage device is set by the SFBW produced per backwash, backwash frequency, and the allowed or design flow release.

The EQ basin storage volume is determined based on a flow balance of SFBW entering the basin and the rate of release of equalized SFBW leaving the basin. The storage curve is typically shown over a 24-hour period of fill-and-draw from the EQ basin. Because the rate of SFBW flow into the EQ basin usually exceeds the rate of discharge from the basin, the volume in the basin increases over time as long as filter backwashing is occurring. The volume in the EQ basin will then decrease until the next cycle of washes begins. The peak of the storage curve is the total EQ basin volume required to meet the backwash assumptions used.

In sizing an EQ tank, it is important to make sure that operation of the treatment plant itself is not impacted. Backwashing of a filter should not have to wait because the EQ tank is full, unless, of course, the plant has a lot of excess filter capacity. In order to size an EQ tank, the operators must determine the frequency at which they need to wash filters, how many in a row, how many per shift, etc. It is also important to associate a production flow with the number of filter washes. Some facilities may wash all filters each day independent of the raw flow, whereas others may operate fewer filters at lower production levels and hence wash fewer filters. Additional decisions need to be made regarding the volume of SFBW produced per day, the design (perhaps worst-case) number of filter washes per day, and the frequency of backwashes. One of the more critical decisions is determining frequency of backwashing filters. The criteria resulting in maximum flexibility would allow for backwashing one filter immediately after another until all filters are backwashed, which is referred to as sequential backwashing (some plants have the ability to backwash more than one filter at a time). This, of course, results in a large EQ basin. On the other end of the spectrum is a criterion of evenly spacing the backwashes. This criterion would minimize the EQ volume required but would also place the largest constraint on operations.

EQ basins can be used to meet various flow goals, as shown in Figure 3.1. They can be sized to meet recycle percentage goals, minimize downstream treatment facility sizes, or meet discharge flow limitations. EQ basins are often mixed to provide a uniform solids concentration in the discharge. The pumps from the EQ basins would be sized to meet the downstream flow restrictions (i.e., percent recycle, thickener loading rate, sewer allowed flow, etc.).

An example taken from sizing EQ facilities at a Cleveland Division of Water (CWD) WTP illustrates EQ sizing methods. The CWD Morgan plant has an existing conventional clarifier for treating SFBW prior to recycling into the treatment process. The recycle rate at minimum plant flow was determined to be the limiting condition for sizing EQ storage.

In order to determine the required EQ storage capacity such that filter backwashing is not limited, a list of backwash operation assumptions was developed by the CWD operators that represented the operational scenario for the Morgan WTP. These operational assumptions were used for calculating the EQ storage volume required and were based on present and future Morgan WTP operations information.

- Minimum plant flow rate = 55 mgd
  (95th percentile minimum flow)
- Instantaneous recycle goal (maximum) = 10 percent of raw flow
- Backwash volume = 260,000 gal/backwash
- Backwash frequency = 30 min/backwash event (first four filters)
  55 min/backwash event (remaining 24 filters)
- Maximum number of washes per day = 28

The required EQ basin storage capacity was calculated using these filter backwash operational assumptions. The parameters were incorporated into a 24-hour storage profile. An example EQ storage curve is presented in Figure 3.2. The curve is constructed by adding in the SFBW corresponding to the backwash schedule and subtracting out the allowed release rate. In this example, the allowed release rate was limited by the clarifier capacity of 3,375 gpm (213 L/sec). The curve demonstrates that a total EQ storage volume of 2.52 MG (9.5 ML) is required to store the cumulative volume of backwash water generated under these assumptions.

The storage curve used for determining required EQ storage volume also demonstrates that at the end of the day the EQ basins would not be empty. That means that more SFBW was produced than could be returned with the existing clarifier capacity. Therefore, during the next day of operation, filter backwashing would have to be halted until the EQ basins could drain down to a level that would allow for storage of additional SFBW. Because of the large EQ basin volume required using these assumptions, an alternative set of assumptions was developed by the CWD staff. For this alternative it was assumed that only the number of filters necessary to produce the minimum production capacity would be online during that day. An analysis of plant flow rate versus number of filters in service was conducted to determine the number of filter washes per day that would be expected if each operating filter was washed once per day. The results from this analysis are summarized in Table 3.3. At the maximum expected plant flow rate of 115 mgd (435 ML/d), a total of 28 filters would be in service. If each filter was washed once per day, the volume of SFBW would be approximately 7.3 MG (28 ML). At lower plant flow rates there would be fewer backwashes required because of having fewer filters in service. Also in this scenario, it was determined that an additional clarifier was needed so that the recycle limitation was a 10 percent return requirement, not the clarifier capacity.

At the lowest plant flow rate of 55 mgd (208 ML/d), the SFBW clarified discharge flow rate is limited by the 10 percent maximum recycle rate. Another EQ storage curve (Figure 3.3)

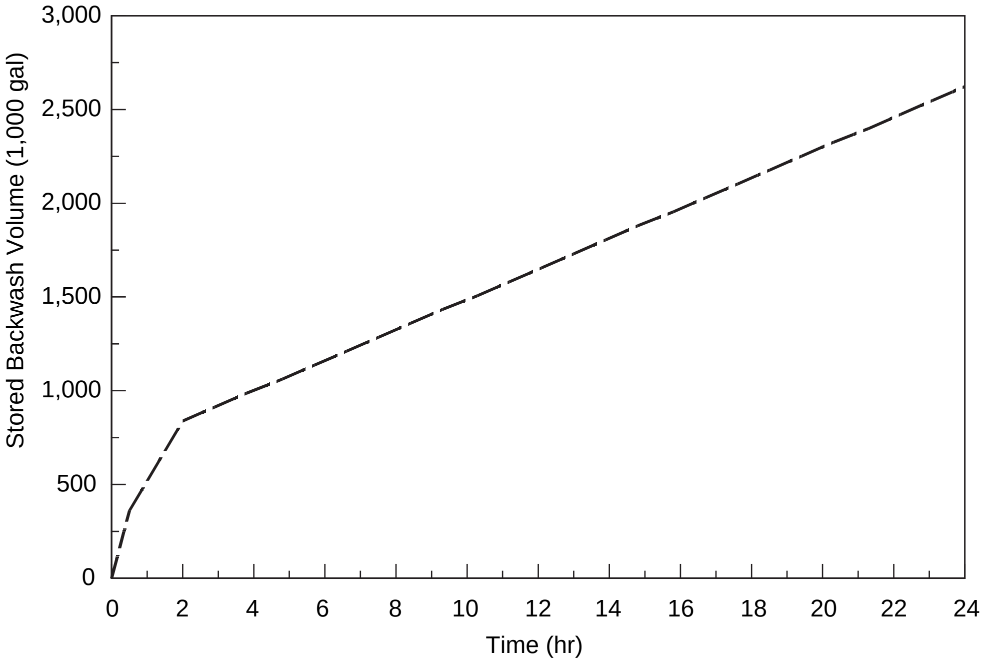


Figure 3.2 Example EQ storage curve used for determining backwash EQ storage capacity

Table 3.3
Morgan WTP filter operations

Plant flow rate (mgd)*	No. of filters in service	Total backwash volume generated (mgd)*
55	14	3.6
65	16	4.2
75	18	4.7
85	21	5.5
95	23	6.0
105	25	6.5
115	28	7.3

*mgd × 3.785 = ML/d.

was constructed using the original assumptions along with a 55-mgd (208-ML/d) raw flow rate and 14 filter backwashes per day from data in Table 3.3. At a flow rate of 55 mgd (208 ML/d), the maximum recycle rate at 10 percent is 3,819 gpm (240 L/sec). The curve shows that the cumulative stored EQ volume after 14 backwashes is approximately 1.2 MG (4.5 ML). Therefore, the limiting condition was a low-flow scenario with 14 filter washes per day resulting in a required EQ volume of 1.2 MG (4.5 ML).

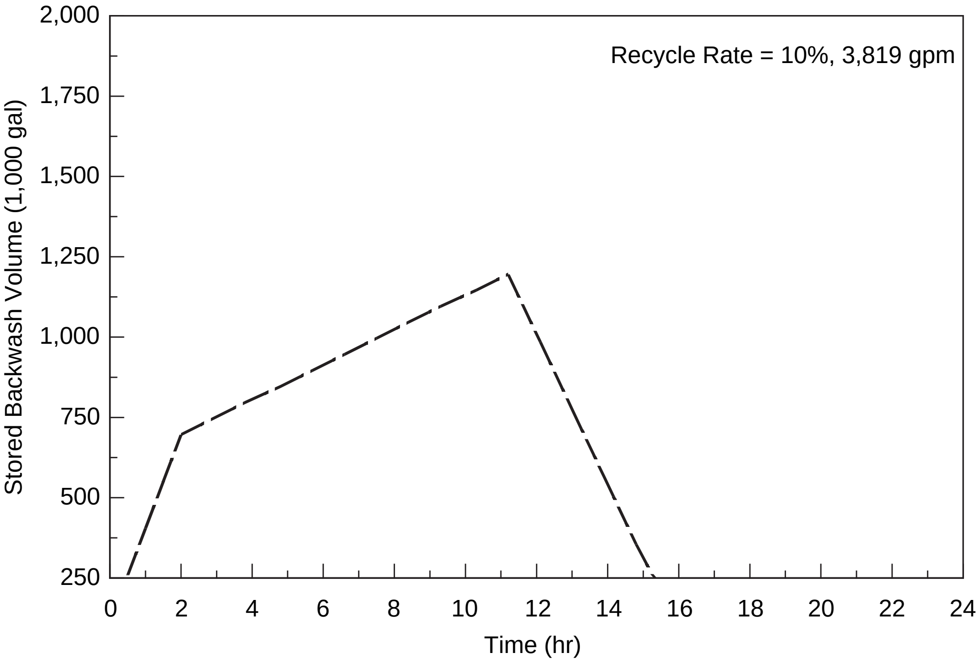


Figure 3.3 Example EQ storage curve for determining backwash EQ storage capacity with revised assumptions for backwash frequency

TREATMENT OF SFBW

SFBW typically has a lower suspended solids concentration than sedimentation basin blowdown but represents a greater residuals volume. SFBW typically flows by gravity to equalization basins, treatment processes, disposal, or recycle. EQ basins assist by buffering the flow of SFBW from filter washing into the treatment process, so that a controlled flow rate can be maintained. SFBW delivered to a lagoon system or batch clarification process may not require EQ as an intermediate step. SFBW treatment systems could include any of the following:

- Lagoons
- Conventional clarifiers
- Decant basins
- DAF
- Other high-rate clarification processes
- Membranes

Chemical addition is commonly used to increase the allowed hydraulic loading rate for clarification processes. Polymers are most commonly used to aid separation, but in some cases metal salt coagulants are utilized. The actual design parameters for many of these processes are discussed in chapter 4 and should be consulted for additional information.

Research conducted by Cornwell et al. (2001) demonstrated the range of turbidity and particle log reduction achievable using seven different process options for treatment of SFBW. These data are summarized in Table 3.4.

Table 3.4
Range of turbidity and particle log reductions from SFBW by treatment type

Process option	Turbidity log reduction	Particle log reduction
Sedimentation without polymer	0.1 to 0.8	0.2 to 0.9
DAF without polymer	0.7 to 1.4	0.8 to 1.7
Sedimentation plus polymer	1.4 to 2.3	1.9 to 3.3
DAF plus polymer	1.7 to 2.7	1.9 to 3.5
Coagulation followed by sedimentation	0.5 to 1.7	0.4 to 2.1
Filtration with pretreatment using DAF or sedimentation plus polymer	2.2 to 3.0	2.4 to 4.4
Membrane MF	2.6 to 3.9	1.6 to 3.5

Source: Cornwell et al. 2001.

These results suggest that the overall highest average particle and turbidity reduction from SFBW was achieved by filtration (preceded by clarification using DAF or sedimentation plus polymer) and MF. Both of these treatment approaches resulted in average particle and turbidity log reductions of about three. Gravity sedimentation with polymer was able to achieve 1.5- to 2-log reduction of turbidity and 2- to 3-log reduction of particles.

Selection of Residuals Treatment Process

Many plants do not treat residuals streams prior to recycle into the water treatment process or discharge to a stream or sewer. In some cases this may be an acceptable practice. WTPs that may need a residuals treatment system, or have a treatment system that is not capable of achieving effluent goals, may perform a residuals treatment evaluation. Bench- or pilot-scale testing can be used to provide the necessary information for evaluating and designing a residuals treatment system.

The first step in the selection of the best available treatment process is to determine the type of residuals stream to be treated and the specific contaminants required for removal. Based on this information, the list of available processes can be reduced to a few possibilities. Treatment processes using chemical addition followed by clarification or DAF could be simulated using bench-scale testing, while membrane systems would require a bench- or pilot-scale membrane unit. Bench- and pilot-scale testing can be used to perform the following tasks:

- Chemical screening
- Mixing strategies
- Comparison of treatment performance
- Determination of chemical dose
- Determination of influent blend ratios
- Evaluation of effluent quality

Several manufacturers have developed methods and/or equipment required to simulate their full-scale treatment processes using bench- or pilot-scale units. The data from these tests could be scaled up for designing a full-scale residuals treatment system. Bench-scale testing for evaluating use of conventional clarification systems for treating residuals is discussed in the following section.

Bench-scale testing can be used to simulate treatment residuals systems using conventional or other clarification processes. The specific tasks that are necessary when simulating full-scale residuals clarification systems using bench testing are as follows:

1. Determination of the blend ratios for testing
2. Polymer screening
3. Mixing velocity and duration analysis, if utilized
4. Bench tests and water quality analysis

Results from these tests should generate the information required to provide an idea of the quality of the residuals stream that is intended for recycle, and the design parameters for the full-scale clarification system required to achieve these water quality goals.

The first task of the bench evaluation is to determine which residuals streams will be tested, and whether blending will be used in the full-scale process. The stream to be treated could be a combination of multiple sources such as SFBW, sedimentation basin blowdown, thickener decant, dewatering liquid streams, and so forth. Before testing can be performed, a utility must first decide which blends occur in the full-scale WTP. Bench-scale tests are relatively quick, allowing for a range of full-scale blending scenarios to be evaluated.

A polymer screening program includes a selection of polymer products of each charge type (cationic, nonionic, anionic). It may also be desirable to evaluate polymers with different molecular weights. However, for residuals treatment, often the high-molecular-weight polymers perform better. There is no rule of thumb as to which charge works best for recycle streams.

A suggested procedure used to perform a quick polymer screening evaluation is as follows:

1. Use multiple beakers of at least 100 mL, each containing the sample to be tested.
2. Add 0.5 to 2 mg/L of polymer to each jar (NOTE: Dose required may be higher or lower depending on type of residual treated).
3. Rapid stir manually or by magnetic stirring bar for 30 seconds.
4. Allow sample to settle for 10 minutes and observe floc formation.
5. Collect decant sample and analyze for turbidity.

The above procedure relies heavily on visual observation to select candidate polymers for further evaluation.

In most cases, clarifiers and thickeners treating SFBW or sedimentation basin blowdown work quite well with minimal mixing. Quick polymer dispersion is always important and can generally be accomplished by adding the polymer near a pumping device. Flocculation is seldom warranted.

However, if polymer treatment is not adequate and a metal salt coagulant is used, then traditional mixing and flocculation become more important. In this case, standard jar test procedures should be used to determine the proper mixing conditions.

The best polymers identified during the screening process should be further evaluated using jar testing. A range of polymer doses is selected for evaluation to determine the optimal dose for each residuals stream tested. The jar test evaluation includes simulation of chemical feed, mixing, and sedimentation, along with sampling and water quality analyses.

If DAF or other clarification process is selected for jar test analysis, the procedures and equipment are different. In the case of DAF, a system manufacturer should be contacted to obtain the necessary test equipment and procedures, or the work should be outsourced to an entity having such equipment. Evaluation of membrane systems for treatment of residuals would also require pilot testing equipment from a manufacturer.

Full-Scale and Pilot-Scale Test Results

An example of a full-scale demonstration study conducted on a residuals treatment system is the work performed in 1999 for CWD (MacPhee, Charles, and Cornwell 2001). This project included an evaluation of a full-scale residuals settling basin and side-by-side pilot testing of DAF for SFBW treatment.

A total of three polymers were screened (bench scale) to determine the best polymer for use in the full-scale clarified demonstration testing. The polymers tested were as follows:

- Stockhausen A3040 LTR (anionic), Stockhausen Inc., Greensboro, N.C.
- Allied Colloids Percol LT 22S (cationic), Allied Colloids, Suffolk, Va.
- Allied Colloids Percol LT 26 (anionic), Allied Colloids

Polymers were mixed into the SFBW using a jar-testing apparatus, and turbidity and particle count samples were collected at different times during settling. The jar test sample collection times corresponded to an overflow rate range of 0.05 to 1.25 gpm/ft^2 (0.12 to 3 m/hr). The untreated SFBW had a turbidity of 86 ntu and a total particle count >2 µm of approximately 144,600 particles/mL. Each polymer was dosed at approximately 0.5 mg/L into the SFBW.

The results show that each of the polymers was effective at removing turbidity from the SFBW. The Percol LT 26 and Stockhausen A3040 LTR (both anionic) each performed slightly better in terms of turbidity removal than the Percol LT 22S (cationic). The particle count data collected during bench testing shows that the anionic polymers seemed to slightly increase particulate removal. The Percol LT 26 generated the lowest total particle count of between 500 and 750 particles/mL.

Overall, both Percol LT 26 and Stockhausen A3040 LTR performed very well for both turbidity and particle removal. In order to simplify polymer feed system operation at the Morgan WTP and minimize space requirements, it was determined that an emulsion polymer feed system would be most practical. Only the Stockhausen A3040 LTR is an emulsion polymer; the Percol products are both dry polymers. The Stockhausen A3040 LTR was therefore selected for full-scale demonstration testing.

The Stockhausen A3040 LTR dose range was investigated by feeding 0.25, 0.5, and 1.0 mg/L to SFBW in jars and measuring settled turbidity over time. The results show that in order to lower turbidity to less than 2 ntu at an overflow rate of 0.6 gpm/ft^2 (1.5 m/hr), a dose of at least 0.5 mg/L is required. Based on the results, the 0.5 mg/L dose was selected for full-scale testing.

During clarifier demonstration testing, the SFBW turbidity was sampled from the clarifier influent pipe prior to polymer addition and from the effluent pipe after clarification. Turbidity data

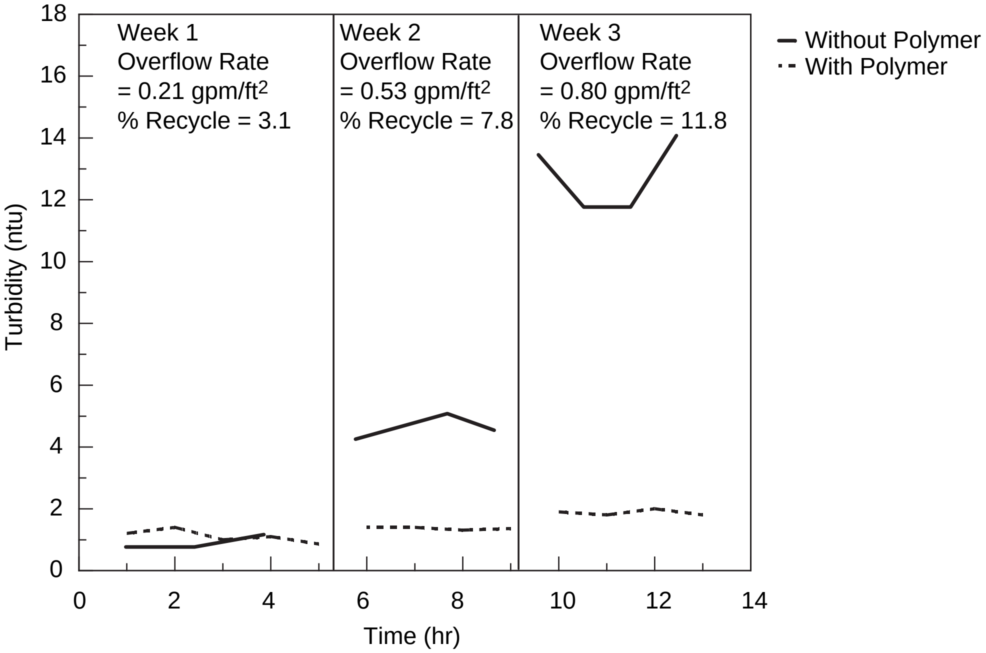


Figure 3.4 Impact of overflow rate and polymer addition on SFBW clarifier effluent turbidities at CWD

from the 3-week study are summarized in Figure 3.4. The data demonstrate that at the lowest overflow rate tested (0.21 gpm/ft^2 [0.5 m/hr]), addition of polymer did not improve solid–liquid separation in the clarifier compared to settling with no polymer addition. However, at the two higher overflow rates tested (0.53 and 0.80 gpm/ft^2 [1.3 and 2 m/hr]), polymer feed significantly lowered clarifier effluent turbidities. The figure shows that at the 0.53 and 0.80 gpm/ft^2 loading rates with polymer addition, effluent turbidity was maintained at below 2 ntu. With no polymer addition, the clarifier effluent turbidity was 5 and 13 ntu, respectively.

The particle count data (Figure 3.5) show the same trend as the turbidity data. Again, at the lowest overflow rate tested, the effluent particle counts with and without polymer addition were similar. At the middle and high loading rates, the clarifier effluent particle count (>2 µm) with no polymer addition was at least twice as high as particle counts generated using polymer conditioning.

Removal of TOC from the SFBW by the clarifier was also evaluated. During the 3 weeks of testing, the SFBW had an average TOC concentration of 4.49 mg/L. The data indicate that neither the clarified overflow rate nor the addition of polymer impacted TOC removal. The average clarified effluent TOC concentration measured during the test period, both with and without polymer, was 2.28 mg/L (48 percent removal).

Clarified influent and effluent samples were collected during demonstration testing and analyzed for instantaneous and simulated distributed system (SDS) TTHM. The average influent TTHM concentration for the 3-week test period was approximately 10 µg/L. No significant differences in TTHM concentration were noted in the clarified effluent at the various overflow rates tested. The average effluent TTHM concentration was 9 µg/L. If the SFBW clarified effluent, for

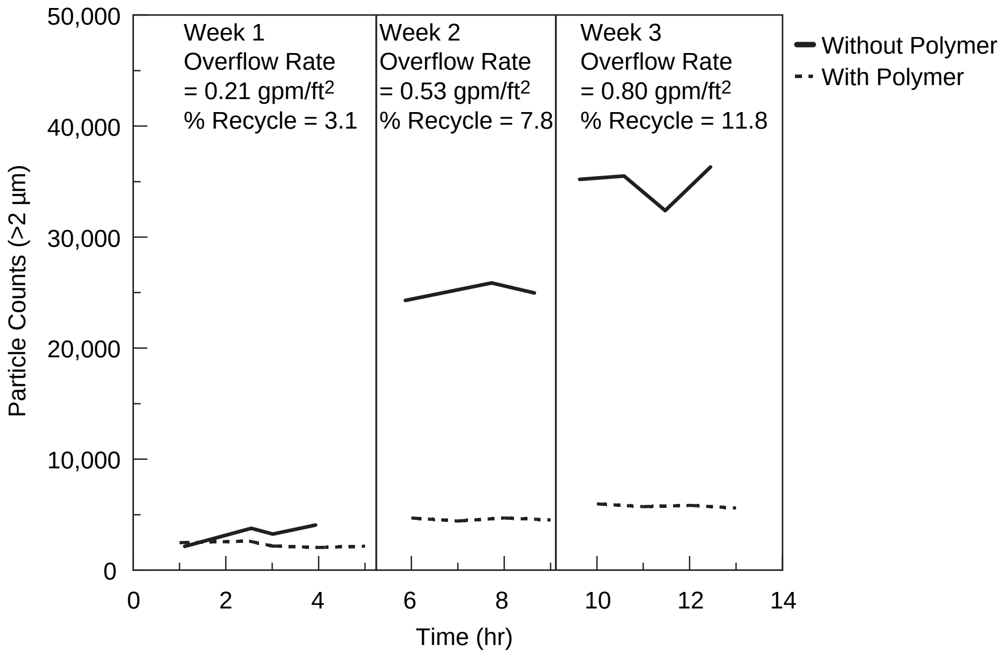


Figure 3.5 Impact of overflow rate and polymer addition on SFBW clarifier effluent particle counts at CWD

example, is 10 µg/L and the plant recycles 10 percent, then the recycle water adds 1 µg/L to the TTHM level leaving the WTP.

SDS TTHM concentrations were also measured from the influent SFBW and clarified SFBW. The influent SDS TTHM concentration averaged 94 µg/L during the 3-week period. The clarified SFBW was able to lower the effluent SDS TTHM concentration to 48 µg/L (48 percent) for weeks 1 and 2, and to 72 µg/L (26 percent) during week 3. Polymer addition would probably maintain the lower TTHM level.

Clarified influent and effluent samples were collected throughout testing for the sum of five haloacetic acids (HAA5) analysis. These data show that the influent and effluent HAA5 concentrations for each of the three overflow rates tested were very similar. The instantaneous HAA5 concentration measured from the clarifier (influent and effluent) during the 3-week test period averaged <10 µg/L.

An SDS HAA5 analysis was also performed using the clarified influent and effluent samples. The data demonstrate that increasing the clarified overflow rate did not affect SDS HAA5 removal. The highest overflow rate tested (0.8 gpm/ft^2 [2 m/hr]) had an SDS HAA5 removal of 48 percent. The backwash clarifier effectively removed turbidity and particulates from the SFBW at a maximum overflow rate of up to 0.8 gpm/ft^2 (2 m/hr) when polymer was added. Clarified effluent turbidity did not exceed 2 ntu when polymer was applied. The clarifier provided a TOC removal of 48 percent during testing and effectively reduced the clarified effluent SDS TTHM and SDS HAA5 concentrations. Based on test results, a conservative maximum loading rate for clarifier operation was set at 0.6 gpm/ft^2 (1.5 m/hr).

A pilot-plant study was also conducted at CWD to evaluate DAF for treatment of SFBW. A mobile DAF pilot unit was set up for 2 weeks of continuous pilot testing. SFBW was pumped from the EQ basin to the pilot unit. The pilot-plant objectives were to determine the following design variables based on pilot performance:

- Polymer dose optimization
- Flocculation mixing intensity
- Flocculation detention time
- DAF loading rate
- Sludge removal frequency

The water quality parameters monitored for determining DAF performance include turbidity, particle counts, color, iron, manganese, pH, and TSS. During the week that the clarified evaluation and DAF pilot testing overlapped TOC sampling, DBP sampling was conducted. The DBP sampling program for the DAF unit included collection of influent and effluent samples for instantaneous TTHM and HAA5 and SDS TTHM and SDS HAA5.

DAF pilot testing included the use of two different polymers—Percol LT 22S and Percol LT 26. These polymers were selected based on bench-scale jar tests. The Percol LT 22S (cationic) was determined to be the best polymer during pilot testing and indicated good performance at a dose rate of 0.3 mg/L. DAF performance was evaluated at a loading rate of between 3 and 6 gpm/ft^2 (7.3 and 14.6 m/hr), with a recycle rate of 8 to 10 percent.

Turbidity and particle count data were the primary indicators of treatment performance. The influent turbidity of the SFBW during testing averaged 25 ntu and ranged between 10 and 60 ntu. The DAF unit was consistently able to reduce the turbidity to <2 ntu up to the 6-gpm/ft^2 (14.6-m/hr) loading rate. Test results also demonstrated that flocculation mixing intensity and detention times did not impact DAF treatment performance within the allowed range of the pilot plant.

Operation of the DAF pilot plant coincided with the clarifier performance testing. DAF influent and effluent samples were collected simultaneously with clarifier sampling to compare removal of turbidity, particle counts, TOC, instantaneous TTHM and HAA5, SDS TTHM, and SDS HAA5.

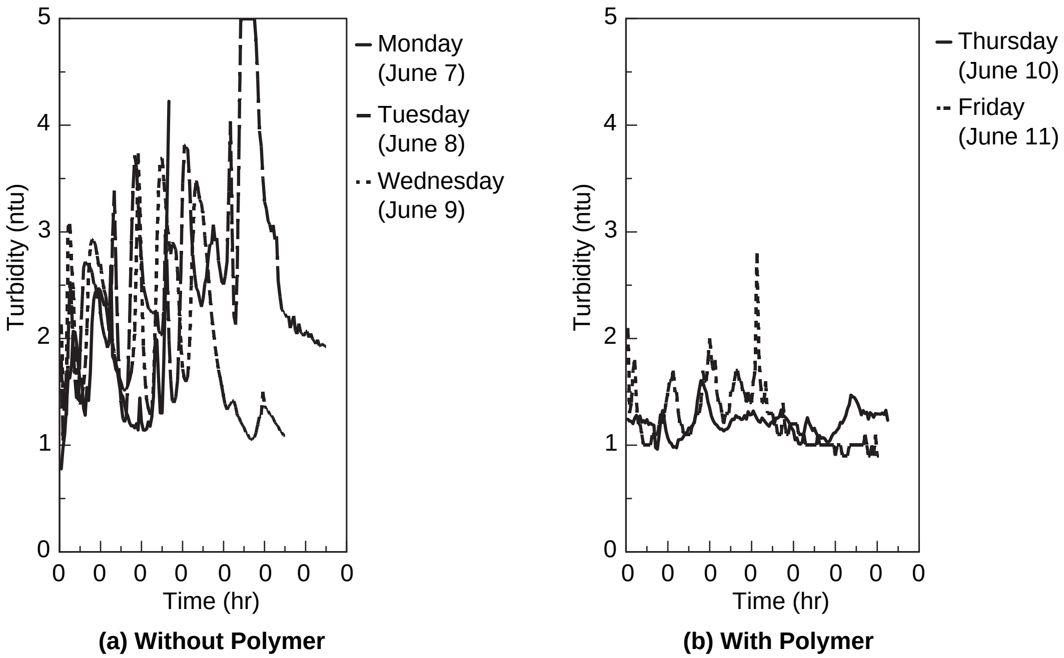
The clarifier and DAF pilot system were each able to lower SFBW turbidity to <2 ntu during testing. The DAF effluent turbidity during the 2-week test period averaged 1 ntu at a loading rate of up to 6 gpm/ft^2 (14.6 m/hr). The clarifier required polymer addition to generate an effluent turbidity of 2 ntu or lower at up to 0.8 gpm/ft^2 (2 m/hr).

The influent TOC concentration of the SFBW ranged between 4.5 and 5.5 mg/L during this week of testing. Both treatment processes were able to lower the effluent TOC concentration to approximately 2.0 mg/L (50 percent removal).

Instantaneous THM and HAA5 results showed that there were no differences in removal of these parameters using either treatment process. Both the DAF and clarified influent and effluent concentrations for both TTHM and HAA5 results ranged from 7 to 12 µg/L.

SDS TTHM and SDS HAA5 results showed that, again, the two treatment processes resulted in similar concentrations of SDS TTHM and SDS HAA5. The influent SDS TTHM and SDS HAA5 concentrations averaged approximately 100 µg/L, whereas the DAF and clarified effluent was less than 50 µg/L for each parameter.

Scale-up from bench to full scale and the importance of polymer addition in SFBW clarification was evaluated at the Central Utah Water Conservancy District (CUWCD) direct filtration plant, which is equipped with a conventional sidestream process for treatment of SFBW intended to recycle (Cornwell et al. 2001). The sidestream plant included rapid (static) mix, flocculation,



(a) Without Polymer **(b) With Polymer**

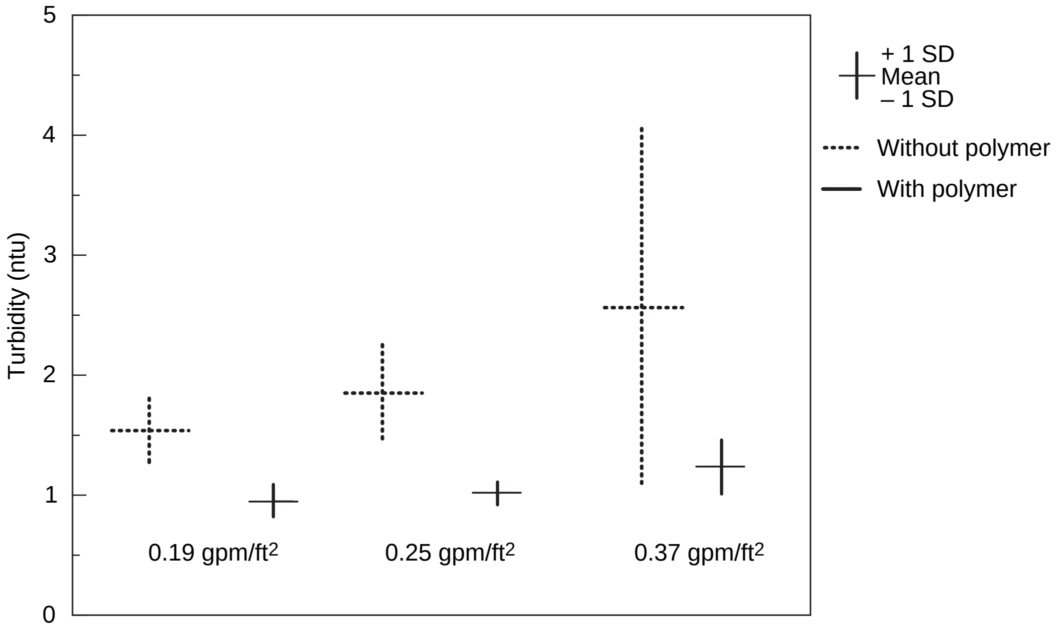
Source: Cornwell et al. 2001.

Figure 3.6 Impact of polymer addition on SFBW settled turbidity in full-scale testing at CUWCD

and sedimentation with tube settlers. Because of the two-train configuration of the sidestream plant, it served as a flexible venue to examine key treatment variables, including the effect of loading rate and impact of polymer addition. Influent and effluent turbidities were tracked for more than a month in the sidestream plant, and effluent particle counts were also measured. The tube settler overflow rate range investigated in the plant was 0.19 to 0.37 gpm/ft^2 (0.5 to 0.9 m/hr), and treatment was compared with and without polymer. The polymer type and dose were determined in bench-scale work.

Figures 3.6 (a) and (b) show the effects of adding polymer to the full-scale sidestream treatment plant for sedimentation of SFBW. Adding the appropriate polymer type and dose at a loading rate of 0.37 gpm/ft^2 (0.9 m/hr) reduced the range of turbidities from 1 to 4 ntu (other than a couple of excursions) down to 1 to 1.8 ntu. Polymer addition also lowered the average settled turbidities by 50 percent, from 2.4 ntu to 1.2 ntu. This was accomplished by addition of only 0.1 mg/L of the appropriate polymer. Online particle count data showed the same trends.

The impact of loading rate on average and settled turbidities from the full-scale sidestream plant, with and without polymer, is shown in Figure 3.7. The figure shows that without polymer addition, average settled turbidities increased steadily as the tube settler overflow rate was increased in successive weeks from 0.19 to 0.25 to 0.37 gpm/ft^2 (0.5 to 0.6 to 0.9 m/hr). More striking is the increasing variability of the data with increasing loading rate with no polymer addition as evidenced by the size of the ±1 SD (±1 SD) bars shown on the plot. By contrast, when the 0.1-mg/L polymer dose was added, average settled turbidities increased only marginally as the loading rate was doubled from 0.19 to 0.37 gpm/ft^2 (0.5 to 0.9 m/hr). Average turbidity was also



Source: Cornwell et al. 2001.

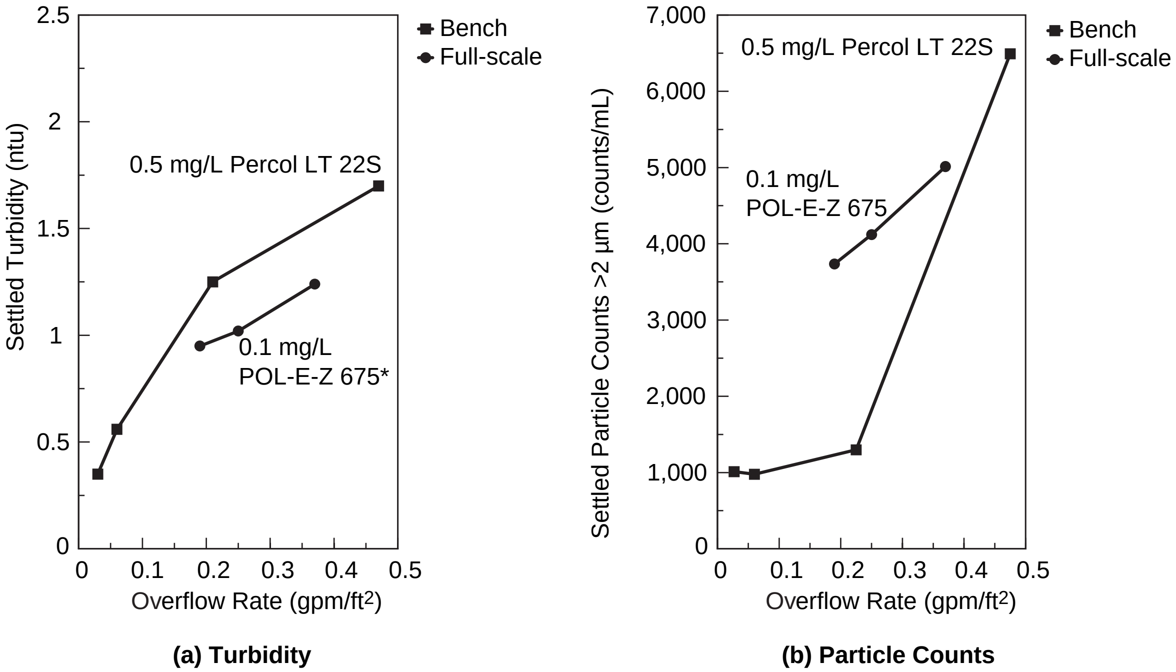
Figure 3.7 Impact of loading rate and polymer on SFBW settled turbidity in full-scale testing at CUWCD

decreased by polymer addition, as shown in Figure 3.7. The particle count data exhibited all of the same trends.

The bench-scale sedimentation work scaled up extremely well to full scale for both turbidity and particle counts. Figure 3.8 (a) shows that the settled turbidity predicted on the bench was within about 0.2 ntu of that attained in full-scale recycle treatment over the range of overflow rates evaluated. Settled particle counts achieved in the full-scale sedimentation tests under optimized conditions were within 1,000 counts/mL, despite the fact that a different optimal polymer type was identified and used in bench testing. The particle count scale-up data are presented in Figure 3.8 (b).

RETURN OF SFBW TO THE TREATMENT PLANT (RECYCLE)

For many years recycling SFBW back to the head of the main treatment plant has been a common practice. Recycle of SFBW has the advantages of conserving water and eliminating the need for discharge permits. A survey in 2001 (Cornwell et al. 2001) reported that more than half of the conventional surface water plants in the United States recycle SFBW on a routine basis. As finished water quality standards have become more stringent, all aspects of water treatment, including recycle, have been examined to find ways to consistently meet these stricter water quality standards. Recycling has come under particularly close scrutiny because of the known presence of *Giardia* and *Cryptosporidium* in raw water. *Cryptosporidium* is resistant to normal disinfection practices, so physical removal is critical. Because SFBW collects and concentrates any pathogens reaching the filters, the pathogens must be removed either from the SFBW prior to recycle or the additional oocysts must be efficiently removed by the main treatment process.



(a) Turbidity **(b) Particle Counts**

*Nalco Chemical, Ellwood City, Pa.
Source: Cornwell et al. 2001.

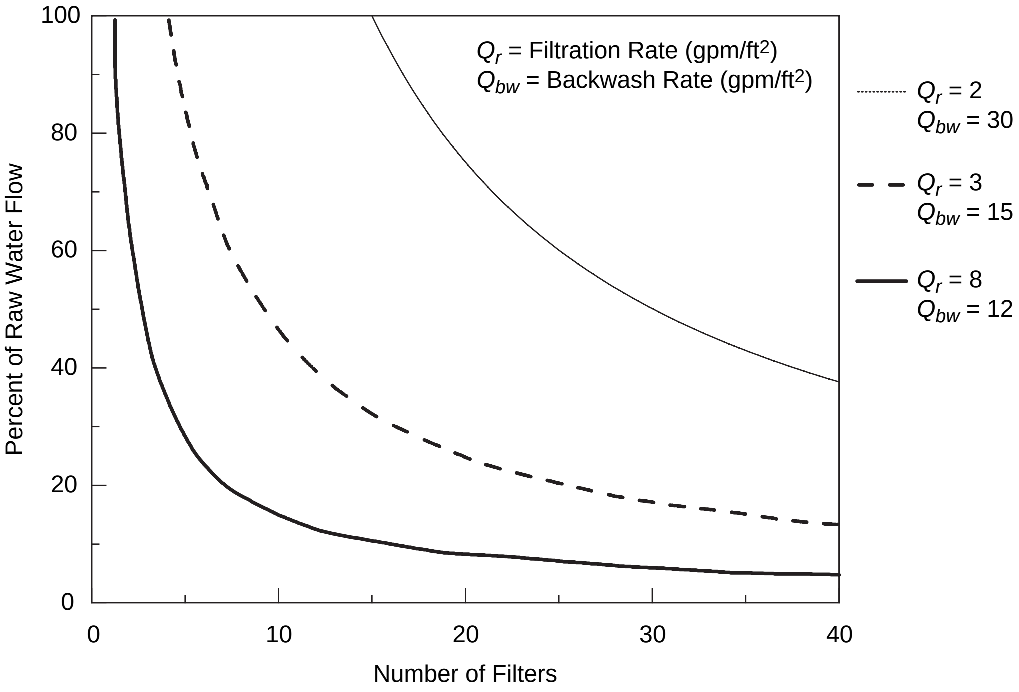
Figure 3.8 Comparison of SFBW settled turbidity and particle counts achieved in bench testing versus full-scale testing at CUWCD

Recycle of SFBW can impact the treatment plant by potentially causing hydraulic upsets or altering the raw water quality in a way that could affect treatment or finish water quality. Because SFBW is produced at a high instantaneous rate, if it is directly recycled it can cause a hydraulic upset, especially for plants with only a few filters.

Figure 3.9 shows the percentage of raw water flow (assuming it is kept constant) that would be recycled if SFBW were directly recycled. As the number of filters is reduced to less than 10, very significant hydraulic surges can result depending upon the ratio of the filtration hydraulic loading rate to the backwash hydraulic loading rate. This impact can be controlled through the use of EQ facilities previously discussed.

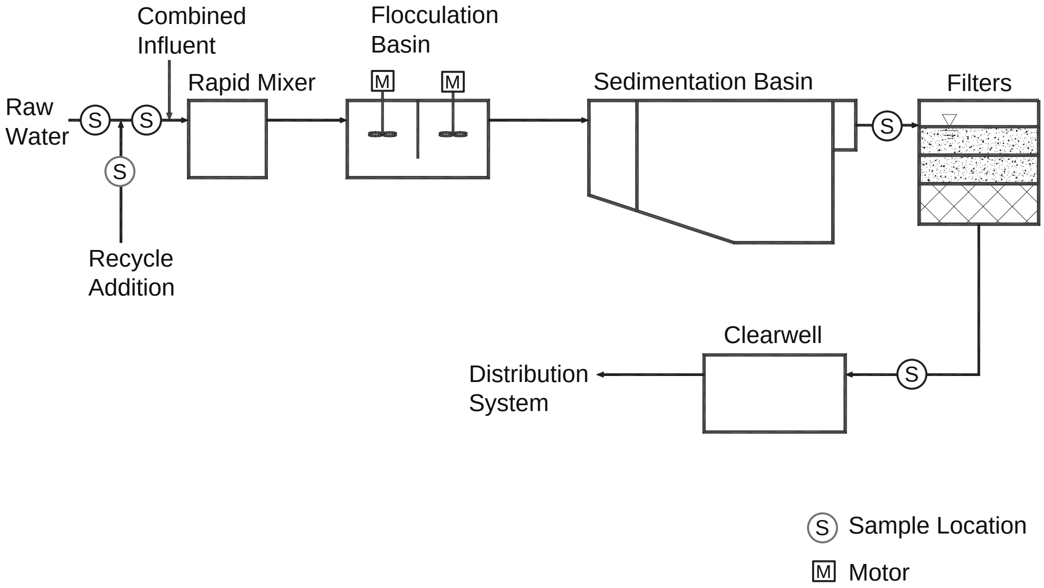
For WTPs that currently recycle, evaluating the impacts of recycle water on plant performance and water quality can be conducted. Sampling locations used to determine the impacts of recycle on water quality should focus on three treatment process streams—raw water, settled water, and finished water. These sampling locations are illustrated in a process schematic of a conventional WTP, shown in Figure 3.10. The schematic also shows a sampling location for the combined raw water and recycle streams (plant influent).

The settled water sample should be collected after the clarification process, but prior to filtration, and is sometimes referred to as applied water. Settled water quality evaluations are useful for determining if recycle is impacting primary coagulation and clarification. Settled water



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 3.9 Percent of plant flow that is recycled with no EQ



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 3.10 Sample locations for process water quality evaluations on recycle impacts

quality can vary more than filtered water quality and, therefore, may provide a clearer indication of potential recycle impacts.

The final sampling location is filtered water. The impact of the recycle stream on filtered water quality could be a determining factor for either practicing or discontinuing recycle of certain residuals streams, or may indicate the need for treatment of the residuals stream prior to recycle.

In order to gather the data required to determine recycle impacts on water quality, the WTP must be sampled both during recycle and no-recycle periods. In order to directly compare these data, sampling should be conducted during periods of stable raw water quality under consistent plant operations. Collection and analysis of water quality data should occur over an extended period of time to develop a more accurate assessment of seasonal impacts. Personnel may choose to continuously assess recycle impacts as part of its routine monitoring, or alternatively conduct its recycle impact evaluations periodically. Regardless of sampling frequency, the key is to generate enough data to accurately determine whether WTP operations or finished quality are altered by recycle.

To obtain these data in WTPs that continuously recycle, the utility would need to make provisions that would allow operators to discontinue recycle flow for periods of time such that samples could be collected with no recycle addition. WTPs that recycle intermittently should be able to collect water quality samples with and without recycle addition, although choosing the time of the sample event is an important consideration. Operators should carefully determine the hydraulic detention times and note whether or not each sample was collected during a period in which the recycle stream was added to the raw water. These determinations will be important for separating and analyzing the water quality data over time.

It is also possible that a WTP may have two or more separate process trains used for treatment. If this is the case, then one process train could include recycle addition while the other train simultaneously treats only raw water. In this fashion, the effects of recycle addition could be compared in real time. This would only be possible if the WTP was able to hydraulically isolate the two trains throughout the entire treatment process.

It may also be important that sampling used to evaluate recycle impacts be conducted during seasonal variations or during times of difficult raw water quality situations.

The frequency of sampling is clearly a site-specific issue. A reasonable approach is to select a month for some extensive testing to allow operations personnel to clearly understand the residuals characteristics. After that month, the utility may decide that there are no issues requiring further evaluation. It may also be reasonable to conduct quarterly followup sampling to view ongoing quality events. If the 1-month test program reveals areas of concern, then a specific test program aimed at those issues can be devised.

The recommended water quality parameters that can be used to assess water quality variability are included in Table 3.5.

When possible, online monitoring of turbidity, particle counts, temperature, and pH are recommended to provide continuous water quality data readings. Online turbidimeters are recommended for monitoring raw, raw and recycle blend, settled, and finished water. Online monitoring of pH, chlorine residual, and particle counts could also assist with evaluating treatment process water quality variability over time. It should be noted that online analyses of these parameters for recycle streams with high suspended solids concentrations is usually not possible because of interference of solids.

Table 3.5
Suggested water quality parameters for recycle evaluation

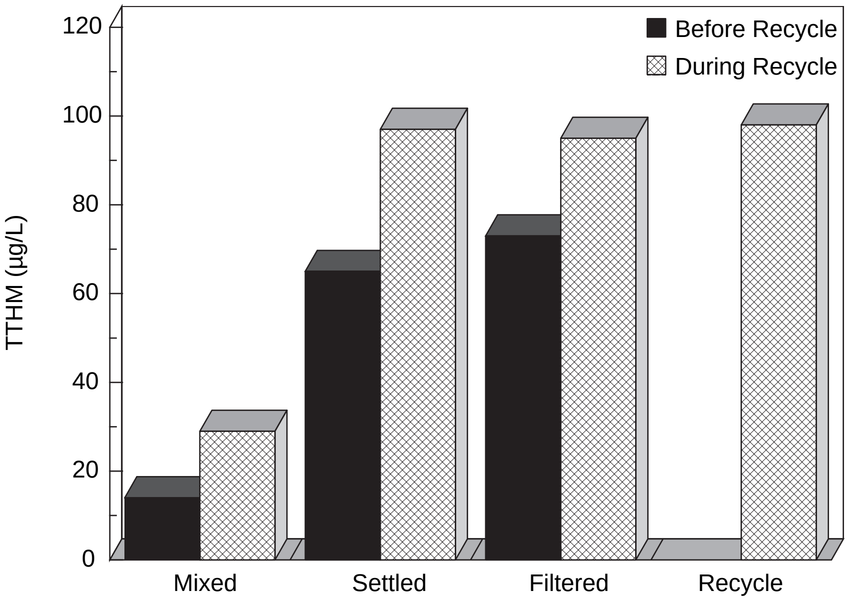
Parameter	Sampling locations	Suggested collection frequency
Turbidity (ntu)	Raw, recycle, combined influent, settled, finished	Hourly or online
TOC (mg/L)	Raw, recycle, combined influent, finished	Daily*
Color (cu)	Raw, recycle, combined influent, settled	Hourly
Aluminum (mg/L)†	Recycle/finished	Daily
Temperature (°C)	Raw, recycle, combined influent	Hourly
pH	Raw, recycle, combined influent	Hourly
Iron and manganese (mg/L)‡	Raw, recycle, combined influent, finished	Daily*
TTHM (µg/L)‡	Finished	Daily*
HAAs (µg/L)‡	Finished	Daily*
Particle counts (total counts/mL)	Finished	Hourly or online

*Collection frequency during short-term demonstration studies after steady state is achieved.
†Only for WTPs using PACl or alum as coagulant.
‡Site specific.

A utility may find that only a few of these analytical parameters are needed for assessing impacts of recycle on water quality at the WTP being evaluated. If this is the case, then sampling and analysis should focus on these parameters.

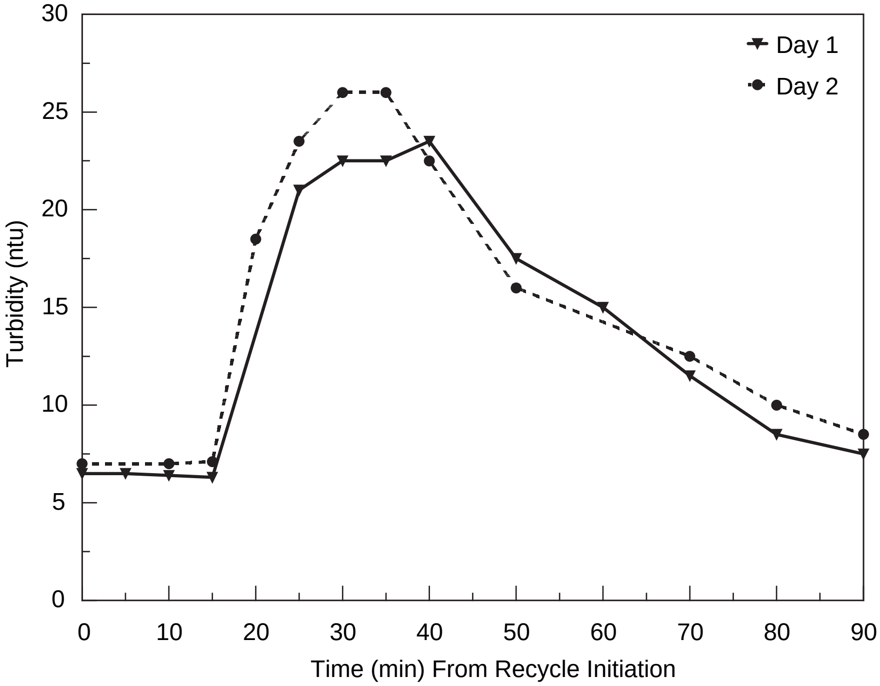
Several examples where utilities have conducted recycle impact evaluations were described by Cornwell and Lee (1993), a few of which are presented as illustrations of the type of data that can be analyzed. At one facility, untreated SFBW was recycled to the head of the conventional water plant. Several analyses were performed at the plant before (without recycle) and during recycle. Figure 3.11 shows TTHM values in the mixed influent (raw water plus recycle), settled and filtered water, and recycle stream itself. Since this is recycle of unsettled SFBW, the TTHM levels are relatively high in the recycled stream because of the presence of solids. Given the TTHM range relative to Stage 1 D/DBP regulations, recycle was causing an impact clearly worth investigating at this utility.

Figure 3.12 shows how raw water turbidity can be traced to assess the impact that recycle may be having on combined influent quality. In this case, raw turbidity spiked for about an hour as recycle was initiated. This increase may or may not translate into a settled or filtered water impact. (In the case of the plant shown in Figure 3.11, no impact on filtered water turbidity was found).



Source: Cornwell and Lee 1993.

Figure 3.11 Monitoring of TTHM to assess recycle impacts



Source: Cornwell, MacPhee, and Mutter 2003.

Figure 3.12 Example of historical combined influent turbidity plot with intermittent recycle events during 24-hour period

Edzwald et al. (2001) examined the impacts of recycle on clarified and filtered waters at six plants in the northeastern United States and reported the results in that document.

Cornwell and Lee (1993) estimated that no treatment (or even minimal treatment) of SFBW could result in high pathogen loading to the head of the plant. They recommended the use of equalized, continuous recycle, proper residual stream treatment prior to recycle, and characterization of residual stream quality through proper monitoring. However, even with elevated pathogen loadings in SFBW, the main process sedimentation basin may be capable of removing any additional cyst loading prior to reaching the filters, and the filters may be able to readily remove these coagulated pathogens. Therefore, direct treatment of the SFBW prior to recycle may not be needed.

Cornwell and MacPhee (2001) supported this theory. They found that increased loading of particles and *Cryptosporidium* oocysts from SFBW recycle did not deteriorate performance of conventional sedimentation or dual-media filtration systems under conditions tested in pilot studies. The SFBW recycle scenarios that were investigated included: (1) no recycle; (2) continuous recycle at 4.3 percent of plant flow; (3) intermittent recycle at 10 percent of influent plant flow (10-hour daily recycle); and (4) intermittent recycle at 20 percent of influent plant flow. Measurement of *Cryptosporidium* oocysts, turbidity, and particles >2 µm in both settled and filtered water showed that these levels were as low, or lower, during studies using any of the SFBW recycle scenarios than during similar studies using no SFBW recycle. The data qualitatively supported the hypothesis that as the oocysts to the plant increased, removal increased correspondingly in order to maintain the same finished water quality. The increased removal appeared to take place in the sedimentation basin. Edzwald et al. (2001) found similar results for DAF and plate settlers.

Although SFBW treatment may not always be necessary for pathogen removal, it may be necessary to remove solids from the recycle stream in order to minimize the risk of WTP upset due to TSS, turbidity, Fe, and Mn or other concentrated contaminants.

By conducting sampling, a utility can make informed judgments as to whether recycle is having an effect on its treated water quality. If an impact is seen, the source of that impact can be better understood; then either EQ to reduce the instantaneous recycle flow or treatment of the SFBW to remove the contaminant of concern can be utilized. It has generally been found (Cornwell and Lee 1993; Cornwell et al. 2001; Edzwald et al. 2001) that removal of the solids from the SFBW removes a majority of the contaminants, although it has also been found that turbidity alone is not a good indicator of contaminant removal and that particle counting should be employed.

REFERENCES

Cornwell, D.A., and R. Lee. 1993. *Recycle Stream Effects on Water Treatment*. Denver, Colo.: AwwaRF and AWWA.

Cornwell, D.A., and M.J. MacPhee. 2001. Effects of Spent Filter Backwash Recycle on *Cryptosporidium* Removal. *Jour. AWWA*, 93(4):153.

Cornwell, D.A., M.J. MacPhee, N. McTigue, H. Arora, G. DiGiovanni, M. LeChevallier, and J. Taylor. 2001. *Treatment Options for* Giardia, Cryptosporidium, *and Other Contaminants in Recycled Backwash Water*. Denver, Colo.: AwwaRF and AWWA.

Cornwell, D.A., M.J. MacPhee, and R.N. Mutter. 2003. *Self Assessment of Recycle Practices*. Washington, D.C.: AWWA.

Edzwald, J.K., J.E. Tobiason, M.B. Kelley, G.S. Kaminski, H.J. Dunn, and P.B. Gallant. 2001. *Impacts of Filter Backwash Recycle on Clarification and Filtration*. Denver, Colo.: AwwaRF and AWWA.

Hall, T., J. Pressdee, and N. Carrington. 1994. *Removal of* Cryptosporidium *Oocysts by Water Treatment Process*. United Kingdom: Foundation for Water Research.

Hartman, D., M. Fredericksen, and R. Kashinkunti. 1997. Alternatives to Untreated Filter Backwash Water Recycle at Cincinnati Water Works. Paper presented at the WEF/AWWA Biosolids Conference, Philadelphia, Pa., August.

MacPhee, M.J. 2001. Filter Backwash Recycle Rule—What a Utility Needs to Know. In *Proc. of the Ohio AWWA Section SDWA Seminar*. Columbus, Ohio: AWWA.

MacPhee, M.J., G.E. Charles, and D.A. Cornwell. 2001. *Treatment of Arsenic Residuals From Drinking Water Removal Processes*. EPA/600/R-01/033. Cincinnati, Ohio: USEPA.

USEPA (U.S. Environmental Protection Agency). 2004. *Implementation Guidelines for the Filter Backwash Recycling Rule (FBRR)*. Office of Water. EPA 816-D-01-001. Washington, D.C.: USEPA.

CHAPTER 4
COAGULANT AND LIME SOFTENING RESIDUALS TREATMENT

The treatment of solid–liquid wastes produced in water treatment processes involves the separation of the water from the solid constituents to the degree necessary for the selected disposal or beneficial use method. Therefore, the required degree of treatment is a direct function of the ultimate disposal or beneficial use method.

Water treatment sludges from a chemical coagulation process typically have a 0.5 to 2.0 percent solids concentration. These solids are difficult to thicken by gravity to greater than a 3 to 4 percent solids concentration. Sludges resulting from lime softening can be removed from settling basins at solids concentrations as high as 10 percent and may thicken by gravity to a 30 percent solids concentration.

There are several residuals treatment methodologies that have been practiced in the water industry. Figure 4.1 shows the most common residuals handling options available, listed by general categories of thickening, conditioning, dewatering, and disposal or utilization. In choosing a combination of possible treatment process trains, it is probably best to first identify the available disposal options and their requirements for a final cake solids concentration. Most landfill applications require a "handleable" residual, and this may therefore limit the type of dewatering devices that are acceptable. Methods and costs of transportation may affect the decision as to "how dry is dry enough." The criterion should not be to simply reach a given solids concentration but rather to reach a solids concentration of desired properties for the handling, transport, and disposal options available.

Table 4.1 shows a generalized range of results that have been obtained for final solids concentrations from different dewatering devices for coagulant and lime residuals.

PUMPS AND PIPING

The unthickened sludge coming from the sedimentation basin in a coagulation process is generally fairly dilute, ranging from 0.5 to 2 percent solids concentration for alum sludges. These sludges can be conveyed by gravity or siphoning from the sedimentation basin to a sludge pumping station. Pumping of unthickened sludge can usually be accomplished using centrifugal pumps. Sludges from lime clarifiers are much thicker and may need to be handled by positive-displacement pumps.

Residuals thickened in gravity thickeners can be expected to achieve a solids concentration of 3 to 4 percent for coagulant sludges and up to 30 percent for lime-softening sludge. These thickened residuals generally can be pumped by progressive cavity pumps or other types of positive-displacement pumps. Coagulant residuals in the 3 to 4 percent range can sometimes be pumped by centrifugal pumps; therefore, engineering judgment is needed as to which system is appropriate.

THICKENING

After removal from a clarifier or sedimentation basin, most water sludges can be further thickened in a gravity concentration tank. Thickening can be economically attractive in that it reduces the sludge volume and results in a more concentrated sludge for further treatment in the

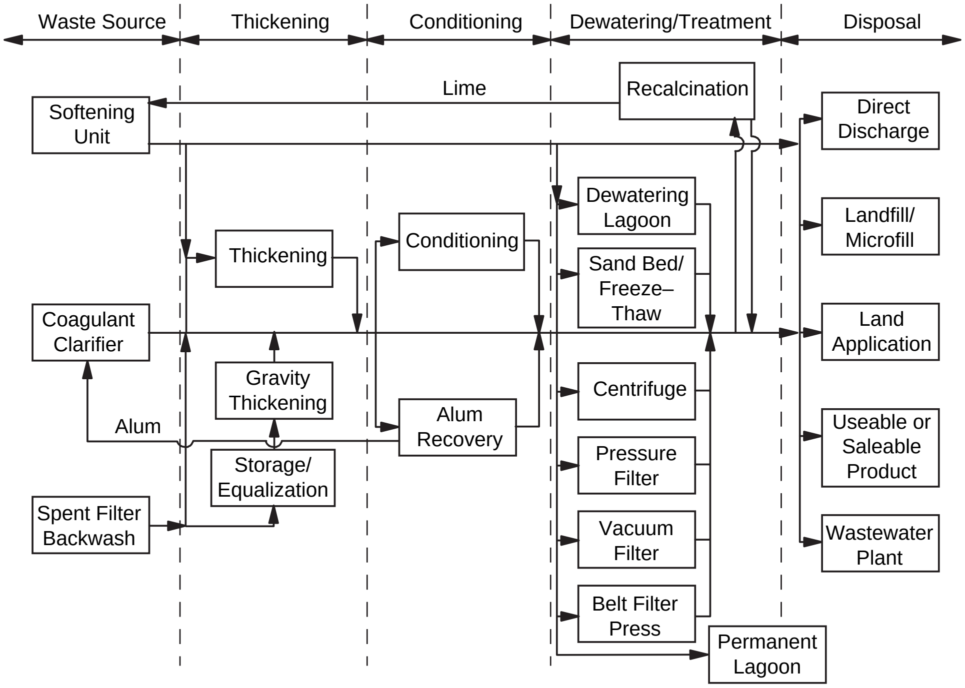


Figure 4.1 Sludge handling options

Table 4.1
Range of cake solid concentrations obtainable

	Percent solids concentration	
	Lime residual	Coagulant residual
Gravity thickening	15–30	3–4
Scroll centrifuge	55–65	18–25
Belt filter press	NE*	15–22
High solids belt press	NE	25–30
Vacuum filter	45–65	NA†
Pressure filter	55–70	25–45
Sand drying beds	50	20–25‡
Storage lagoons	50–60	7–15‡

*NE = not estimated.
†NA = not applicable.
‡Can be much higher if extended air drying is used.

dewatering process. Some dewatering systems will perform more efficiently with higher solids concentrations. Thickening tanks can also serve as EQ facilities to provide a uniform feed to the dewatering step.

Although there are a few types of thickeners available on the market, the water industry almost exclusively uses gravitational thickening.

Residuals thickening is performed primarily for reduction in the volume of residuals that will require subsequent treatment and disposal. The relationship between the volume of residuals and the solids concentration is expressed as

$$V = \frac{M}{r S_s P} \tag{4.1}$$

where V = volume of residuals (m^3)
M = mass of dry solids (kg)
r = density of water (10^3 kg/m^3, at 5°C)
S_s = specific gravity of the sludge–water mixture
P = percent solids expressed as a decimal (w/w)

An approximation for determining volume reduction based on percent solids is expressed as

$$\frac{V_2}{V_1} = \frac{P_1}{P_2} \text{ (approximate volume ratio)} \tag{4.2}$$

This is a quick and useful equation because the specific gravity of the residuals is not always known. Therefore, for thickening a 1 percent solids concentration sludge to 10 percent solids concentration, a volume reduction of approximately 90 percent is achieved.

Gravity residuals thickeners are generally circular settling basins with either a scraper mechanism in the bottom (see Figure 4.2) or equipped with sludge hoppers (Figure 4.3). They may be operated as continuous flow or as batch “fill-and-draw” thickeners. For continuous flow thickeners, the residuals normally enter the thickener near the center of the basin and are distributed radially. The settled water exits the thickener over a peripheral weir or trough and the thickened residuals are drawn off the basin. For tanks equipped with a scraper mechanism, the scraper is located at the thickener bottom and rotates slowly. This movement directs the residuals to the draw-off pipe near the bottom in the center of the basin. The slow rotation of the scraper mechanism also prevents bridging of the solids. The basin’s bottom is sloped to the center to facilitate collection of the thickened residuals.

Batch fill-and-draw thickening tanks are often equipped with bottom hoppers, as shown in Figure 4.3. Sludge flows into the tank, usually from a batch removal of sludge from the sedimentation basin, until the thickening tank is full. The sludge is allowed to quiescently settle, and a telescoping decant pipe is used to remove supernatant. The decant pipe may be continually lowered as the solids settle until the desired solids concentration is reached, otherwise the sludge will not thicken further. The thickened residuals are then pumped out of the bottom hoppers to further treatment or disposal.

Design of batch or continuous flow thickeners is usually accomplished based on previous experience of similar full-scale installations, laboratory settling tests, or pilot thickening studies.

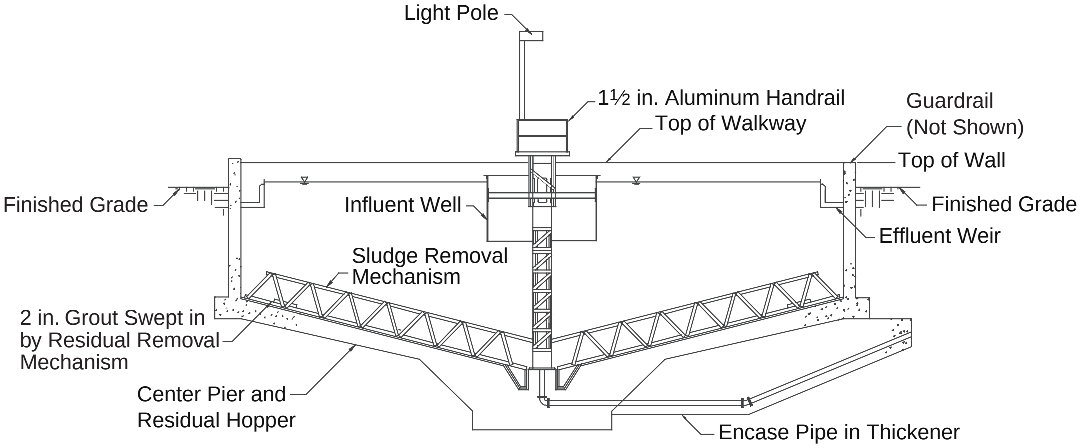


Figure 4.2 Continuous flow thickener

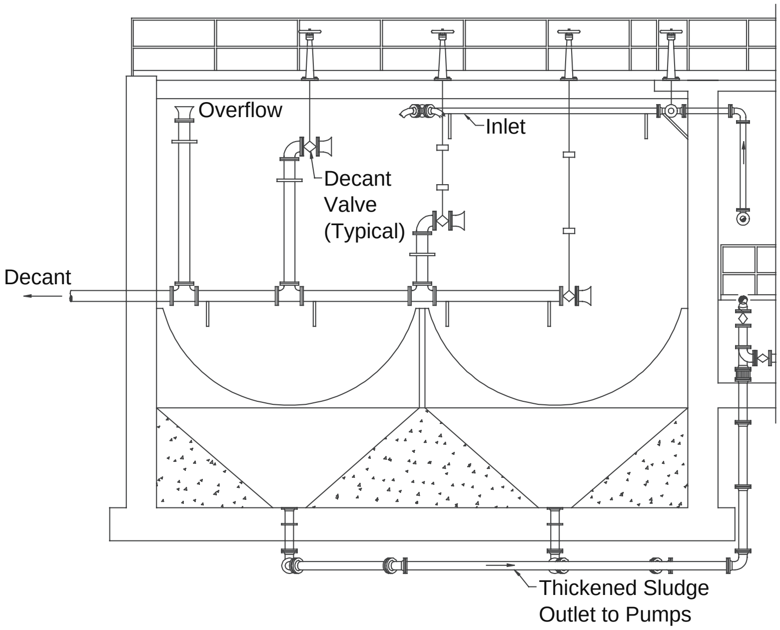
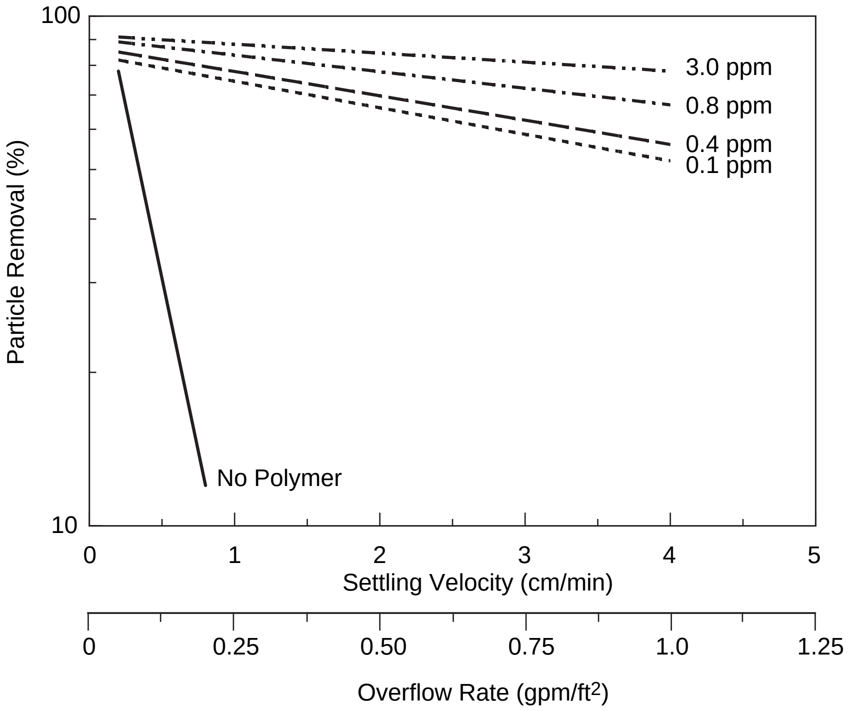


Figure 4.3 Batch-thickening tank schematic

Two different performance factors can affect the sizing of thickeners. If the supernatant from the thickener is discharged to a receiving stream, then generally there will be a suspended solids limitation placed on the discharge. If the supernatant is recycled, some utilities will desire to reach a certain quality in the stream prior to recycle. For both of these cases, the overflow rate of the thickener must be sized to meet the effluent quality requirements. Sizing of the thickener in this case is based on the hydraulic loading flow per thickener area and is expressed as gallons per minute per square foot (gpm/ft^2) or cubic meters per minute per square meter (m^3/min·m^2). The hydraulic loading is equivalent to the solids settling velocity, feet per minute (ft/min) or centimeters per minute (cm/min). The second performance factor of the thickener is the desired underflow



Source: Cornwell and Lee 1993.

Figure 4.4 Removal of *Giardia*-size particles (5–15 µm) from adsorption clarifier sample

solids concentration. In some cases, achieving a certain thickened solids concentration will set the thickener size. Sizing of the thickener in this case is based on the solids flux, sometimes referred to as the solids loading rate, and is expressed as mass per time per thickener area, pound per square foot-hour ($lb/ft^2{\cdot}hr$) or kilogram per square meter-hour ($kg/m^2{\cdot}hr$).

Plate settlers have found application in the United States for settling solids from SFBW and from clarifiers that discharge a residual with a low solids content such as contact clarifiers and some upflow blanket clarifiers. Some thickening of the solids can take place in the bottom storage area of the plate settlers if it is appropriately sized.

Laboratory evaluations to meet a given suspended solids or turbidity quality are often performed using standard jar test techniques. In this case the required overflow rate with and without polymers can be estimated.

Figure 4.4 shows an example of jar tests being used to achieve a certain percent of particle removal by thickening a clarifier residual. It shows that a very low overflow rate (<0.1 gpm/ft^2 [0.24 $m^3/hr{\cdot}m^2$]) was required to achieve significant removal without polymer addition. Figure 4.5 shows a similar test for achieving a specific effluent turbidity. Achieving effluent turbidities of <25 ntu required overflow rates of <0.1 gpm/ft^2 (0.24 $m^3/hr{\cdot}m^2$). Very often, discharge quality limitations will set the thickener size because of the low overflow rates required.

In order to determine the solids flux rate, larger tests than can be accomplished in the laboratory are required. Vesilind (1979) evaluated the effects of various cylinder diameters on the performance of pilot thickeners. At low suspended solids concentrations (<0.4 percent), the smaller cylinders (<1 ft [<0.3 m] diameter) tended to underestimate the settling velocity, which

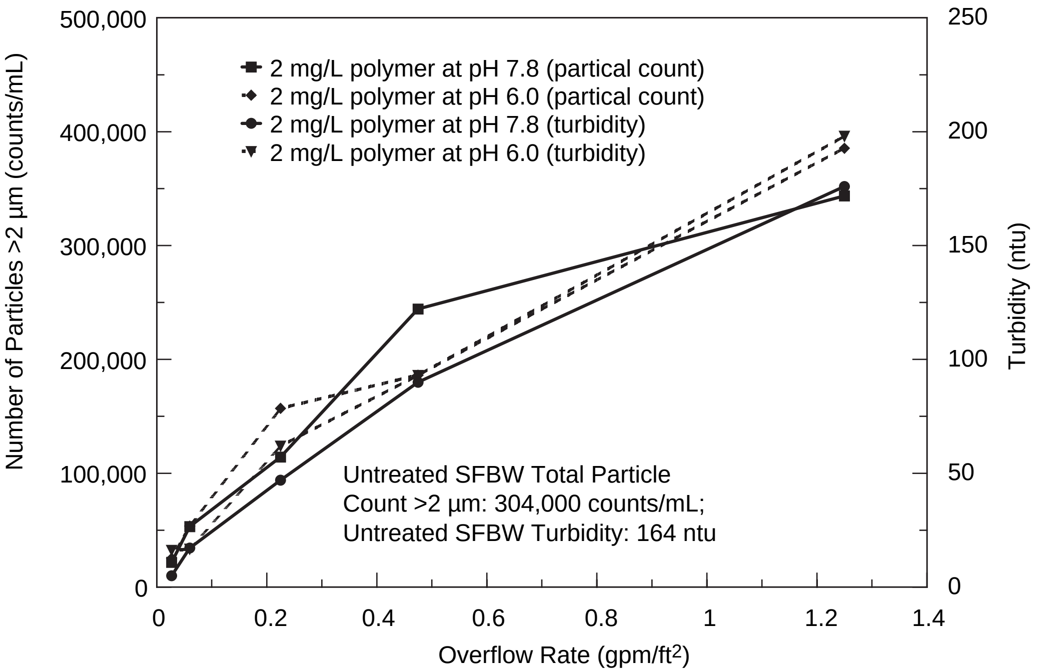


Figure 4.5 Settled water particles >2 µm and turbidity for SFBW using coagulation with pH adjustment

would result in a more conservative design. However, at suspended solids concentrations >0.5 percent, the smaller cylinders overestimated the settling velocity. Vesilind went on to recommend four considerations in conducting pilot thickening tests:

1. The cylinder diameter should be large as possible; 8 in. (20 cm) is a practical compromise.
2. The initial height should be the same as the prototype thickener depth. When this is not practical, 3 ft (0.9 m) should be considered minimum.
3. The cylinder should be filled from the bottom.
4. The sample should be stirred throughout the test, but very slowly; 0.5 rpm is a reasonable speed for an 8-in. (20-cm) cylinder. This slow stirring will help the test results of small cylinders better approach that of full scale.

Solids loading rates can be determined by using continuous flow studies or batch tests. For continuous flow studies, several different loading rates would be tested and solids concentrations in the solids underflow and supernatant determined. In each test it is important to reach a steady state solids depth and underflow solids concentration, which can take several days to achieve.

Vesilind (1979) has reviewed the various batch techniques available to determine flux rates that were summarized by Cornwell in *Water Quality and Treatment* (AWWA 1999). These techniques have generally not found use in the water treatment field. Continuous flow studies are more widely used to predict solids loading rates. Solids loading rates for coagulant residuals are reported in the 4-lb/ft^2•d range (20-kg/m^2•d) (American Society of Civil Engineers [ASCE],

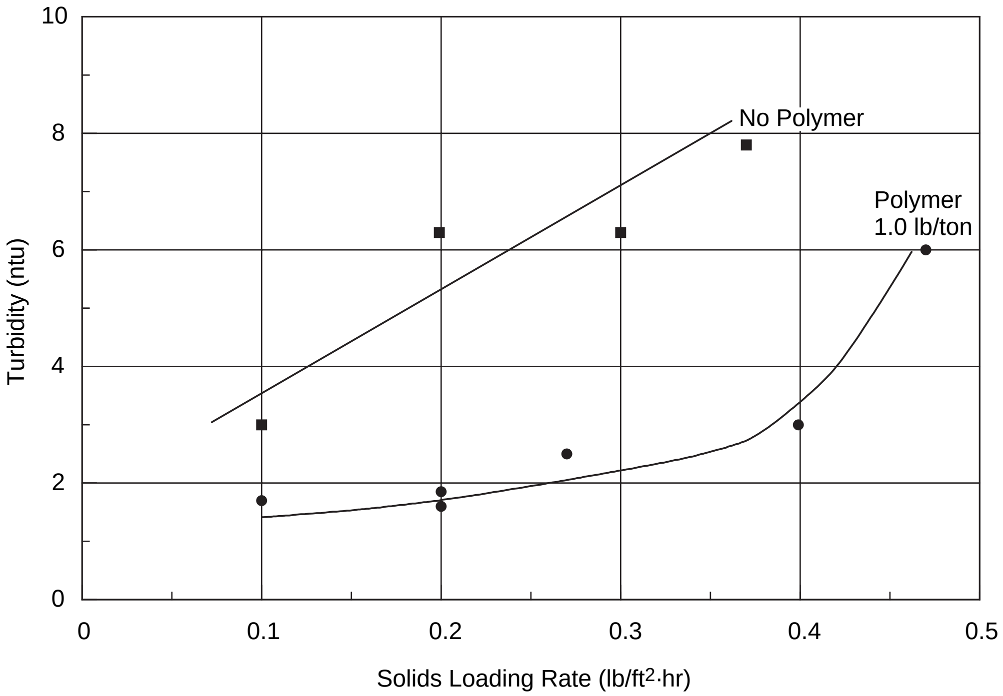


Figure 4.6 Pilot-scale thickening tests at the utility in Charlotte, N.C.

AWWA, and USEPA 1996). A hydraulic loading rate of 0.1 gpm/ft^2 (0.24 m^3/hr·m^2) for a sedimentation basin sludge at 0.1 percent solids concentration is equivalent to a solids loading rate of about 1.2 lb/ft^2·d (6 kg/m^2·d). If low hydraulic loading rates are used to reduce turbidity and particles in the supernatant, then the hydraulic loading rate will generally be limiting. However, as the hydraulic loading rate increases through the use of polymers, or other methods, then the solids flux needs to be evaluated.

The utility in Charlotte, N.C., conducted pilot thickening studies using an 8-ft (2.4-m) tall, 1-ft^2 (6.09-m^2) surface area settling column to treat an alum sludge from the sedimentation basins. Polymer was used at 1 lb/ton (0.5 g/kg) and compared to no polymer use; Figure 4.6 shows the testing results. The test objective was to achieve an overflow turbidity of <10 ntu and to determine the resulting underflow solids concentration. It was concluded that polymer conditioning would allow for a solids loading rate of 0.4 lb/ft^2·hr (2.0 kg/m^2·hr), and an approximately 2 percent underflow solids concentration was obtained. Again, note that overflow turbidity was the limiting design condition.

The City of Baltimore conducted similar tests also using 8-ft (2.4-m) tall columns of 1-ft^2 (0.09-m^2) surface area on an alum sludge. The results from those studies are shown in Table 4.2. Again, to achieve a supernatant turbidity of under 10 ntu, a solids loading rate of 0.4 lb/ft^2·hr (2 kg/m^2·hr) was acceptable with the use of polymer. Without polymer use, the lowest loading rate tested of about 0.2 lb/ft^2·hr (1 kg/m^2·hr) was too high. Note the relatively high polymer dose used in this test in order to achieve acceptable overflow turbidities.

Table 4.2
Alum sludge pilot thickening results*

Influent feed solids (mg/L)	Influent flow rate (gpm)	Solids loading rate (lb/ft^2•hr)	Polymer dosage (lb/ton)	Column A (unconditioned) thickened sludge (ppm)	Column B thickened sludge (ppm)	Column A supernatant (ntu)	Column B supernatant (ntu)
1,300	0.36	0.23	15.4	1,800	4,700	27	8
				7,500	7,300	56	9.6
3,100	0.2	0.3	12.5	2,800	4,100	30.4	9.5
				5,500	19,000	15.6	9.45
2,500	0.3	0.4	14	5,000	11,000	157	8.9
				3,700	10,000	198	7.2
				6,800	10,000	16.4	9.04
				3,600	14,000	32	7.5
				4,000	18,000	43	7.2
				8,400	20,000	56	6.8
2,800	0.6	0.8–0.9	16.8	10,000	9,500	40.7	17
				9,500	9,100	255	11.2
						260	15
3,100	0.8	1.3	18.8	5,400	5,900	230	14.6
				4,700	3,500	470	10
						480	260
						227	220

NOTES: gpm × 0.06309 = L/sec; lb/ft^2•hr × 4.8824 = kg/m^2•hr; lb/ton × 0.5 = g/kg.
*Study results were obtained from locations in Baltimore, Md.

MECHANICAL DEWATERING

Conditioning

Water residuals conditioning refers to the variety of chemical and physical techniques for altering residual characteristics to make subsequent removal of water more efficient. There is no clear-cut, accepted conditioning method practiced for a given type of sludge. A conditioning agent that works well at one plant may not work at a similar plant. Residuals properties used for evaluating the effectiveness of conditioning agents include SR, TTF, and CST (as discussed in chapter 2).

Conditioning of water plant residuals is generally only applicable to coagulant residuals and SFBW. Residuals from lime-softening clarifiers are more easily dewatered, and conditioning agents are seldom used. With hydroxide residuals, conditioning agents are needed to assist in the water removal processes or may be used to affect compressibility and minimize media clogging, such as in filter press operation. When conditioning is used for water–solids separation, polymers are generally the agent of choice. When the objective of conditioning is to prevent media clogging, lime has traditionally been utilized although recently polymers have successfully been used for this purpose.

Polymers vary in structural composition, molecular weight, and charge density. For most cationic polymers the charge density is near 100 percent, and the molecular weight of cationic polymers is generally less than anionic or nonionic polymers. Anionic polymers will vary in both charge density and molecular weight. Nonionic polymers have no charge density but have high molecular weights. As a broad generalization, for hydroxide sludge conditioning, the higher the molecular weight of the polymer, coupled with a long carbon chain length, the lower the dose required for conditioning. Molecular weight may even be more important than the charge type or density.

Polymer addition has been useful and, in fact, almost required for dewatering coagulant residuals by either nonmechanical methods such as sand drying beds or mechanical methods such as centrifuges, belt filter presses, and pressure filters. It appears that the primary mechanism is one of interparticle bridging such that the polymers form a porous matrix that permits water decant or drainage. It has been postulated that the polymer does not alter the chemical structure of the hydroxide particles themselves.

Particular use of chemical agents is discussed within the specific dewatering applications in this chapter. However, some general comments are in order. When first selecting a polymer type, a series of screening tests is required. It is possible to visually screen several polymers by simply adding increasing doses to small beakers of sludge and viewing the floc. While ideal doses do not look the same for different polymers and sludges, this procedure does allow rapid screening of different polymer types. To determine comparative doses and quantitative results, either the CST test or the TTF test is recommended.

Pilot Tests

The best way to develop design data for residuals dewatering systems is to conduct onsite pilot tests. As applicable in each of the following treatment sections, specific pilot methods are discussed. In this section, an overview is presented of the considerations that must go into a pilot study for water plant residuals treatment.

There are certain decisions that must be made prior to determining whether a pilot study is needed. The first steps are to ask what kind of residual is it, how much residual is there, and what are the available treatment options. Most of these steps could be pilot tested, although some are difficult to test on a small scale. One of the inherent difficulties in testing sludge options is that the pilot facility may require several other treatment steps in order to pilot one option. For example, a pressure filtration pilot plant may require that thickening and conditioning facilities be built on a pilot scale also. Land application of dried lime residuals cannot be tested without dewatering first, and so forth.

There are four reasons for conducting a pilot study. Studies should be considered if (1) the technical feasibility is unknown; (2) the economic feasibility is unknown; (3) there may be a less expensive alternative available; or (4) some process refinement or redevelopment of design data is needed. The technical feasibility of a process may be unknown if the method has not been utilized on large-scale applications or on wastes that are similar in nature to this particular type of water plant residual. In this case, the pilot emphasis would be on several pertinent factors, such as

- Does it meet the treatment objectives?
- What variables affect performance?
- How much will it cost?
- What are the design and scale-up factors?

A technical feasibility pilot study can be quite involved and requires both research and engineering approaches.

The economic feasibility of a process may be unknown if it is technically proven elsewhere, but the factors affecting cost for the particular application are not well defined. The emphasis of this type of study would be to vary as many factors as possible and to secure good operating cost data, such as operating time, chemicals, and maintenance requirements. Variations in the residuals feed concentration or characteristics may also be tested to determine the effect on cost. Multiple processes may be evaluated to find the most economical alternative.

If it is known that a given alternative will work but several options within the alternative are available (e.g., different manufacturers) then a side-by-side study may be needed. Comparison studies require that as many variables as possible be held constant except for the process or equipment differences. The residuals feed to each process should be the same and operating variables matched so that a true difference in performance can be evaluated.

Process refinement studies can be justified if either the design parameters need definition in order to ensure proper performance or parameters affecting cost for final process evaluations are needed. These studies can also be very useful in optimization of an existing treatment process.

By assessing the alternatives according to the guidelines previously discussed, a decision can be made as to whether a pilot study is warranted. Assuming the answer is affirmative, the size of the study must be determined. The choices are to conduct bench-scale studies and/or onsite, larger-scale pilot studies.

The terms *bench* and *pilot studies* have the implied meaning of size. Although size is generally associated with the difference between bench and pilot studies of mechanical dewatering equipment, there is a more important distinction. Pilot studies are of a size sufficient to produce reliable data for the design and costing of a full-scale installation, whereas bench studies do not provide that information. Bench studies have the advantages of allowing quick screening of various alternatives and testing of several variables on performance at relatively low costs. They do not provide reliable scale-up data and only offer limited design and cost information. Examples of very useful bench studies include specific resistivity tests, filter leaf tests, CST tests, manufacturer dewatering equipment evaluations, and conditioning studies.

In most cases, the largest costs associated with pilot tests are labor and analytical testing. Therefore, sufficient funds should be appropriated for the labor and lab analysis necessary to gain the required data. Pilot studies often require large volumes of residual feed material that may need special handling. Several weeks may be necessary to evaluate many variables. Examples of pilot studies for residuals treatment would include nonmechanical drying operations, such as a drying bed, manufacturer tests of mechanical drying equipment, and specialized studies such as coagulant recovery.

If the difference between a bench and pilot study is one of size required to provide design data, then the question of “how big is big enough” must be addressed. In some cases, a small lab system may be large enough, while in other cases, even a major project would not answer all the design questions. Each type of process must be addressed on its own merit. As an example, in exploring tests of dewatering equipment, each manufacturer should be consulted as to the type of equipment available for testing, the costs of rental for each size, and their scale-up factors. Some pilot dewatering facilities are production units and the manufacturer can use a 1:1 scale-up factor, whereas other units may need scale-up factors of up to 2.5:1. Still other units may not have any scale-up factors because they are not prototype units. The utility should be aware of these differences as well as the potential impact on full-scale costs when choosing pilot equipment.

One of the difficulties in running bench or pilot studies on water plant sludges is obtaining representative sludge samples. Obviously, if the sample tested does not “represent” the sludge to ultimately be treated or does not reflect seasonal variations, then the pilot study may not provide reliable results. The use of the methods presented can help assess if the sludge sample is representative. By knowing the alum dose, the raw water suspended solids concentration, and the solids concentration of the sludge sample, it is therefore possible to estimate the aluminum content of the sludge. Measurement of the aluminum can then be used to check the sludge sample. The problem of obtaining a good sample is generally more acute with manually cleaned basins than with continuous withdrawal. With manually cleaned basins, the sludge will often stratify with heavier solids on the bottom and lighter solids (with a higher aluminum hydroxide content) on the top. Similarly, for softening plants, the calcium carbonate may be on the bottom and the magnesium hydroxide on the top. With a manual basin, small samples can be collected by sampling at several depths and then blending the samples. For a pilot study, it is probably best to collect a uniform sample in an external holding tank.

If the full-scale installation is to include sludge thickening, then provisions must be made to test various solids concentrations during the study. This can be accomplished by a batch fill-and-draw operation if other methods are not available. Of course, it would also be a good idea to collect information for the design of the thickener itself.

During the test, it is easy to overlook important items. Preplanning, creating good record logs, and establishing a system of checkpoints is paramount to successful studies. The testing program should be divided into at least two phases. First, the complete program is established, starting with formulating the study objectives and desired results followed by identification of the key variables and tests required for data development. A fairly rigid schedule is set for the first half of the study where as many variables as possible are evaluated, one at a time. Although data should be carefully monitored daily, a complete data analysis should take place at the halfway point. The data are then reduced to graphs and models as appropriate, and attempts are made to draw the desired conclusions. The course of the testing program is often altered by this review process. In the situation where expensive equipment is being rented on a weekly basis, the review process may have to be scheduled during a weekend. The time necessary for stopping and assessing the data is an integral part of any pilot program.

Also, during the testing program, the engineering and water plant personnel should keep good records and observe what is going on in other parts of the water plant. It is very useful to keep a running log in addition to any specific data sheets. The log will provide a sequential listing of operating conditions, unusual conditions, observations, starts and stops, and so forth. Many entries may seem unnecessary, but no matter how detailed the information, there will always be some gaps in the data or unexplained periods in operation.

The final task of any pilot study is interpretation of the results. A key aspect is to compare the results actually received to any projections that may be made as to what could have been accomplished. It is easy to say “we only got 15 percent solids *but* if we would have done something differently, we could have gotten 25 percent.” That may be true, but be careful and assign risks to any projections used in cost comparisons.

In summary, pilot studies of residuals can be valuable and are often required in the predesign phase of the planning process. Their use should follow as much prescreening as possible to eliminate nonviable alternatives. When pilot studies are used, proper definition of data needs is necessary, and if they are to provide design data, then proper sizing is very important. Properly designed and operated pilot programs can result in significant cost and performance savings to the utility.

Table 4.3
Bench-scale test results of mechanical dewatering*

Parameter	Centrifuge	Belt filter press	Diaphragm filter press
Loading rate		700 lb/m•hr	0.35 lb/ft^2•hr
Polymer dose (lb/ton)†	1.6–2.8	2.0	0
Feed solids (%)	3.5	2.0	2.0
Cake solids (%)	15–20	17	25

*Study results were obtained from locations in Charlotte, N.C.
†lb/ton × 0.5 = g/kg.

The utility in Charlotte, N.C., conducted both bench and pilot dewatering studies. Bench-scale tests were performed by manufacturers of centrifuge, belt filter press, and diaphragm filter press equipment. For the centrifuge and belt filter press, the objectives were (1) to determine whether or not these dewatering technologies could achieve a dewatered residuals solids concentration of 20 percent with full-scale equipment and (2) to screen various polymers and polymer doses before testing onsite with full-scale equipment. The diaphragm filter press bench-scale tests were conducted to generate sufficient data for evaluating the technology, sizing the equipment, and costing of full-scale facilities.

Table 4.3 summarizes the bench-scale test results, which confirm that all three technologies were able to dewater the residuals. The centrifuge and belt filter press achieved 15 to 20 percent dewatered residuals solids (cake), and the diaphragm filter press achieved 25 percent solids concentration. Bench-scale dewatering tests are typically less efficient than full-scale dewatering operations, and, therefore, full-scale tests should achieve higher solids concentration.

Full-scale pilot tests with the centrifuge and belt filter press were conducted. Before testing each machine, performance criteria were established for the manufacturers as a basis to determine whether the process could operate successfully and provide the desired results. Each manufacturer was allowed to adjust variables as necessary to meet the minimum performance criteria, which included

- Cake solids concentration
- Solids recovery or capture level
- Effluent quality

The cake solids concentration for the tests was established at a minimum of 20 percent cake solids, although it was noted that higher solids concentrations were desirable if they could be achieved in an efficient manner. The 20 percent cake solids concentration was based on the residuals handleability, minimization of residuals volumes, and requirements for final end-use options. A solids recovery or capture level of at least 95 percent was required to ensure adequate equipment efficiency.

Variables, such as polymer dose, loading rate, and equipment speeds, were developed by each manufacturer to provide optimum machine performance.

Before onsite testing, fresh sedimentation basin residuals were collected over a period of time in a lagoon. The residuals from the sedimentation basins were collected in the lagoon and

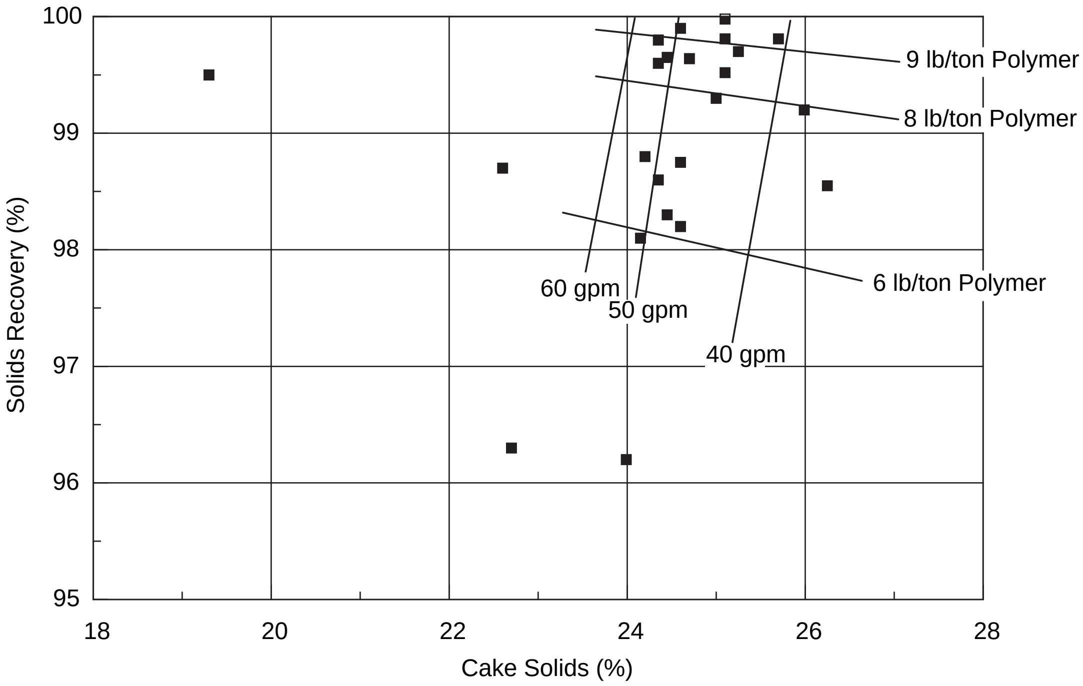


Figure 4.7 Pilot centrifuge test results at hydraulic feed rates and polymer doses shown

allowed to settle, and decant water was drawn off periodically. Once an adequate volume of residuals was collected in the lagoon, further collection of residuals was discontinued. Lagoon mixers were used to mix the collected residuals, to eliminate any settling of residuals, and to provide a uniform, consistent residuals solids concentration for testing. A submersible pump was placed at the bottom of the lagoon for transferring residuals to each pilot dewatering unit.

Equipment operating data were collected during the testing period. Information about the centrifuge included bowl differential speed, torque, and residuals and polymer feed rates and concentrations. Parameters for the belt filter press included belt speed, belt tension, wash-water flow rate, belt type and residuals, and polymer feed rates and concentrations. The manufacturers were allowed to screen and select polymers, with the stipulation that the polymers must be approved for use in potable water.

A useful way of presenting pilot centrifuge data is shown in Figure 4.7. A general relationship between cake solids concentration, solids recovery, polymer dose, and hydraulic feed rate is shown. Most data collected during the pilot test showed 24 to 26 percent cake solids concentrations at a corresponding 98 to 99 percent solids recovery. The 98 percent solids recovery was achieved with polymer doses of up to 6 lb/DT (3 g/kg), while higher solids recoveries were achieved by increasing the polymer dose to 9 lb/DT (4.5 g/kg). Note that the cake concentration was higher than the bench predictions, but the polymer dose was also higher.

Testing of a belt filter press was also conducted. Feed solids concentrations during the entire belt filter press test period ranged from 0.59 to 1.17 percent, with an average of 0.74 percent.

Figure 4.8 shows the relationship between the cake solids concentration and various solids loading rates during the testing period. The solids loading rate range of 100 to 350 lb/m·hr (45 to

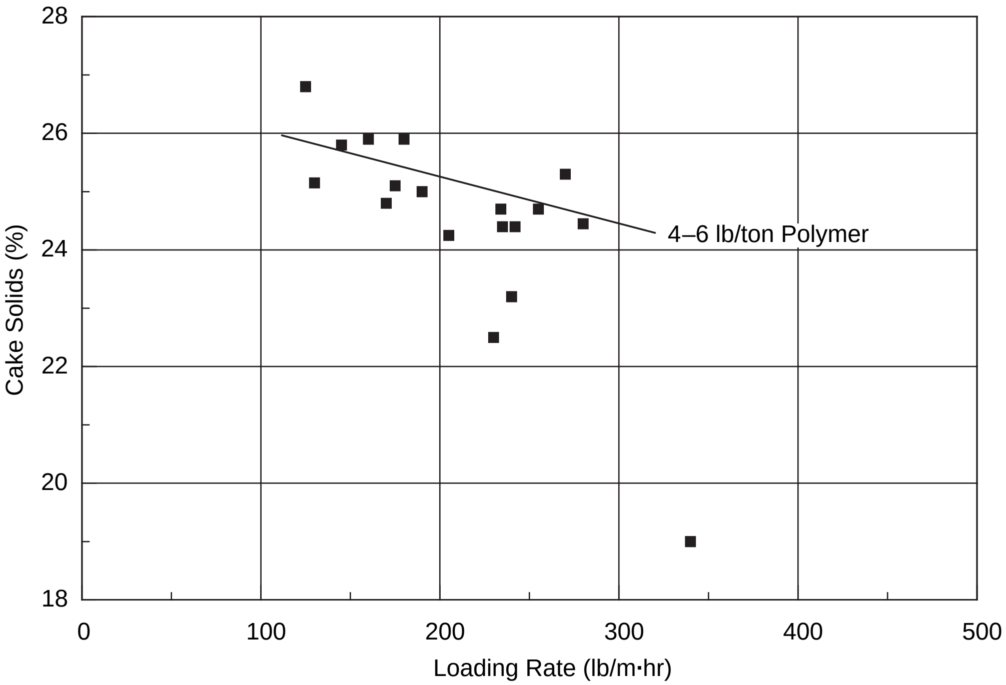


Figure 4.8 Belt filter press pilot results obtained at the Charlotte, N.C., utility

156 kg/m·hr) was achieved, with an average feed solids concentration of 0.74 percent. The cake solids concentrations were in excess of 24 percent, with a corresponding polymer dose of 4 to 6 lb/ton (2 to 3.9 g/kg). Of significance is that the pilot test results for the design loading rate were about half that found in the bench scale.

Centrifuges

Description

Centrifugation of residuals is basically a shallow-depth settling process enhanced by applying centrifugal force. The basic physical principle of centrifugal force is that a moving body tends to continue in the same direction; if that body is forced to change directions, it resists the change and exerts a force against whatever is resisting it. In the case of centrifugal force, the force applied by the body is radially outward from the axis of rotation.

Centrifugation enhances settlement of the solids. In conventional settling tanks, the solids are acted on by the force of acceleration due to gravity, g. In centrifugation, the applied force is rw^2, where r is the distance of the particle from the axis of rotation and w is the rotational speed. In modern centrifuges rw^2 may be 1,500 to 4,000 times the value of g.

The comparison of rw^2 to g has led to efforts by many to develop equations for centrifugation by substituting rw^2 for g. However, this substitution relates to discrete particles only and does not account for hindered settling and the effect of scrolling (moving the solids out of the bowl). These deficiencies in the theory of centrifugation limit the use of sedimentation as a basis for the design of centrifuges, and the evaluation and design needs to be based on pilot studies.

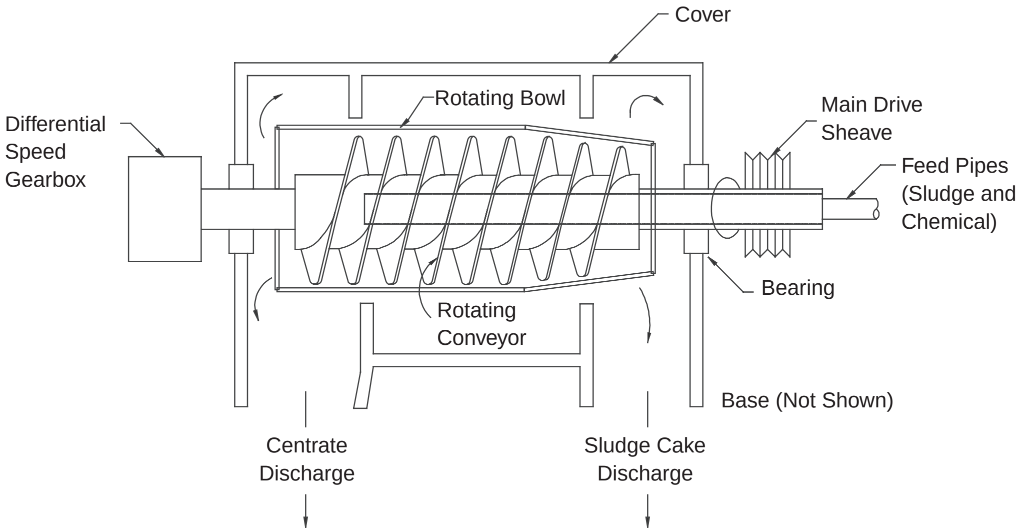


Figure 4.9 Example centrifuge

Laboratory-bottle centrifuge tests and small-scale centrifuge tests have been used with some success. However, scale-up from these small-scale tests have been best based on the development of empirical relationships and judgment. These procedures should only be used for initial process screening studies.

The major type of centrifuge used for the dewatering of water plant sludge is the scroll-discharge, solid bowl decanter. The solid bowl centrifuge (also called scroll or decanter centrifuge) is a horizontal unit that utilizes a scroll conveyor inside the centrifuge bowl (see Figure 4.9). The unit is fed continuously with the solids settling against the bowl wall. The scroll rotates at a slightly different speed than the bowl and conveys the dewatered residuals to the small end of the centrifuge where it is discharged. The water is directed from the central axis of the centrifuge toward the centrifuge's large end where it is discharged. The water exits through adjustable weirs (level rings), which also control the pool depth.

The best procedure for evaluation of centrifuges is conducting pilot tests on prototype equipment. Tests should be conducted on a centrifuge exactly like that to be used in full scale, except smaller. Tests should be conducted for operational parameters of concern such as feed flow rate, feed suspended solids concentration, and polymer conditioning, bowl speed, pool depth, and scroll speed.

The best indicators of performance are the cake solids concentration and the centrate suspended solids concentration. It is preferred to obtain a pilot machine with a variable-speed-drive motor to alter the bowl speed and a machine where the pool depth can be easily changed. This will allow evaluation of machine variables as well as residuals characteristics.

Having found the optimal conditions for operation on a pilot machine, the problem of scale-up to production units still remains. While this process is often left to the manufacturer, it is to the utilities' and the engineers' advantage to remain closely involved in the scale-up considerations. Many times scale-up will be done strictly assuming that liquid loading is the limiting parameter. However, with water plant residuals, solids loading can often be limiting. When using only liquid loading parameters, full-scale units could be installed only to find their actual capacity is less than anticipated.

Scale-up considering liquid loading is often referred to as the sigma concept. The sigma concept is based on Stoke’s law describing the settling of discrete particles under the influence of gravity. Gravity is replaced by the centrifugal acceleration, and the expression is integrated over the depth of the water pool. One then ends up with a term for the allowable flow through the centrifuge:

$$Q = \left(\frac{Vw^2}{g \ln \frac{r_2}{r_1}} \right) \frac{g(p_s - p)d^2}{18\mu} \tag{4.3}$$

where V = volume of sludge/water in the pool
w = radial velocity of centrifuge (radians/sec)
r_2 = radius from centerline of centrifuge to bowl
r_1 = radius from centerline of centrifuge to pool level
p_s = particle density
p = fluid density
d = particle diameter
μ = viscosity

Note that the left-hand term is made up of machine variables and the right-hand term is residual variables. Therefore, in scale-up, if it is assumed that the residuals are the same for full scale as in the pilot studies:

$$Q_2 = \frac{Q_1 \Sigma_2}{\Sigma_1} \tag{4.4}$$

where Q_2 is the allowable flow in the full-scale centrifuge based on the optimal flow (Q_1) obtained in the pilot plant, and

$$\Sigma = \frac{Vw^2}{g \ln \frac{r_2}{r_1}} \tag{4.5}$$

which are variables obtainable for the particular size pilot- and full-scale centrifuge. Therefore, it is straightforward to predict the full-scale flow equivalent to the pilot-scale flow by a ratio of the two sets of machine variables that are readily obtained from the manufacturer.

An analysis of the solids loading limitation is known as the beta concept and is expressed as

$$Q_{s2} = \frac{Q_{s1} \beta_2}{\beta_1} \tag{4.6}$$

where Q_s is the solids throughput in units such as kilograms per hour and

$$\beta = \Delta WSNDZ\pi \tag{4.7}$$

where ΔW = bowl/conveyor differential speed
S = pitch of blades
N = number of leads
D = total bowl diameter
Z = pool depth

Again, all beta terms are made up of machine variables and set by the manufacturer for the unit of interest.

In scale-up the limiting conditioning should be calculated using both the sigma and beta concepts.

Design

The centrifugation system includes the centrifuge and for hydroxide residuals, a residuals conditioning method.

An advantage to the centrifuge when compared to other dewatering methods is the small space requirement. A large centrifuge (200 to 700 gpm [13 to 44 L/sec]) may require an area of 400 ft^2 (37 m^2). Additional space is required near the centrifuge for the following:

- Polymer storage, mixing tank, and pumps
- Sludge feed pumps and piping
- Overhead hoist
- Proper operational and maintenance space

Centrifuges are often located on upper floors of the sludge building so that the cake may be discharged into trucks or hoppers below.

Foundations for centrifuges require special attention in order to support the heavy machine. The cost of the sludge building structure is usually increased when locating the centrifuge on the upper floors. Vibration from the machine may require special treatment.

Operation Considerations

Operating considerations for centrifuges include feed flow rate, rotational speed, differential speed of the scroll, depth of settling zone, and the controlled properties of the residuals (e.g., temperature, percent solids, etc.). To a point, the higher the rotational speed used and the resulting “g” force applied, the dryer the cake that can be obtained from the centrifuge. However, the wear and tear on the centrifuge is proportional to the cube of the bowl speed and thus precludes the use of excessively high “g” forces.

The reported cake dryness from solid bowl centrifuges varies depending on the many operating considerations. The feed rate has been seen to be a primary operational gauge of the ultimate performance. The best performance data have been obtained at 75 to 80 percent of the hydraulic or solids capacity of the centrifuge. Lower polymer dosages are required and a dryer cake is achieved at this slightly lower than capacity rate.

Another operational consideration is the solids concentration of the residuals feed. It has been demonstrated that for a particular set of centrifuge operational constraints, a well-controlled feed concentration will produce consistently better results than a varying concentration. For centrifuges, it is usually advantageous to vary the hydraulic feed rate and to hold constant the solids loading rate for incoming residuals that have changing percent solids concentrations. This calls for an EQ and thickening facility prior to the centrifuge itself.

If the centrifuge is not to be used for any significant time period (24 hours or more), the inside of the bowl needs to be washed down with significant quantities of water. If not washed, the solids remaining in the bowl will dry and possibly cause unbalanced operation.

In June 1994, Newport News Department of Public Utilities (Waterworks) stopped discharging wastes from its two WTPs into local rivers that flow into the Chesapeake Bay and began dewatering the residuals with centrifuges for land application on pine plantations owned by the Waterworks (Harbin 1995).

The equipment used by the Waterworks for dewatering residuals includes two centrifuges, two centrifuge feed pumps, a polymer makeup system, two polymer feed pumps, and a microwave solids analyzer. The Waterworks also has two large progressing cavity pumps (250 gpm [15 L/sec]) for transferring thickened residuals from one treatment plant through an 8-mi (13-km) pipeline to the other treatment plant where the centrifuges are located.

The centrifuges have a rated capacity of 200 gpm (1.2 L/sec) each with feed solids between 0.5 and 2.0 percent. The bowl speed is 2,600 rpm, which generates a centrifugal force that is more than 2,700 times the force of gravity. The direct current (DC) backdrive system provides several advantages over other systems. It has the ability to operate in the same direction or reverse, giving a broader range of differential speed variation as well as regenerative breaking and monitoring functions. The DC backdrive works as a DC generator and recycles some power from the rotation of the pinion shaft, generating significant power savings over time. The DC backdrive can also be used for cleaning the bowl when it is stationary, by turning the conveyor. On sensing high torque (90 percent), the speed control sends a signal to shut down the centrifuge feed and polymer pumps. At high torque (100 percent), the control will also shut down the centrifuge and increase the conveyer differential speed to a maximum to scroll the excess solids from the centrifuge as it decreases in speed. The advanced backdrive controller allows the operation of the centrifuges in several different automatic modes.

The centrifuge feed pumps are rotary lobe type with variable-speed hydraulic drives that have a maximum rated capacity of 250 gpm (16 L/sec). The polymer makeup system is a 1,000-gal (3.8-m^3) mix tank and 1,400-gal (5.3-m^3) feed tank. The polymer feed pumps are positive-displacement diaphragm-type pumps. The microwave solids analyzer is used to check the feed solids content hourly for solids handling records and fine tuning of polymer dosage. The microwave solids analyzer is also used for periodic checking of cake solids to evaluate centrifuge performance. The microwave solids analyzer gives reliable results in only 5 to 10 minutes.

The mode of operation selected by the Waterworks personnel was a percent torque setting, which allows the operator to select a percent torque setting whereby the controller automatically adjusts the differential speed setting to maintain the torque setting through fluctuations in feed solid concentrations and polymer dosage changes. This feature allows for a more constant cake solids content during operation. The centrifuges are also equipped with several other safety devices and/or monitoring devices that will send information, signal alarms, and/or shut down equipment.

The optimum pond depth in the Waterworks centrifuges was determined during startup and performance testing. This was done by setting adjustable plate dams located in the front hub of the bowl assembly. Increasing pond depth increases retention time in the centrifuge and favors better clarification of the centrate. A high pond depth can cause an unacceptable wet cake to be discharged from the centrifuge. A low pond depth favors a dryer cake but can reduce centrate clarity. A shallow pond depth to optimize cake dryness while averaging 97 percent solids recovery was chosen. Operationally, the shallow pond depth has also helped to speed up startup and shutdown of the centrifuge by maintaining a lower level of residuals in the bowl.

Residuals feed rate is usually determined by plant production requirements. Increasing the feed rate decreases the retention time in the centrifuge and may cause centrate clarity to decrease. The centrifuges have operated at or above capacity because of higher than expected residuals production by the plants. By running at capacity, higher polymer dosages are required to maintain centrifuge efficiency. An average of about 11.0 lb/DT (5.5 g/kg) polymer dosage is used.

The polymer solution feed rate is dependent on the percent solids of the centrifuge feed solution. This is checked and adjusted hourly based on feed solids results from the microwave solids analyzer and visual observations of the centrate.

An optimum torque setting of 70 percent was determined during startup based on cake dryness. To obtain a 17 percent dry solids concentration, the percent torque setting is 70 percent. Typically the torque setting is adjusted to 60 to 70 percent and polymer is adjusted as needed to achieve 17 percent cake solids.

Pressure Filter Press

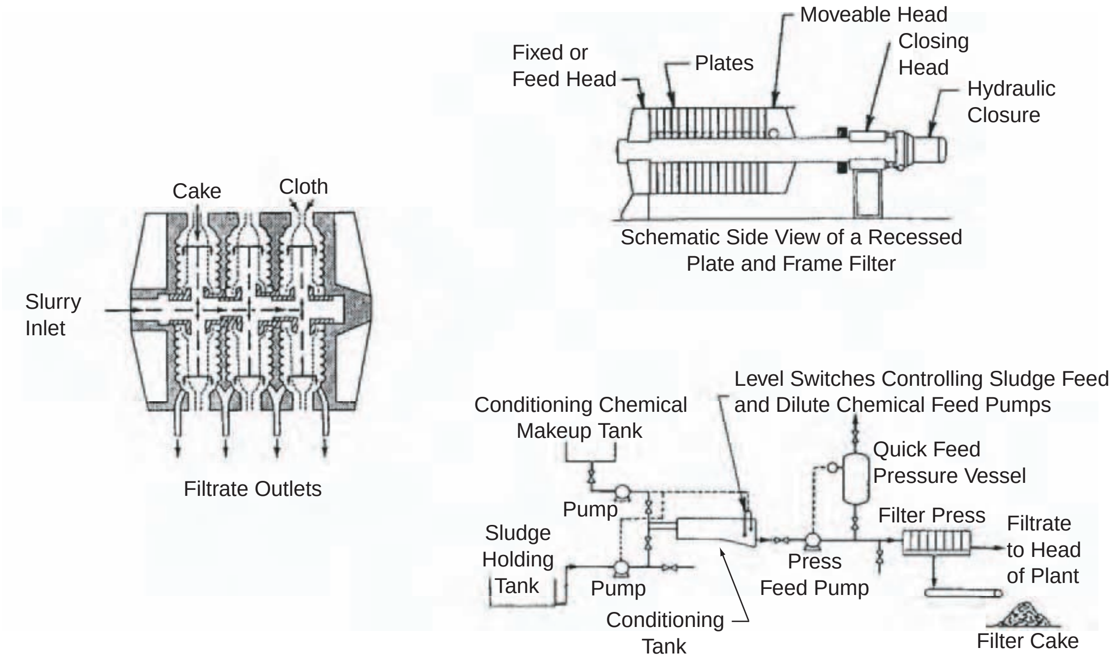
Description

The basic concept of pressure filtration is the separation of water from a liquid residuals slurry using a positive pressure differential as the driving force. The pressure filter press is also called a pressure filter, plate-and-frame press, recessed plate press, filter press, and (with modification) diaphragm filter press. The pressure filter contains a series of filter plates supported by and contained in a structured frame. The plates are designed such that when two adjacent plates are brought together, a compartment or chamber between the plates is formed to hold residuals. The plates are pushed tightly together, by hydraulic or electromechanical means, to make the compartment leakproof (Figure 4.10).

Lining the compartment is a cloth medium that is porous enough to retain the residuals solids while releasing the water in the residuals. While the residuals are being pumped into the compartments, the solids are retained and the water is released from the pressure filter press. The residuals pumping continues after the compartments are full (thus pressurizing the compartment) until the solids concentration of the cake in the compartment is at an optimum value. Then the pumping is ceased, the plates are separated, and the retained sludge cake is released by gravity for ultimate disposal.

The above describes one “cycle” of pressure filter press. At the end of one cycle, the plates can be automatically realigned for loading the next batch of residuals.

During a filtration cycle, the pressure filter press will experience variations in residuals flow rate and pressure. At the beginning of the cycle, the flow is at the maximum and pressure at



Source: Innocenti 1988.

Figure 4.10 Example filter press

the minimum. At the end of the cycle, the reverse is true, and the cycle is terminated when the peak design pressure and the low feed flow rate is reached.

The original pressure filter press was the plate-and-frame type as previously described. Improvements in the plate-and-frame press have included plate-shifting mechanisms and better cycle controls.

A variation of the plate-and-frame filter press is the diaphragm filter press. The diaphragm filter press utilizes a flexible membrane or diaphragm across the face of each recessed plate to squeeze the forming cake. The filtration process is the same as with the plate-and-frame press, except that after the press is filled with residuals and the cake is forming, the diaphragm begins to squeeze. Diaphragm filter presses usually produce thinner cakes and consequently have shorter cycle times than plate-and-frame filter presses.

As with other mechanical dewatering devices, it is very difficult to predict performance, design, or cost from theoretical equations or laboratory analysis. State of the art is such that pilot-plant testing is necessary. Pilot filter presses are generally available in two types: those that simulate full-scale performance and those that are, in reality, small-scale production units. Simulation units have plates that are generally less than 1 ft^2 (0.09 m^2) of filter area and contain only two to four plates. They can provide information on conditioning requirements, final cake solids concentrations, filtrate quality, and cycle time. They are obviously less expensive to use than prototype units because of lower residuals feed requirements, and lower shipping and rental fees. Manufacturers will often evaluate residuals shipped by the utility in their own lab using these units. Depending on the total funds to be spent by the utility, results from these simulation units may be satisfactory. Generally, however, the sums of money to be spent for the full-scale plant are sufficient to justify the cost of testing prototype units. In testing a prototype unit, the utility should ensure that the equipment is essentially the same as the full-scale unit. This includes such items as pressurization mechanisms (pump types, etc.), degree of automation, cleaning and wash methods, and cake release methods. Beyond the mechanical and operational similarities, the general items to consider in pilot-plant studies discussed earlier should also apply.

If a production unit has been pilot tested, then scale-up is fairly straightforward. The final results of pilot testing will be total solids to be treated (this is solids produced via water treatment plus solids added during conditioning, which can be as much as 30 percent); cake thickness (and therefore plate chamber volume required); and cycle time for one batch press operation. Scale-up then is a matter of calculating the total plate volume requirements and picking the number and dimensions of appropriate units.

Design

As previously discussed, the capacity of pressure filter presses is determined by the number of plates, the size of the plates, and the thickness of the cake.

Plate-and-frame press manufacturers have three standardized chamber thicknesses: 25, 30, and 40 mm (0.98, 1.18, and 1.57 in.). Filtration tests determine the most economical cake thickness.

The plates are provided in many designs, sizes, shapes, and materials. Most plates are recessed and come in sizes from 0.5 to 2.0 m (2.0 to 7.0 ft). They can be circular, square, or rectangular. Plate materials include cast and ductile iron, coated steel, and polypropylene. Materials for filter plates should be evaluated for strength, cost, and mass affecting both handleability and structural support costs.

The number of plates in a filter press can affect performance. Poor distribution of sludge to remote chambers is possible when feeding well-conditioned sludge to a long press (more than 80 plates).

Plate-and-frame filter presses are available operating at medium (700 kPa or 100 psig) or high (1,600 kPa or 225 psig) filtration pressures. Higher pressure usually results in a thicker cake with a higher solids concentration. However, research has determined that when higher pressures are applied to compressible residuals, the solids become tightly packed, which increases resistance and decreases filtration flow. Proper residuals conditioning is usually required to alter the compressible residuals characteristics.

The filter cloth lining each plate is almost exclusively of the monofilament type (i.e., woven from a single filament rather than twisted together of many fibers, as with multifilament). The multifilament cloth has been found to blind in some applications because of swelling of the yarn. Filter media are rated by particle retention and permeability. Additional factors in the selection of filter cloth media are durability, cake release, blinding, and chemical resistance. Air permeability is a measure of the openness of the filter cloth, determined by air flow through a unit area of media at a given pressure drop (e.g., 800 $L/m^2 \cdot hr$ at 0.1 kPa [100 cfm/ft^2 at 0.5 in.] H_2O). Air permeability is a measure of comparison between filter cloths. However, it doesn’t represent the actual working permeability, which is affected by strand swelling, solids impregnation, and weave distortion. Also, it has been determined that early in the cake formation stage the cake itself acts as the filter and the filter media permeability is less important. Low permeability media (<250 $L/m^2 \cdot hr$) are generally utilized on residuals that are relatively easy to dewater. While low-permeability media yield high solids capture values, a greater tendency for media blinding, poor cake release, and difficult cleaning would be expected. High-permeability media (>1,500 $L/m^2 \cdot hr$)

are generally utilized on difficult-to-dewater residuals. High solids retention during early cake formation is forfeited for improved cake release and less media blinding.

The pumps feeding the residuals to the pressure filter press must be capable of operating under widely varying conditions. At the beginning of the filling cycle, there is very little discharge pressure head, but there is substantial flow. Near the end of the filling cycle when the chamber is full and the residuals are being compressed, the pressure head is large and the flow small. It is also important that the flocs formed in conditioned residuals are not sheered while being pumped or pressurized. Generally, positive-displacement pumps (such as plunger, piston, or progressive cavity pumps) are better than centrifugal pumps in retaining the floc integrity. Some facilities have included two sets of pumps: one set of high capacity, low-head pumps for filling the press and a second set of low-capacity, high-head pumps for pressurization.

Pneumatic residuals feeding has been successfully demonstrated, but the initial capital cost is much greater than that for the pumped system. With the pneumatic system, the residuals are conditioned in a pneumatic tank and forced into the press with compressed air.

All filter media will eventually clog (blind) and washing is required. If the residuals conditions cause calcium carbonate to plate out on the media, an acid wash is required. The acid recirculatory system is by far the most common acid wash method. The acid dissolves the calcium carbonate, and the recirculating motion dislodges loose objects from the media.

A second acid wash system that has been practiced is the acid soaking system. The press is simply filled with acid and allowed to stand, usually overnight. A disadvantage in the soaking system is that pockets of gas usually form at the top of the press and prevent the acid from coming in contact with all the media.

The washing process can be costly and time consuming. A precoat system can be added to a pressure filter press to help prevent premature blinding and prolong the period of time between washings. Precoat systems also assist in cake release by providing a film between the cake and the media.

In a precoat system, wash water is circulated through the press at high velocities at the beginning of the filtration cycle to dislodge and remove most particles on the media. The wash water is stored in a holding tank and can be used several times before needing to be discarded. After washing, a mixture of water and a small amount of filter aid (usually either fly ash or diatomaceous earth) is circulated through the press to coat the media. The precoating step takes about 3 to 5 minutes.

The above-described system is termed a *dry material feed system*, which is used primarily on the larger installations. The wet system is quite similar to the dry system except a precoat material preparation with mixer is utilized instead of a precoat tank.

A typical fly ash application rate is 4.8 kg/10 m^2 (10 lb/100 ft^2) compared to 2.9 kg/10 m^2 (6 lb/100 ft^2) for diatomaceous earth. Fly ash, however, is a more economical filter aid than diatomaceous earth, because the purchase price of diatomaceous earth is generally 10 to 20 times that of fly ash.

The precoat pumps are generally sized to pump at the rate of 0.2 to 0.3 $L/m^2 \cdot sec$ (0.3 to 0.5 gpm/ft^2) of filter area. The filtrate storage tank is generally twice the volume of the press.

When dewatering hydroxide residuals with plate-and-frame filter presses, the addition of a residuals conditioner has been the standard practice. The conditioner can either be a flocculating agent (like polymer) or a flocculating and bulking agent (like lime and fly ash). Lime has been used almost exclusively as the conditioning agent.

In conditioning of alum residuals, lime has been added in sufficient quantities to adjust the sludge pH to about 11. The amount of lime added is a function of the desired yield. The higher the lime dose, the higher the yield. Lime doses can be as much as 20 to 30 percent by weight.

A decreased lime requirement has been demonstrated in the two-stage conditioning system. A portion of the lime is initially added to the residuals, followed by polymer addition. After a short residence time, the supernatant is decanted and additional lime is added to raise the pH to about 11. Different conditioning systems and variations should be tested on the pilot scale before full-scale implementation.

If lime conditioning is utilized, the filtrate from the pressure filter press is operationally at a pH value of about 11, requiring special attention. The filtrate is to be treated and/or disposed of separately. Another waste stream generated by a filter press operation is the acid wash waste. Generally, neutralization and discharge to a sewer system has been allowed.

Operation Considerations

Probably the most important controllable factor that affects the rate of filtration after a particular pressure filter press is in operation is the conditioning of the residuals. The SR test, the CST test, and the high-pressure filtration test can be used to measure the effectiveness of the conditioner used. Also, it has been determined that a compressibility coefficient of <1.0 is required for pressure filtration. At values >1.0, the specific resistance to filtration increases faster than the applied pressure differential, and a poor cake is formed. The addition of lime or other bulking agents can be added to reduce the residuals compressibility.

Proper maintenance of filter plates and filter cloth is critical in maintaining the design performance and preventing damage to the plates. Frequent cleaning of the media (perhaps as often as once every 8 to 10 cycles) will improve performance, reduce the time spent manually removing cake "stickers," and prolong the life of the cloth and possibly the plates themselves. Plates can be warped or broken if the media are nonuniformly blinded because of the pressure gradients created across the plate. Plate damage may be caused by uneven pressures in a chamber or between adjacent chambers. Media replacement may be required every 1,000 to 4,000 filter cycles.

Small amounts of leakage between plates are usually not cause for alarm. As the cake formation stage proceeds, the leakage should reduce. Cleaning or removing of filter cloth wrinkles along the adjoining surfaces of the plates will generally end any remaining leaks.

Filtrate cloudiness is a potential sign of poor press performance. At the beginning of the filter cycle, some filtrate cloudiness is expected because the filtering cake has not yet formed. Continued cloudiness during the cycle may be a sign of a tear in the media or that the system pressure is too great or is pulsating.

Uneven filter cakes can be caused by several factors. Insufficient pressure or feed flow, or a clogged feed port can cause uneven filter cakes. Clogged feed ports can also create a differential pressure across a plate (one chamber full of sludge under pressure, the adjacent chamber empty) resulting in plate damage. A free-filtering residual in a bottom discharge press may experience rapid filtration at the chamber bottom resulting in an uneven buildup of cake along the full media surface. A top discharge press should be considered for this type of residual.

Operational safety is generally provided by an electric light curtain that automatically shuts off the machine when an object crosses its path. This protects workers from injury when in the vicinity of the shifting plates. Photoelectric cells are stacked vertically in pairs across the front of the press. Each pair of cells emits a light beam which, when broken, activates the signal to stop the machine.

The City of Norfolk, Va., operates two 150-chamber diaphragm filter presses at the Moores Bridges Water Treatment Plant (Albertson 1995). The original press was installed in 1987, and the second press was added in 1994. The filter presses dewater aluminum hydroxide residuals that are generated from a conventional water treatment process. The plant is currently rated for a maximum capacity of 78 mgd (295 ML/d); treats an average of 56 mgd (212 ML/d); and generates residuals at an average rate of 16,800 lb/d (7,620 kg/d) (dry basis).

Residuals are collected in the sedimentation basins and transferred to gravity thickening tanks, where the residuals are conditioned with polymer, then thickened and stored. The thickener decant overflow is discharged directly to the sanitary sewer. During the dewatering operation, thickened residuals are transferred from the gravity thickeners to residuals holding tanks. Lime slurry is metered into the tanks simultaneously with the thickened residuals and mixed. The conditioned residuals are then fed to the filter presses in a batch operation. Throughout the cycle, filtrate is released, equalized, and returned to the gravity thickeners. As the filter press is opened, the dewatered residuals drop directly into a trailer container for final disposal at a landfill.

The diaphragm filter press primarily consists of a steel frame that supports recessed filter plates and filter cloths. The plates are pressed together by a hydraulic ram system to form water-tight chambers lined by the filter cloths into which the conditioned residuals slurry is fed. The slurry is fed through a bottom center port from both sides of the filter press. As the chambers fill with the slurry, solids are trapped between the cloths, while the filtrate passes through. The filtrate drains beneath the cloths to two ports located in opposite corners of the filter plates and is discharged from the end of the press by gravity. After the chambers are filled with partially dewatered solids, diaphragms that line the filter plates are expanded with pressurized water to squeeze additional filtrate out of the solids. The plates are then opened sequentially to allow removal of the dewatered residuals.

A computer control system monitors and controls the operation of the complete dewatering facility including gravity thickeners and dewatering system. The dewatering cycle of the filter press is performed in the following sequence:

1. Plate closure
2. Initial feed
3. Final feed
4. Squeeze
5. Core wash
6. Core purge (air)
7. Filtrate purge
8. Diaphragm purge (air)
9. Cake discharge

The majority auxiliary equipment of the filter press includes initial fill pumps, final fill pumps, squeeze water pumps, and a compressed air system. The hydraulic system consists of a hydraulic cylinder, oil reservoir, and two hydraulic pumps. One hydraulic pump operates during the opening and closing of the press, and a smaller pump maintains a required 3,000-psi (21,000-kPa) pressure to seal the press during dewatering. When the sealing pressure of the hydraulic closure is obtained, the initial feed pump is started to quickly fill the chambers of the press. Upon completion of this step, the final feed pump is initiated to provide a lower feed rate at higher pressures. The initial feed and final feed steps may be terminated automatically based on an adjustable time period, feed pressure, or filtrate flow rate. The squeeze water pump pressurizes the diaphragms with water stored and returned in steel tanks for reuse. The compressed air system provides a high volume of compressed air for the purge steps at the end of the dewatering cycle.

The production rate of the diaphragm filter press is expressed as the net yield, which is expressed in terms of pounds of dry solids dewatered (excluding the weight of conditioning chemicals, such as lime) per hour per square foot (lb/hr•ft^2) of filtration area. The filtration area is the total combined area of the filter cloths. The time used to determine net yield includes time for discharge and mechanical time, which is time required for equipment turnaround, periodic cloth washing, and equipment maintenance.

The net yield gives a direct indication of the performance ability of the filter press. Factors affecting the net yield include feed concentration, chemical conditioning, and the type of residual being dewatered. To determine the net yield, various operating parameters need to be measured to determine the amount of dry solids dewatered such as combination of feed volume and concentration prior to the addition of conditioning chemical, dewatered residuals total weight and concentration, filtrate volume, and total cycle time including discharge and an estimated allowance for mechanical time.

Maximizing the net yield may be accomplished through optimization testing and evaluations to determine the proper balance between feed concentration, filling and squeezing times, final pressure, dewatered residuals concentration, lime use, and cloth selection. An increased yield means that the press has to be operated less for a given amount of solids. This will reduce operation and maintenance costs and extend equipment life. However, in some cases, a maximum net yield may have to be sacrificed for other operating parameters such as the dewatered residuals concentration, which may be more important.

The following performance is typical of the dewatering operation at the Moores Bridges WTP before and after optimization:

Feed residuals concentration (% TS)	30
Lime dosage (wt/wt % as CaO)	50 to 67
Dewatered residuals concentration (% TS)	30 to 40
Filtrate TSS (mg/L)	<50
Net yield (lb/hr•ft^2)	0.35 to 0.50
Total cycle process time (min)	48

The solids concentration of the feed residuals is basically a fixed parameter that is controlled by the operation of the gravity thickeners and the transfer pumping operation. For higher feed concentrations, the chambers of the filter press will fill with solids quicker, less filtrate will have to be released through the cloths, and cake will form more rapidly.

During the planning of a filter facility, an evaluation should be conducted to determine whether it is cost-effective to provide gravity or mechanical thickening prior to the filter press. If thickening is provided, the net yield of the filter press may be increased enough to significantly reduce equipment requirements for the filter press. For existing facilities without thickening prior to the filter press, it may also be cost-effective to provide thickening. This will provide a higher net yield which will permit lower operating and maintenance costs, extend equipment life, and allow more time to perform preventive maintenance.

Lime dosage is typically expressed as percent CaO based on weight (pounds of lime, CaO, per pound residuals solids). The lime used at this facility is delivered in slurry form as $Ca(OH)_2$ at about 25 percent solids and 60 percent CaO. The lime dosage for this facility can be calculated with Equation 4.8, provided the lime pumps are operated at the same time as the thickened residuals transfer pumps (started and stopped together):

$$\text{percent lime (as CaO)} = \frac{(\text{lime flow}) \times (\text{lime sp gr}) \times (\text{lime purity}) \times (\text{lime concentration})}{(\text{sludge flow}) \times (\text{sludge concentration})} \tag{4.8}$$

where lime flow = total lime metering rate into the residuals holding tanks (gpm)
lime sp gr = specific gravity of lime solution (typically 1.15)
lime purity = the percent CaO in the lime solution (typically 60 percent)
lime concentration = the percent total solids in the lime solution (typically 25 percent)
residuals flow = the total pumping rate of residuals from the thickeners (gpm)
residuals concentration = the percent total solids in the residuals (percent)

As the lime dosage is increased, the press will perform better by obtaining drier residuals; however, only a certain amount of lime is required to dewater effectively. A higher dosage will result in excessive lime use which will lower the net yield. The net yield will be decreased because lime solids are taking the place of residuals solids which limits the amount of solids that can be dewatered per cycle. The residuals generated at this facility have relatively consistent characteristics and do not generally require excessive variations in the lime dosage. However, if a change in residuals characteristics occur, the operator will notice a change in the filtrate flow rates throughout the cycle and adjust the lime dosage accordingly. Also, the dewatered residuals may by visually inspected for color; the residuals range from gray to very dark black depending on the amount of lime added.

The fill-and-squeeze step times directly affect the net yield. The time for the initial feed, final feed, and squeeze steps should be optimized to minimize the total cycle duration and still produce an acceptable final dewatered residuals concentration. The filtrate flow rate is a very important indication as to when these steps can be terminated. As the filtrate flow rate decreases, a point will be reached where any extension in step time will have a very minimal increase in solids concentration, but will severely lower the net yield and equipment life. A computer control system proved to be a large benefit to control the cycle times. The computer can graphically produce a continuously updated curve of filtrate flow versus time on the control monitor screen. The operator may use this curve to optimize fill and squeeze step times, terminate early, or extend times, and adjust lime dosage from run to run. Feed pressure of the initial and final feed steps is also a good indication as to when these steps may be terminated.

The minimum solids concentration is regulated by the landfill and is presently at 30 percent total solids. Because the disposal fees are based on weight ($/ton), an increase in solids concentration will result in a savings in disposal costs. However, the disposal costs are relatively less than the cost resulting in an increase in lime dosage; therefore, the lime dose should not be increased as a practice to obtain drier residuals and reduce disposal costs. The fill time or squeeze times may be extended to obtain drier residuals, but this will reduce the net yield. This practice may be beneficial if it is critical to obtain dry residuals.

The filtrate that is discharged from the filter press is of high quality, relative to total suspended solids, with concentrations typically <50 mg/L. However, the filtrate is susceptible to high suspended solids, has a pH in the range of 12.5 to 13.5, and has a high calcium content. This represents difficulties with either recycling the filtrate to the treatment plant or discharging to the sanitary sewer without pretreatment.

At this facility, the filtrate is recycled to the influent of the gravity thickeners and combined with the residuals being transferred from the sedimentation basins to help optimize the operation. This increases the pH of the residuals and helps reduce the amount of lime required; it also reduces the pH of the filtrate. Since the filtrate recycling operation was implemented, lime use was reduced by 10 to 20 percent. It has also increased the net yield of the filter press because less lime is used. The decant overflow of the gravity thickeners, which includes the filtrate, is discharged directly to the sanitary sewer.

Belt Filter Press

Description

Belt filter presses use a combination of gravity draining and mechanical pressure to dewater residuals. A typical belt filter press consists of a chemical conditioning stage, a gravity drainage stage, and a compression dewatering stage (see Figure 4.11).

The dewatering process starts after the feed residuals have been properly conditioned, usually with polymer. The slurry enters the gravity drainage stage, where it is evenly distributed onto a moving, porous belt. Readily drainable water passes through the belt as the slurry travels over the full length of the dewatering stage. Typically, 1 or 2 minutes are necessary to allow for the filtrate separation in the drainage stage.

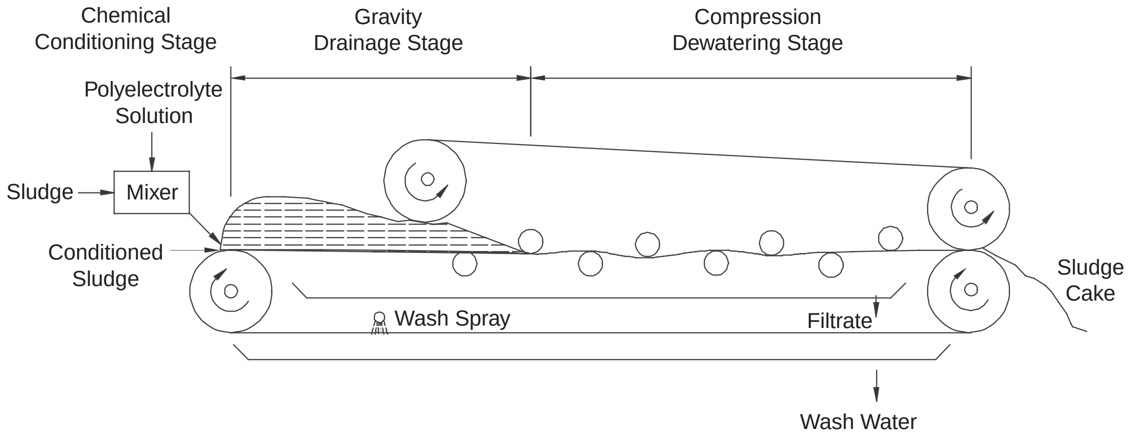
Following gravity drainage, the partially dewatered residuals enter the compression dewatering stage. Here, the residuals are "sandwiched" between two porous cloth media belts that travel in an S-shaped path over numerous rollers. Both belts operate under a specific tension, which induces dewatering pressure into the residuals. The S-shaped path that the residuals follow creates shear forces that assist in the dewatering process. The compressive and shear forces working on the residuals increase over the length of this dewatering stage. The final residuals cake is removed from the belts by blades.

Design

Proper residuals conditioning is considered critical for obtaining acceptable dewatering results. A typical residuals conditioning unit consists of chemical conditioner storage, metering pumps, mixing equipment (chemical and chemicals/residuals), controls, and process piping.

In general, polymer is used for chemical conditioning. To achieve proper residuals conditioning, the polymer is first diluted to between 0.25 and 0.50 percent by weight before it is applied to the feed residuals. Next, the residuals and the polymer are thoroughly mixed. The required mixing time depends on residuals characteristics and type of polymer used.

Pumps associated with belt filter press operations include residuals feed pumps and chemical metering pumps. For residuals feed pumps, various types of equipment have been used such as piston pumps, progressive cavity pumps, and rotary lobe pumps. However, constant rate pumps are preferred, because intermittent flow affects the belt filter press performance. Design considerations



Source: Innocenti 1988.

Figure 4.11 Example belt filter press

for residuals feed pumps should include hydraulic requirements, type of residuals to be dewatered, and range of solids concentrations. Generally, each press has its own residuals feed pump to control the residuals loading. Standby pumping equipment is usually provided by either interconnecting piping between presses or by separate pumps.

Chemical metering pumps are associated with the residuals conditioning system to provide a proper polymer flow rate to the feed residuals. In general, the positive-displacement pumps (diaphragm or progressive cavity type) are used. Variable-speed drives should be provided to control the output rate.

The design and selection of a belt filter press is often based on the throughput of the machine (i.e., the rate at which the residuals can be dewatered by the press). The throughput capacity can be limited either by the water in the residuals (hydraulically) or by the solids. A belt filter press having a particular type of belt at a particular width has a maximum loading capacity for a particular type of residuals. Generally, the solids loading is considered the most critical factor, and the throughput is expressed in terms of solids loading. The loading units are usually similar to a yield except expressed as belt width, mass/width·time.

The structural main frame of belt filter presses consists of either steel channels, I-beams, or tubing. Regardless of the type of frame used, a protective coating is generally applied. Corrosive-resistant coatings include epoxy paints, hot-dipped galvanizing, and fiberglass envelopes.

Most belt filter presses are equipped with two porous cloth belts, namely, the upper and lower belts. The residuals are supported and compressed between the two belts as they move through the compression dewatering stage of the filter press. Belts are either of the seamed or endless type and should be able to withstand tensile strengths several times greater than the maximum operating tension. A wide variety of belt materials exist, and selection should be based on residuals characteristics, solids capture, and durability. In the compression stage, the belts are supported by rollers that are situated such that tension, shear, and compressive forces are induced onto the residuals. Rollers are available in different types of materials including stainless steel. Protective coatings, such as rubber, should be considered dependent upon the corrosiveness of the environment. Maximum allowable roller deflection should be 1 mm (0.05 in.) at the roller mid-span.

Filtrate from gravity and compression dewatering processes, as well as wash water from the belt washing system, is collected under the belt filter press. From the collection housing units, drainage piping transports the water to a sump or floor drain. The floor area around the belt filter press slopes toward the drainage system to allow the unit to be washed down.

After the residuals have passed through the various dewatering stages of the press, the final residuals cake is removed from the belts by a “doctor blade.” From this point, there are various methods available to remove the cake from the press location. One such option is direct discharge into a dump truck. However, press location and building layout are critical when using this method. Other options include conveyors and hoppers, of which there are many different types available on the market. When selecting cake handling equipment, the following criteria are to be considered: operating history, reliability, seasonal storage, and disposal constraints.

The traveling belt is generally cleaned by a belt washing system that sprays high-pressure water on the belt. Because the belt is continuous, the belt washing system is located so as to wash the belt after cake discharge and prior to the next dewatering cycle. A spray nozzle is generally provided for each belt.

An adjustable belt tensioning system is provided for control of the pressure imposed by the belt on the residuals. The tensioning system can be either pneumatic, hydraulic, or mechanical. Belt life is a direct function of the belt operating tension. Automatic belt tension is sometimes provided.

A belt tracking system is normally provided on a belt filter press to keep the belts tracking over the center of the rollers. Sensors and limit switches are employed to detect belt misalignment. A continuously adjustable roller realigns the belt by pneumatic, electric, or hydraulic means.

Operation Considerations

As with the other mechanical dewatering processes, the successful operation of the belt filter press should consider the solids concentration of the residuals cake produced, the quantity and quality of filtrate and wash-water wastes, and the solids capture ratio. These parameters are dependent on the belt design and fabric, the type and conditioning of the residuals, and the pressure (both magnitude and time) applied to the residuals by the belts and rollers while in the press.

The belt tension is generally set by the manufacturer and although increasing the tension can slightly increase the cake solids concentration in some cases, the belt life can be significantly reduced. In most belt presses, the pressure applied to the residuals by the rollers increases as the residual passes through the cycle.

The belt speed determines the retention time of the residuals in the press and the time that the residuals are subjected to pressure. The belt speed also dictates the throughput through the filter press. It is apparent that although the belt speed is the most controllable operational variable, it is probably also the most critical one.

While the majority of belt presses sold today have some automatic operating features, smaller installations may prefer manual presses. Although manual operation adds flexibility to the operation, protective system interlocks should be provided between each system to prevent mechanical damage.

An emergency “panic button” should be located near the primary operator station or control panel to immediately shut down the press if activated. If the belt press is not to be used for a long period of time (a few days), the press should be properly prepared for the downtime. The residuals should be purged from the pumps, pipes, and press. A nonpotable or potable water supply (with backflow preventers) fed into the suction side of the residuals feed pump can purge the system. Finally, the belts can be sprayed clean with a high-pressure-wash sprayer.

The belt filter press was first introduced in the United States in the 1960s. The presses were quite similar in appearance to the Fourdrinier papermaking machine invented in 1799, which concentrates 0.5 percent slurry into a 20 percent solids cake.

In Europe, belt filter presses have experienced tremendous popularity in dewatering wastewater sludges and are the most common type of mechanical dewatering device in use. Their popularity primarily stems from their relatively low energy requirements when compared to the other mechanical dewatering equipment.

The City of Raleigh, N.C., installed one of the first belt filter presses in the early 1980s to dewater its alum residuals. Sedimentation basin sludge and SFBW were gravity thickened to a 4.75 percent solids concentration. This is a relatively high concentration for gravity thickened alum residuals, which helps the ultimate performance of the belt filter press. The city has two belt filter presses, each 2 m in belt width. The residuals are treated with about 5 to 6 lb/ton (2.5 to 3 g/kg) of polymer and then loaded to the presses at a rate of about 270 to 380 lb/ft•hr (400 to 570 kg/m•hr). Final cake solids concentration averaged 22 percent. The filtrate solids concentration contains about a 1 percent solids concentration, equivalent to about 80 percent solids capture. The relatively low solids capture also helps contribute to the high cake solids concentration that is achieved. Some problems were encountered with short belt life (only about 400 hours). The belts were replaced with seamless fabric consisting of alternating left- and right-hand spirals joined by larger-diameter, cross-machined monofilament. This system makes the ends easy to join on the press, and it becomes an endless fabric belt. The belt life has increased to about 2,000 hours.

Vacuum Filter

Description

In sludge filtration, pressure differential across a filter medium is required to force the water in the sludge through the medium while retaining the solids and ultimately forming a cake. The pressure differential in vacuum filtration is a vacuum applied to the downstream/receiving side of the medium.

The best way to express the conceptual theory of a vacuum filter press is the filter yield. The filter yield is defined as the mass of dry cake solids discharged from the filter media per hour per square foot of filter. Filter yield can be expressed as the product of filtrate production per unit area per unit time multiplied by the parameter *W*, mass of cake deposited per unit volume of filtrate. In the specific resistance equation (Equation 2.12) presented in chapter 2:

$$t = V\left(\frac{\mu S R_r}{PA}\right) + \left(\frac{\mu S R W}{2PA^2}\right)V^2 \tag{4.9}$$

The term $V(\mu S R_r/PA)$ relates to resistance caused by the filter medium. The value of this term is generally small when compared to the other term, $(\mu SR\,W/2PA^2)V^2)$ and in most cases is considered to be negligible. The following expression for filter yield is developed:

$$Y = \left(\frac{2PWD}{\mu SRt_c}\right)^{0.5} \tag{4.10}$$

where Y = filter cake yield (kg/m^2•sec)
P = pressure (N•sec/m^2)
W = feed solids concentration (kg/m^3)
D = drum submergence (fraction of the drum circumference below the sludge surface in the pan) (dimensionless)
μ = viscosity (N•sec/m^2)
SR = specific resistance (m/kg)
t_c = cycle time (time for a complete revolution of the drum) (sec)

All variables in this expression are easily determined by the characteristics of the residuals and the equipment operational setup, except the variable *SR*. While this equation is theoretically correct, it has not proven accurate enough for design. One investigation (Eckenfelder and Ford 1970) indicated the values of W and t_c had a more dramatic impact on the yield than the equation indicated. This same investigation generated the following expression for filter yield, which was recommended for use in vacuum filter design:

$$Y = \left(\frac{2DP^{1-S}}{\mu SR}\right)^{\frac{1}{2}} \frac{W^m}{t_c} n \tag{4.11}$$

where S = coefficient of compressibility (dimensionless)
m, n = dimensionless constants
SR = specific resistance at unit pressure drop
other variables = same units as Equation 4.10

The constants m and n are determined by filter leaf testing. From this equation it is apparent that the filter yield is increased by increasing the drum's submergence (D), vacuum pressure (P) (for normal pressure ranges), and feed solids concentration (W), and by decreasing the residuals' specific resistance and cycle time (t_c). Many times, efforts to increase filter yield will decrease the cake solids concentration.

Most vacuum filters employ a rotating drum with filter cloth on its surface. The drum is partially submerged (10 to 50 percent) in a vat of residuals. The residuals may be agitated to maintain the solids in suspension.

The drum revolves around a horizontal axis of rotation. A vacuum applied at the surface of the drum draws the filtrate through the cloth and cake to the collection piping. The filtrate flow is controlled by a timing valve located at one end of the drum along the axis of rotation. A complete revolution of the drum is divided into three phases: cake pickup or formation, cake drying, and cake discharge.

The cake formation stage takes place while the drum is submerged in the residuals vat. Wet residuals are collected on the filter cloth by the vacuum applied in the drum's surface.

The cake drying stage begins when the residuals collected on the rotation drum surface leave the vat and are exposed to air. The vacuum is continued and the air drawn through the residuals dewaters and assists in drying.

In the cake discharge phase, no vacuum is present and the cake is discharged from the press by various means depending on the type of vacuum press.

Washing of the filter cloth after cake discharge is performed on almost every vacuum filter. This washing removes the solid particles and conditioning agents that could clog the cloth openings and cause blinding. The washing is usually accomplished with a high-pressure spray.

The original vacuum filter was the drum type. In the drum vacuum filter, the filter cloth is attached to and completely covers the drum surface. The formed cake is removed by a scraper plate angled very close to the rotating filter cloth and drum. The cloth is generally washed after cake discharge.

Belt vacuum filters are quite similar to drum filters but employ a nonattached cloth belt along the surface of the rotating drum. During the discharge cycle, the belt is directed away from the drum by a series of rollers. As the belt turns at a sharp angle over the discharge roller, the cake breaks away. The belt can then be spray washed on both sides prior to its return to the drum face and the residuals slurry vat.

Horizontal vacuum filters have also been utilized in the water treatment field. The filter cloth is fed from the top through a feed box designed to evenly distribute the residuals across the width of the filter. As the belt moves down the length of the unit, a series of vacuum boxes draw the filtrate through the cloth into filtrate receivers. The cake is discharged at the end of the unit via the sharp turn of the filter cloth around the end roller. The cloth may be washed on the underside of the machine.

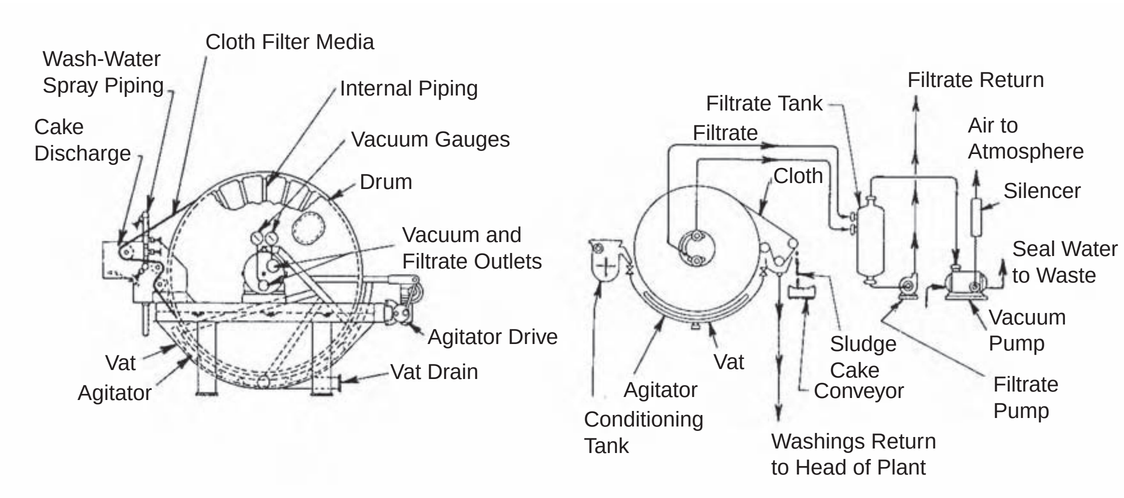
Design

A typical vacuum filtration system is shown in Figure 4.12. Vacuum filtration, in general, is a continuous process (i.e., a continuous supply of residuals produces a continuous discharge of cake and filtrate).

Precoating of the filter medium may be required to prevent blinding when dealing with residuals that are difficult to dewater such as coagulant residuals. Precoating interrupts the continuous process because a preparation period is required to precoat the cloth prior to residuals filtration. In precoating, the drum rotates slowly (normally 5 to 12 rpm) in a vat of precoat material (generally diatomaceous earth, fly ash, or lime) to obtain a precoat layer of 2 to 3 in. (5.1 to 7.6 cm). Time to precoat is generally 50 to 60 minutes.

After forming the precoat cake, the filtering cycle then proceeds and the residuals cake accumulates against the outside of the surface of the precoat cake. An automatic knife blade continually advances at a preset rate of approximately 0.001 to 0.03 in./min (0.003 to 0.08 cm/min). The knife removes the residuals cake and a small amount of the precoat material. When the precoat material is exhausted or reaches a predetermined minimum thickness, the filtration process is discontinued until new precoat is applied.

Cycle time is a critical design and operating consideration that can significantly affect the performance of the vacuum filter. The residuals cake will often begin cracking (therefore breaking the vacuum) at a drying time slightly less than the form time. Therefore, it is advantageous to establish the cycle time accordingly, which normally dictates a high submergence of the drum in the vat.



Source: Innocenti 1988.

Figure 4.12 Example vacuum filter

With longer form times, the cake thickness is increased while the yield is decreased. Optimizing both parameters, cake thickness and yield, has been accomplished using a cake thickness of 0.4 to 0.6 in. (1 to 1.5 cm). Thinner cakes may crack early causing a vacuum break.

The vacuum system for a vacuum filter includes a vacuum pump and vacuum receiver. The vacuum pump supplies the necessary pressure differential across the filter. The vacuum receiver is a tank that separates the filtrate from the air pulled by the vacuum pump during the cake drying stage. The air is allowed to continue through the receiver tank to the vacuum pump while the filtrate is stored for subsequent pumping by a filtrate pump.

Vacuum pumps are generally required to provide from 10 to 25 in. (0.34 to 0.85 kPa) of Hg vacuum. Vacuum receivers are designed to provide an air velocity of up to 4 fps and capacity enough for 2 to 3 minutes of air detention and 4 to 5 minutes of filtrate detention.

The residuals feed system includes the feed pump and the residuals vat. Feed pumps are required to feed residuals to the vacuum filter at the specified rate and are therefore generally of the positive-displacement type. Feed pumps generally are controlled to maintain the liquid level in the residuals vat.

Operation Considerations

The operational variables for a vacuum filter include the cycle time, the residuals feed rate, the residuals level in the vat, the vacuum applied, and choice of residuals conditioning and precoating agents. To some extent, the choice of filter cloth itself is an operating variable, but generally the selection is made during the design phase.

Cycle time can be varied by changing the rotational speed of the drum. Also, by changing the residuals level in the vat, the ratio of the formation time to total cycle time is varied.

The sludge vat is curved to match the drum's curvature and is designed to provide sludge to the vacuum filter. The sludge is slowly agitated (11 to 15 strokes/min) to maintain the solids in suspension and to ensure a homogeneous mixture.

Table 4.4
Average performance and operating data by vacuum filter on lime-softening residuals

Parameter	Range
Feed solids	5–30%
Cake concentration	40–70%
Cake yield	0.8–4.0 kg/m^2•hr 4–20 lb/ft^2•hr
Filtrate solids	950–1,500 mg/L
Solids recovery	95–99%
Filter speed	0.2–0.5 rpm
Operating vacuum	381–635 mm Hg 15–25 in. Hg

The best way to demonstrate the effects of varying the operational variables is to present and discuss data operated from tests performed for that purpose.

Vacuum filters have been evaluated on a pilot-plant scale for use on alum residuals. Although some success has been achieved on the pilot scale, no known full-scale application of vacuum filters for dewatering alum residuals exists. This has primarily been due to the high amounts of conditioning chemicals required, poor cake solids concentrations, and poor yields.

Lime softening residuals dewatering by drum vacuum filters using multifilament polypropylene medium is widely accepted. The most important factors affecting the dewatering of lime sludges are the feed solids concentration and the magnesium content.

A wide range of operating data on traveling belt medium is shown in Table 4.4. The feed concentration, filter yield, and operating vacuum show close correlation. Increased vacuum and feed concentration improves filter yields while cake solids concentration shows only slight fluctuation.

Cycle time selection for vacuum filters on dewatering lime-softening residuals will critically affect the filter's performance. Because both field and laboratory experience has shown that cake cracking occurs at a drying time just slightly less than the form time, a high submergence vat is necessary for obtaining maximum yields.

The first use of a vacuum filter in water treatment was the application of a belt vacuum filter to a lime sludge in a water treatment plant in Minot, N.D. A relatively high magnesium content residuals was successfully dewatered. In the late 1960s, personnel in Boca Raton, Fla., tested and installed a belt vacuum filter on a highly calcium carbonate sludge. A 1 to 4 percent solids concentration sludge from the softening reactors was concentrated to 28 to 32 percent by a thickener. The vacuum filter could be loaded at rates of 60 lb/ft^2•hr (300 kg/m^2•hr) resulting in a final cake concentration of 65 percent. The cake was further air dried prior to disposal.

Numerous applications have since been installed throughout the United States with similar results. The two primary factors affecting performance are the solids feed concentration and the magnesium hydroxide content.

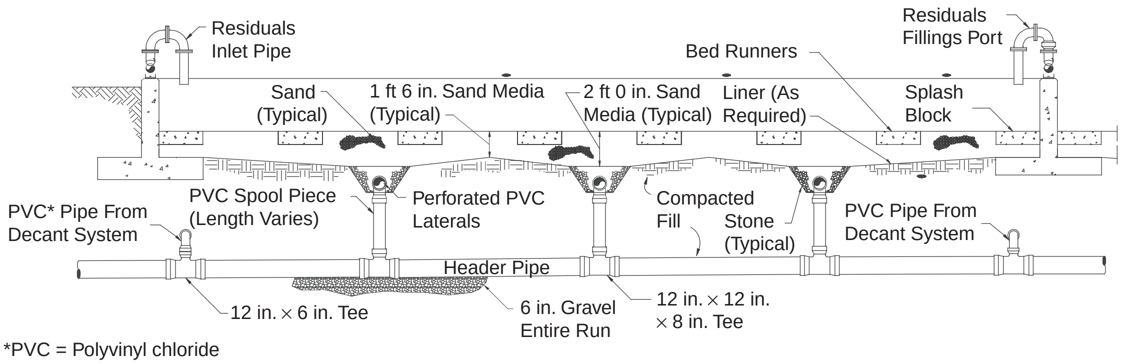


Figure 4.13 Sand drying bed schematic

NONMECHANICAL DEWATERING

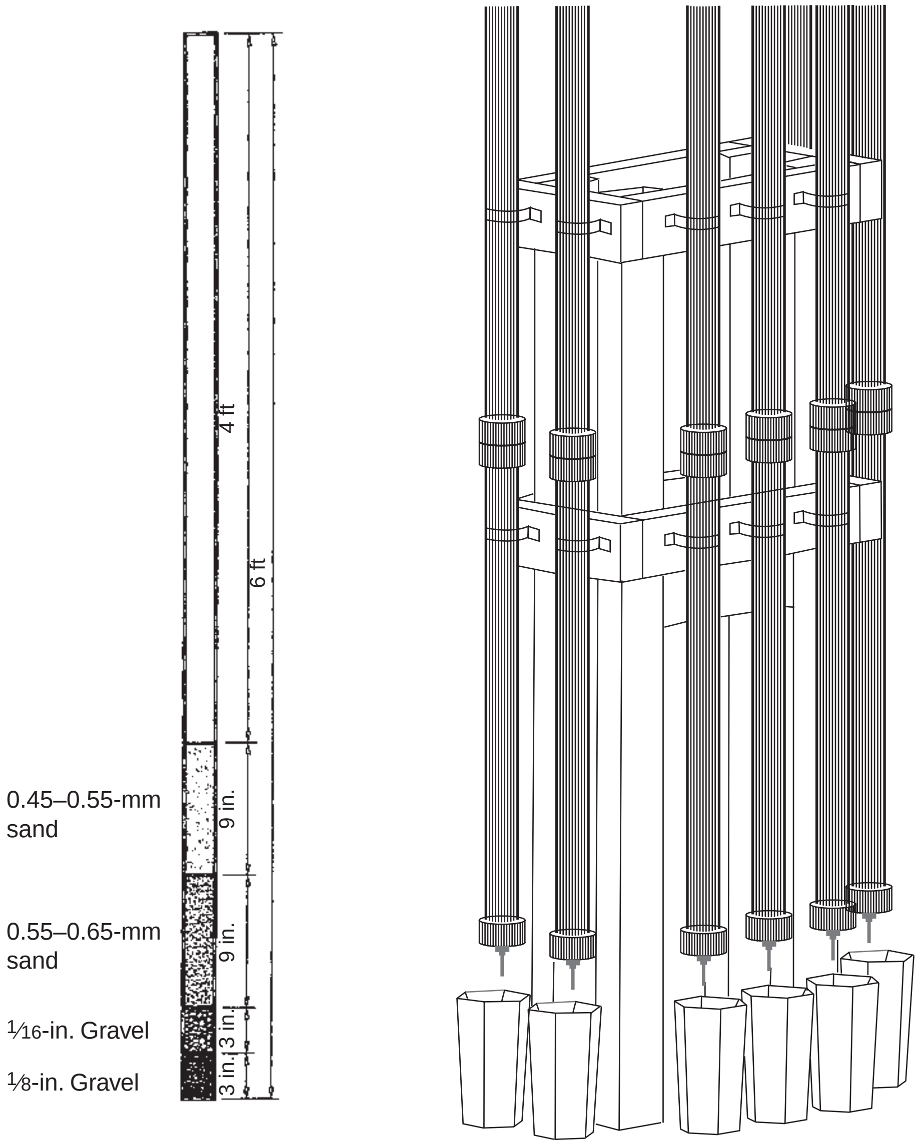
Sand Drying Beds

Description

Developed initially for dewatering municipal wastewater sludges, sand drying beds have been widely used for dewatering WTP sludges. Sand drying beds operate on the simple principle of spreading out the residuals and letting them dry. As much water as possible is removed by drainage or decant, and the rest of the water evaporates until the desired final solids concentration is reached. Sand drying beds have been built as simply as cleaning an area of land, dumping the residuals, and hoping something happens to initiate the drying process, to as sophisticated as installing automated drying systems. A typical sand drying bed layout is shown in Figure 4.13.

Drainage (percolation), decanting, and evaporation are the primary mechanisms for dewatering residuals in sand drying beds. Following residuals application, free water is allowed to drain from the residuals into a sand bottom from which it is transported via an underdrain system consisting of a series of lateral collection pipes. This process continues until the sand is clogged with fine particles or until all the free water has been drained, which may require several days. Secondary free water removal by decanting can take place once a supernatant layer has formed. Decanting can also be utilized to remove rainwater that would otherwise hinder the overall drying process. Water remaining after initial drainage and decanting is removed by evaporation over a period of time necessary to achieve the desired final solids concentration. Because the amount of water that needs to be evaporated is the controlling factor in bed sizing, maximizing the water removed by drainage and decant will reduce the overall required bed size.

Solar drying beds are similar to sand drying beds in terms of shape and operation. However, they are constructed with sealed bottoms, and have sometimes been referred to as paved drying beds. These beds have little or no provisions for water to be removed through drainage; all residuals thickening and drying is accomplished through decant of free water followed by evaporation. A principal advantage of this type of drying bed is low maintenance costs and ease of cleaning. No sand replacement costs are associated with this type of drying bed, and because the bottom of these beds are sealed, neither initial underdrain costs nor underdrain repair costs are



(a) Typical Media Arrangement **(b) Pilot Column Arrangement**

Source: Vandermeyden and Cornwell 1998.

Figure 4.14 Pilot sand drying bed dewatering system

incurred. Also, because the entire solar bed bottom is often paved or concrete, cleaning with front-end loaders can be done quickly and efficiently. Because solar beds rely primarily on evaporation, they typically have lower solids loading rates than sand drying beds. Most solar beds are located where evaporation rates are high.

A dewatering lagoon has a sand and underdrain bottom, similar to a drying bed, and it can be designed to achieve a desired dewatered residuals cake. The dewatering lagoons are deeper than sand drying beds and have fewer cleanings per year. The advantage of a dewatering lagoon over a sand drying bed is that storage is built into the system to assist in meeting peak solids

Table 4.5
Summary of field and pilot comparison test drainage results

Location	Initial solids (%)	Loading rate (lb/ft^2)	Full-scale drainage* without polymer (%)	Pilot-scale drainage* without polymer (%)
A	1.1	0.9	73	73
B	6.5	3.4	37	47
C	1.7	5.0	64	63
D	1.9	3.4	48	44
E†	6.5	3.1	20	21

Source: Vandermeyden and Cornwell 1998.
*Drainage includes decant volume.
†Full-scale and pilot tests performed on solar beds without underdrains.

production or to assist in handling residuals during wet weather. The disadvantage is that the bottom sand layers can clog or blind with multiple loadings, thereby increasing the required surface area compared to conventional drying beds. Polymer treatment can be useful in preventing this sand blinding. Also, evaporation often does not reach to the bottom depth, and tillage is necessary to completely dry the solids.

Laboratory tests, such as the CST and TTF tests (described in chapter 2), are useful for screening polymers or other conditioning agents and to qualitatively assess physical variables such as suspended solids concentration. It has not been demonstrated that laboratory tests can be directly used to quantitatively predict water release in a full-scale drying bed operation. Predicting the percentage of water that can be drained or decanted from the solids is an important variable in sizing the dewatering facilities. If pilot drainage and decant studies could be used to predict full-scale operation, then that information would be useful for developing design models. Figure 4.14 shows a setup that can be used to evaluate pilot test methods as described by Vandermeyden and Cornwell (1998). Acrylic cylinders approximately 6 ft (1.8 m) tall were used. The cylinders were open at the top and had a ¼-in. (0.64-cm) drain valve on the bottom to remove water that drained through the cloth. Sludge can then be placed into the columns with or without the use of polymer conditioning and at varying loading rates. Loading rate is one of the important variables in sand bed performance. Loading rate is the dry weight of sludge applied to a given area of bed, expressed as pounds per square foot or kilograms per square meter. Loading rate is a function of the solids concentration of the sludge applied to the bed and the depth of sludge application. After sludge application, drainage volume can be recorded with time and the height of free water above the solids in the column measured to estimate decant volume. The drainage volume and decant volume are added together to determine the total volume of water removed by these processes. The researchers compared the pilot column drainage results to full-scale results, as shown in Table 4.5. The results show that the pilot tests were quite good at predicting full-scale performance.

A second important aspect in determining bed size is the time it takes for evaporation to remove sufficient water to reach the desired solids concentration. Rolan's (1980) approach and the approach developed by Vandermeyden and Cornwell (1998) use pan evaporation data to estimate the drying time.

The results from tests by Vandermeyden and Cornwell (1998) suggested that pan evaporation calculations underestimate the evaporation through the initial drying cycle. In three tests, a 25 percent solids concentration was reached in actual field drying beds in about two thirds of the time predicted by using pan evaporation data. Thus, under these test conditions, using pan evaporation would result in a conservative sizing of the sand drying beds or solar drying beds. At higher solids concentrations, pan evaporation predicted evaporation at much higher rates than actually took place. This point of crossover could be when the free water has been evaporated and the pore water and the chemically bound water remains. The bound water would be more difficult to evaporate. The slower drying at the higher solids concentration could also reflect a crusting of the surface in a water treatment plant in Minot wherein tillage could be used to increase the field evaporation. Overall, the researchers concluded that it was reasonable to use pan evaporation data to predict the drying time, realizing that the beds may be oversized when achieving a final cake solids concentration of 25 to 30 percent, but undersized if high final cake solids concentrations are required.

Design

Several models have been previously developed to estimate a required sand drying bed area for a particular water utility. In a book published by ASCE and AWWA (1990), the drying bed size is based on the effective number of uses that may be applied to each bed and the depth of residuals applied onto the bed based on the following equation:

$$A = \frac{V}{AA \times D(i) \times 7.5} \tag{4.12}$$

where A = drying bed area (ft^2) (ft$^2 \times 0.093$ = m^2)
V = annual volume of residuals (gal)
AA = number of uses for the beds each year
$D(i)$ = depth of residuals applied (ft)
7.5 = gal/ft^3

In this particular model, the number of uses for each bed per year must be estimated. This would be a function of the drainage time, drying of the residuals through evaporation, and cleaning of the bed after the desired final solids concentration has been achieved. This would be a difficult number to estimate or to predict for different operating conditions.

A different criterion for sizing drying beds would be the use of a specific drying bed yield, in terms of pounds per square foot per year (kilograms per cubic meter per year). An appropriate yield should take into account seasonal conditions such as temperature, wind velocity, precipitation, residuals characteristics, and initial residuals solids concentration. Ignoring such factors and using only average annual conditions could result in undersizing of the sand drying beds.

The design and operation of a sand drying bed is primarily a function of the following parameters:

- Initial solids concentration of the applied residuals
- Depth of the residuals applied on the beds
- Loss of free water through the underdrain or decant system
- Net evaporation rate
- Desired final solids concentration

All of these factors must be considered in order to determine the optimum loading rate and resulting yield for a given location. Since most of these factors are very site specific, any mathematical modeling for determining the drying bed design should take these local factors into consideration.

Rolan (1980) developed a series of equations that can be used to determine the sand drying bed yield based on a certain loading rate. Rolan's model also takes into account the operational parameters listed previously and is a comprehensive mathematical approach to assess the sand drying bed performance and size for a particular location. The model calculates the initial loading (*L*) in pounds per square foot for a given application based on the residuals depth applied, *D(i)*, and the initial residuals solids concentration, *SS(i)*, by using the following equation:

$$L = \frac{D(i)}{12} = \frac{SS(i)}{100} = 62.4 \tag{4.13}$$

where L = the initial loading (lb/ft^2) (lb/ft$^2 \times 5$ = kg/m^2)
$D(i)$ = the initial depth (in.)
$SS(i)$ = the initial solids concentration (percent)
62.4 = density of water (lb/ft^3)

The drying bed yield, *Y*, in pounds per square foot per year is a function of the residuals loading applied to the bed (*L*) and the number of applications per year (*AA*). The drying bed yield can be calculated as follows:

$$Y = L \times AA \tag{4.14}$$

where Y = the drying bed yield (lb/ft^2•year) (lb/ft^2•year × 5 = kg/m^2•year)
L = the initial loading (lb/ft^2)
AA = the annual number of applications

Rolan (1980) proceeded to determine *AA* by estimating the required drying time per bed application. The required drying time is calculated by knowing the amount of water that is removed by decant and drainage, and applying an evaporation time to the remaining water to be removed. The drawback of the model is that it is somewhat cumbersome, and monthly and seasonal variations in sludge production or evaporation are difficult to account for.

By building on the work of Rolan, the approach was modified by Vandermeyden and Cornwell (1998) to both simplify the relationships and develop a monthly mass balance approach to sand bed sizing that allows seasonal variables to be taken into account. The following expression was developed to find the bed yield:

$$Y = (0.624)\frac{SS(f)SS(d)(E)}{SS(f) - SS(d)} \tag{4.15}$$

where Y = sand bed yield (lb/ft^2•year) (lb/ft^2•year × 5 = kg/m^2•year)
0.624 = unit-specific conversion constant
$SS(f)$ = desired final solids concentration (percent)
$SS(d)$ = drained solids concentration (percent)
E = net pan evaporation (in./mo)

Because E is fixed, and $SS(f)$ is set as desired, the only variable is the drained solids concentration. Clearly, examination of the equation shows that maximizing $SS(d)$ will maximize Y. Therefore, a utility would run a series of pilot drainage tests in order to find $SS(d)$. The first variable in the pilot studies would be the initial suspended (or total) solids concentration. Two suspended solids values may be sufficient to run; one would be the suspended solids concentration as directly obtained from the sedimentation basins. A second condition to test would be after addition of a thickener. This comparison would essentially allow a cost trade-off to be made between thickener cost and sand drying bed cost. However, if seasonal variations in the initial sludge suspended solids concentrations exist, those conditions should be evaluated. The next variable to evaluate would be the solids loading. Good results for coagulant sludge have been found for loadings between about 2 to 6 lb/ft^2 (10 to 30 kg/m^2); for lime sludge the loadings could be more in the range of 10 to 15 lb/ft^2 (50 to 60 kg/m^2). The final variable would be polymer conditioning. Therefore, to test two solids concentrations, three loading rates, with and without polymer, 12 pilot drainage tests would be needed. For each condition the drained solids concentration should be calculated. The drained solids concentration can be determined by measuring the initial applied solids concentration and multiplying by the ratio of the initial depth to the drained solids depth. Those conditions that maximize the drained solids concentrations would be logical to carry over for sizing analysis.

An initial rough estimate of the sand bed area required could be made based on annual average conditions. As an example, use a 1.1 percent applied SS and 2-lb/ft^2 (10-kg/m^2) loading rate, and assume the drained solids concentration was found from the pilot studies to be 4.0 percent without polymer and 7.3 percent with polymer. Using an average annual net evaporation of 4.1 in. (10.4 cm) and a sludge production of 1,000 tons/year (907 kg/year), for a 20 percent final solids concentration:

$$Y(\text{without polymer}) = (0.624)\frac{(20)(4)(4.1)}{20-4} = 12.8\ \text{lb/ft}^2\cdot\text{year}(64\ \text{kg/m}^2\cdot\text{year}) \tag{4.16}$$

and the required area would be

$$\text{area} = \frac{(1{,}000)(2{,}000)}{12.8} = 156{,}000\ \text{ft}^2(14{,}600\ \text{m}^2) \tag{4.17}$$

Finding the required area with polymer would result in a yield of 29.4 lb/ft^2·year (147 kg/m^2·year) and an area requirement of 68,000 ft^2 (6,400 m^2). In this example, the use of polymer would reduce the required drying area by about 55 percent. This type of comparative analysis, perhaps coupled with general cost assumptions, would result in the selection of the desired design conditions.

This approach is based on annual yields and does not account for monthly variations. A monthly calculation should be done based on a mass balance approach by determining the bed area required each month to accommodate the sludge production and determining how long the bed is used to provide the necessary drying. There are several approaches to this calculation, but the most straightforward is to convert the required evaporative drying into inches of evaporation required. This can be found as

$$\Delta D(e) = D(d) - \left(D(d)\frac{SS(d)}{SS(f)}\right) \tag{4.18}$$

where $D(e)$ = required depth change due to the evaporation
$D(d)$ = drained depth

Note that it is possible for $D(d)$ to vary seasonally, especially if $SS(i)$ varies seasonally.

Using Equation 4.18, $D(i)$ was 35 in. (88.9 cm) and $SS(d)$ with polymer was 7.3 percent, therefore:

$$D(d) = 35\frac{1.1}{7.3} = 5.27\text{ in.}(13.4\text{ cm}) \tag{4.19}$$

$$\Delta D(e) = 5.27 - \left(5.27\frac{7.3}{20}\right) = 3.4\text{ in. }(8.6\text{ cm}) \tag{4.20}$$

In order to dry the solids to 20 percent, 3.4 in. (8.6 cm) of net pan evaporation is required. On the average, this would take less than 1 month for each loading at an annual average pan evaporation of 4.1 in./month, but monthly evaporation can vary greatly.

The first step in the monthly tabulations is shown as an example in Table 4.6. Table 4.6 shows the average daily solids production and the net evaporation for each month. Solids production data must be determined by the utility. Evaporation data, if not available locally, can be obtained from the National Climatic Data Center in Asheville, N.C. The drying bed area that must be loaded for a particular month is calculated as the daily production multiplied by 30 days/month divided by the chosen solids loading from pilot studies, in this case, 2 lb/ft^2 (10 kg/m^2). The area indicates the amount of drying bed that must be available for loading during the month. Next, the solids drying time is calculated as the sum of the monthly evaporations until the required evaporation ($D(e)$) is reached, in this case, 3.4 in. (8.6 cm). So for January,

$$D(e) = 3.4 = 1.0\text{ (January)} + 1.9\text{ (February)} + 0.1(3.5)\text{ (March)} \tag{4.21}$$

or 2.1 months are needed—January, February, and one tenth of March. This calculation is completed for each month and shown in Table 4.6.

A mass balance can now be constructed. In this example, it has been assumed that all the solids for the month are produced on day 1 of the month and that they are on the bed all of that month. A bed that has solids on it from a previous month is not available at all during the month with the assumption that the solids are all loaded on day 1. So in the January example, 2.1 months were required for drying; therefore, the bed would be occupied for 3 months. The mass balance can be broken down to a weekly basis for a more accurate analysis. The monthly versus weekly approach would also be impacted by the amount of sludge storage available. However, this somewhat conservative approach of using a monthly loading assumption is generally justified given that time is required to remove solids from the bed, there may be inclement weather, and so forth.

The mass balance can begin in any convenient month. In this case, October is convenient as a review of the drying time data would clearly show that there would be no carryover beds for this month. Table 4.7 shows the mass balance approach.

The first column of Table 4.7 shows the bed area that must be loaded that month (from Table 4.6), and the second column shows the bed area that is still in use from previous months. Using this approach, the required bed area is 378,000 ft^2 (35,100 m^2). This required area is about

Table 4.6
Tabulation of monthly sand drying bed area requirements

Month	Solids production (lb/d)	Net evaporation rate (in./month)	Solids drying time (months)	Drying bed area loaded* (ft^2)
January	7,820	1.0	2.1	117,000
February	8,275	1.9	1.4	124,000
March	9,145	3.5	1.0	137,000
April	9,350	4.7	0.7	140,000
May	5,265	5.8	0.6	79,000
June	3,395	6.9	0.5	51,000
July	3,240	7.0	0.5	49,000
August	4,880	6.5	0.5	73,000
September	3,145	4.5	0.8	47,000
October	4,125	3.5	1.0	62,000
November	3,650	2.6	1.4	55,000
December	4,470	1.8	2.3	67,000

Source: Vandermeyden and Cornwell 1998.
NOTES: lb/d × 0.4536 = kg/d; in./month × 2.54 = cm/month; $ft^2 \times 0.093 = m^2$.
*Design solids loading is 2.0 lb/ft^2 (10 kg/m^2).

five times the area required based on the average annual sludge production and loading rate shown previously (68,000 ft^2 [6,400 m^2]). This high required area compared to the annual average is due to this utility having a high winter sludge production when the net evaporation is quite low.

Solar Drying Bed

The solar bed sizing process would be similar to that used for the sand drying bed. This process consists of loading the solar bed to a certain depth and, after a settling period, decanting the supernatant. The remaining residuals depth and solids concentration would be exposed to the evaporative cycle until the desired final cake solids concentration is achieved. In this case, there is generally no drainage and the first step would be to maximize the amount of decant that can be achieved in order to minimize the amount of evaporation. Again, polymers may aid in increasing the water that can be removed. As for a sand bed, a pilot settling test under different loading and polymer conditions could be conducted in order to determine the volume of decant and hence the decanted solids concentration. In the pilot study for a solar bed, the bottom would be sealed to prevent drainage. A mass balance approach would then be conducted similarly to the sand drying bed example.

Using the data from the sand drying bed example, but at a loading rate of 1 lb/ft^2 (5 kg/m^2), assume that a 4 percent decanted solids concentration could be achieved, which would

Table 4.7
Monthly mass balance example for sand drying bed

Month	Bed loading area (ft^2)	Carryover bed area (ft^2)	Net bed required (ft^2)
October	62,000		62,000
November	55,000	0	55,000
December	67,000	55,000 (Nov.)	122,000
January	117,000	67,000 (Dec.)	184,000
February	124,000	67,000 (Dec.) 117,000 (Jan.)	308,000
March	137,000	117,000 (Jan.) 124,000 (Feb.)	378,000
April	140,000	0	140,000
May	79,000	0	79,000
June	51,000	0	51,000
July	49,000	0	49,000
August	73,000	0	73,000
September	47,000	0	47,000

Source: Vandermeyden and Cornwell 1998.
NOTE: $ft^2 \times 0.093 = m^2$.

be typical for a polymer-conditioned alum sludge. Then the amount of water that must be evaporated is found as

$$D(d) = 17\frac{1.1}{4} = 4.68 \text{ in. } (11.9 \text{ cm}) \tag{4.22}$$

$$\Delta D(e) = 4.68 - \left(4.68\frac{4}{20}\right) = 3.75 \text{ in. } (9.5 \text{ cm}) \tag{4.23}$$

Using only the beginning of the month bed availability criteria therefore results in the mass balance tables (Tables 4.8 and 4.9). In this example, 750,000 ft^2 (70,100 m^2) of solar bed area would be required, as compared to 378,000 ft^2 (35,100 m^2) for the sand drying bed. In general, a sand bed will result in a lower area requirement except for a residual that can be settled and decanted to reach the same drained solids concentration as that to which it can be drained and decanted. However, in areas with a high evaporation rate, solar drying beds may still be preferred over sand beds because of their ease of construction, low maintenance, and ease of cleaning.

Table 4.8
Tabulation of monthly solar bed area requirements

Month	Solids production (lb/d)	Net evaporation rate (in./month)	Solids drying time (months)	Solar drying area loaded (ft^2)
January	7,820	1.0	2.2	235,000
February	8,275	1.9	1.5	248,000
March	9,145	3.5	1.1	274,000
April	9,350	4.7	0.8	280,000
May	5,265	5.8	0.7	158,000
June	3,395	6.9	0.5	102,000
July	3,240	7.0	0.5	97,000
August	4,880	6.5	0.6	146,000
September	3,145	4.5	0.8	94,000
October	4,125	3.5	1.1	124,000
November	3,650	2.6	1.6	109,000
December	4,470	1.8	2.5	134,000

Source: Vandermeyden and Cornwell 1998.
NOTES: lb/d × 0.4536 = kg/d; in./month × 2.54 = cm/month; $ft^2 \times 0.093 = m^2$.

Table 4.9
Monthly mass balance example for solar drying bed

Month	Bed loading area (ft^2)	Carryover bed area (ft^2)	Net bed required (ft^2)
October	124,000		124,000
November	109,000	124,000 (Oct.)	233,000
December	134,000	109,000 (Nov.)	243,000
January	235,000	134,000 (Dec.)	369,000
February	248,000	134,000 (Dec.) 125,000 (Jan.)	617,000
March	274,000	235,000 (Jan.) 248,000 (Feb.)	757,000
April	280,000	274,000 (Mar.)	554,000
May	158,000	0	158,000
June	102,000	0	102,000
July	97,000	0	97,000
August	146,000	0	146,000
September	94,000	0	94,000

Source: Vandermeyden and Cornwell 1998.
NOTE: $ft^2 \times 0.093 = m^2$.

Supplemental Air Drying

In many cases it is advantageous to use air drying to further dewater residuals that result after mechanical dewatering. As previously discussed, a centrifuge or belt press dewaters coagulant sludge to the 20 percent solids concentration range. Air drying the residuals to a higher cake concentration can reduce hauling costs, reduce landfill space, or render the residuals more acceptable to beneficial users.

Air drying can be accomplished by spreading out the residuals in layers, rows, or mounds. The shallower the application, the shorter the drying time. Area requirements and drying time can be predicted by using the sand bed equations. Utilities have reported improved drying results when using deeper applications if the residuals are occasionally turned.

Dewatering Lagoons

Dewatering lagoons are very similar to sand drying beds except that they operate at much higher loadings. The dewatering lagoon should be equipped with a decant structure and underdrains. The lagoon is filled over a long time period and then allowed to dry for a long period while another lagoon is filled. Dewatering lagoons can have an advantage over sand drying beds in reducing peaks, as the loading is often spread over several months. Because dewatering lagoons use a much higher loading rate, the drainage volume would generally be lower than a sand drying bed. The main difficulty in sizing a dewatering lagoon is in predicting the drained solids concentration after the loading is complete. Plugging of the sand media on the bottom of the dewatering lagoon with multiple loadings is difficult to predict and would require a carefully planned pilot test with 2-in. (5.1-cm) or 6-in. (15.2-cm) dewatering columns, or even pilot-scale dewatering lagoons might be necessary to accurately size and design the system. Solids at the bottom of the lagoon would have a higher solids concentration than solids at the top of the lagoon and a net average solids concentration must be estimated. During the evaporation phase, the bottom layers often do not dry out. Some utilities have found that tilling the sludge during the evaporative cycle helps to expose all of the residuals to drying.

Again, considering the previous example, a lagoon can be sized by selecting a desired fill cycle and estimating the drained solids concentration. In this example, a 6-month fill cycle was chosen to help reduce the winter sludge production peaks. The highest production period is from January to June wherein about 650 tons (589 metric tons) of sludge are produced (see Table 4.8).

If a 5-ft (1.5-m) depth of sludge in the lagoon is selected, then the lagoon area can be calculated for varying drained solids concentrations as

$$\text{dewatering lagoon area} = \frac{(\text{lb sludge})}{(\text{depth})(\text{percent } SS(d))(62.4)} \tag{4.24}$$

Table 4.10 shows the required area for drained solids concentrations of 4, 6, and 8 percent.

The required drying time is found by Δ*D(e)*, so that at 6 percent *SS(d)* the required change in depth is

$$\Delta D(e) = 60 - \left(60\frac{6}{20}\right) = 42 \text{ in. } (107 \text{ m}) \tag{4.25}$$

Table 4.10
Sizing example for a dewatering lagoon

Drained solids (%)	Area required per lagoon (ft^2)	Drying time required (years)	No. of lagoons	Total area required (ft^2)
4	104,000	1.0	3	312,000
6	69,000	0.9	3	230,000
8	52,000	0.7	3	156,000

Source: Vandermeyden and Cornwell 1998.
NOTE: $ft^2 \times 0.093 = m^2$.

Because the average evaporation in this example is 4.1 in./month (10.4 cm/month), it would take about 1 year to dry the lagoon if evaporation could reach the complete depth through tilling. Therefore, a total lagoon cycle would be 18 months and three lagoons would be needed. The total lagoon area required would be 230,000 ft^2 (21,500 m^2) if a 6 percent drained solids concentration could be achieved as an average in the 5-ft (1.5-m) sludge depth. In the example, all scenarios required three lagoons. Slightly higher than 8 percent solids concentration such that the drying time is reduced to 6 months may be able to reduce the number of lagoons to two, especially considering, in this case, that the high sludge production is in the winter, which is the fill cycle, and the high evaporation is in the summer, which is the drying cycle of the “fullest” lagoon. Several treatment scenarios could be prepared using different fill cycle times to minimize the area. However, the largest impact on the required area is still the estimated drained solids concentration, which is difficult to determine, and oversizing would be appropriate.

Operation Considerations

The prime objective in sand bed residuals dewatering is to reduce the moisture content of the residuals cake to a level consistent with the means of residuals cake removal and ultimate disposal. Most well-managed existing plants have standard procedures for the cycle of residuals dosing and removal of dried cake based on the type of residuals, the allowable moisture content of the dried residuals cake, and local climatological conditions. The allowable moisture content, which is dependent on the place or method of disposal and the type of removal equipment, may be the controlling factor for most drying bed operations.

Rarely are mechanical dewatering devices designed without provision for chemical feed. Designers now typically make the same provision for residuals drying beds. New designs should include chemical conditioning to offset unpredictable weather conditions and variable residuals characteristics (low percent solids, poor drain ability, or changing chemical characteristics). Conditioning also may be useful in improving the residuals drainage rate and thereby increase the capacity of drying beds.

The CST meter and the TTF test may be used for comparative evaluation of both polymer type and dosage. The best economic evaluation makes comparisons based on the grams of chemical added per kilogram of dry residuals solids (pounds per ton of dry solids) rather than on a parts-per-million dosage.

In addition, optimum dosages should be determined with care, because their effectiveness can be hampered by both under- and overdosing. The net and gross residuals bed loadings for chemically treated and untreated beds should be compared in laboratory tests and under actual field conditions. Blinding of the sand may result if excessive amounts of chemicals are used.

When the use of polymer is planned, the bed operation should take into account the angle of repose of applied residuals. With polymer treatment and the rapid release of free water, the uniform distribution of residuals on the sand surface can be affected by the angle of repose of the residuals—often as much as 1 in. of vertical for each 10 ft of horizontal. This problem can be resolved, however, by providing multiple points of addition, providing for partitioning of beds, or limiting the size of each bed.

Equipment limitations (i.e., type of pumps and dimensions of beds) may require the operator to apply a more dilute residual to minimize problems with pumping and distribution of residuals on the bed. By applying a thinner residual at a high rate, the angle of repose effect can be reduced. The free water that could have otherwise been removed by decanting prior to applying residuals to the bed can be easily removed through filtration and decanting on the bed. A uniform layer of residuals is essential to uniform drying and efficient utilization of drying bed area. A trade-off is that thinner residuals may blind the sand at a lower net drainage, thereby increasing the bed requirements.

Increased labor costs have made manual residuals removal economically feasible only in the smallest plants. During manual removal, hand tools are used to lift the dried cake from the sand. One of the best tools is a shovel-like fork with several tines, approximately 1 in. (25 mm) apart. For best results, the cake must be dried sufficiently (generally 25 to 39 percent dry solids) so that it cracks and begins to peel away from the sand. The cake then can be lifted from the sand with a relatively small loss of sand. The removed cake may be forked directly into wheelbarrows; small trucks; or small, rubber-tired wagons. The sand surface should not be required to take wheel loads; concrete treadways or appropriate temporary planking should be used instead.

Many plants now use mechanical removal equipment consisting of either front-end loaders or truck-mounted vacuum removal systems, thereby reducing the labor requirements for residuals removal to a minimum. Because of the cost of operating mechanical removal equipment, the dry cake thickness and moisture content must be optimized. Generally, a dry solids content of 15 to 25 percent is sufficient for mechanical removal.

The City of Durham, N.C., owns and operates two WTPs, one of which (Williams Water Treatment Plant) utilizes sand drying beds for alum residuals dewatering. The source of water for the plant is a nearby surface water reservoir. The plant has utilized alum as a coagulant at an annual average dose of 30 mg/L alum. The monthly average dose is 15 to 45 mg/L.

Sand drying beds have been used for residuals dewatering at the plant since 1978. The residuals handling facilities associated with the beds include decant tanks, residuals pumping, and polymer feed equipment. The facility has both a batch solids thickening (decant tanks) and a dewatering process. The settling basins were constructed with hopper bottoms equipped with manually operated plug valves. Residuals from the settling basin having a solids concentration from 0.5 to 1.0 percent are sent to one of two 77,000-gal (2.9×10^5 L) decant tanks. Thickening of the residuals in the decant tanks for several days to a week has resulted in a settled solids concentration of 3 to 5 percent. Decant water from the tanks is returned to the raw water terminal reservoir for recycle to the WTP. The solids from the decant tanks are pumped to the drying beds. A nonionic polymer at a dose of 12 to 30 lb (6 to 15 g/kg) per ton has been used to aid in dewatering.

The polymer is fed on the suction side of the residuals pumps to provide mixing prior to discharge onto the drying beds.

The residuals are applied to four drying beds, each 5,100 ft^2 (470 m^2) in surface area, at a loading of approximately 2 to 3 lb (908 to 1,362 g) of solids per square foot. The residuals depth generally varies from 1 to 2 ft (30 to 60 cm). The solids have been allowed to dry to a concentration of 15 to 20 percent before being removed from the beds. Residuals are removed from the beds by the use of vacuum trucks which also haul residuals to the local city-owned landfill.

One full-time operator has been assigned to the facility and has been responsible for solids transfer, thickening, decant of supernatant water, polymer feed operations, and general facility maintenance. Plant laboratory personnel have been responsible for the selection of polymer dosage based on the performance of CST testing. Based on operational experience, a polymer dose that would provide a CST of 15 seconds or lower has been desired to provide good dewatering characteristics. Generally, the thickened residuals without polymer addition has a CST of more than 150 seconds. Polymer-conditioned residuals have generally had a CST in the 6-to-15-second range.

The amount of residuals produced at the plant has been estimated based on plant tests and correlation of alum dose and turbidity. Approximately 650 tons/year (1.78 tons/d) (5.9 × 105 kg/d) of solids on average have been produced. The monthly average of solids production has ranged from 0.5 to 3.9 tons/d (453 kg to 3.5×10^3 kg/d) based on historical records.

The drying time on the beds has been a function of climate. During warm weather, the beds have been unloaded after 1 to 2 weeks of drying time. During winter months, the drying time has been 1 to 2 months. One of the most frequent operational problems with the beds has been the need to dry for long periods during winter months. Typically, sludge has been hauled by tanker trucks to nearby lagoons during high solids loading periods or during winter months when the drying beds cannot keep up. The city has planned construction of more drying beds to handle all sludge production peaks.

Geotex Tubes

Geotextile (geotex) fabric is defined as a type of cloth or synthetic material placed between the soil and a pipe, gabion, or retaining wall to enhance water movement and retard soil movement, and as a blanket to add reinforcement and separation. A geotex tube, as its name implies, is a geotextile fabric shaped into a tube. The tubes are made from high-strength, woven polypropylene and polyester fabrics. Traditional uses of the tubes have been for dewatering dredge materials, wetland reclamation, beach nourishment, and offshore breakers. Typical tube circumferences are 15 to 30 ft (4.6 to 9.2 m) with lengths of 50 to 300 ft (15.3 to 91.8 m).

The tubes operate by pumping the material to be dewatered into the tubes. The water passes through the membrane, and the solids are retained. The material can be left in the tubes to air dry. The polyester tubes are almost completely resistant to overpumping and rupture. Drying occurs over a long period of time. The polypropylene tubes are black and use solar heat for surface drying and can thus decrease the overall drying time required.

There are no literature reports of geotex tubes being used in the potable water industry, but manufacturers of the materials are making them available for such use.

Freeze–Thaw Beds

It has long been recognized that when residuals are subjected to freezing, the resulting volume reduction and increased solids concentration is appreciable. Typically, the volume reduction is greater than 70 percent, and solids concentrations may reach as high as 80 percent when freeze–thaw is followed by evaporation. Freeze–thaw followed by evaporation dramatically converts the residuals from a fine particle suspension to granular particles. The granular particles often resemble coffee grounds in both size and appearance, and they do not break apart even after vigorous agitation. If the frozen mixture is placed on a porous medium, the water drains away easily upon thawing (Martel and Diener 1991). Freeze–thaw beds are operated most effectively in northern climates, with a range of effective operation beginning at approximately 40° north latitude and extending northward. (The 40° north latitude runs horizontally across the United States, roughly through Philadelphia [Pa.], Indianapolis [Ind.], and Salt Lake City [Utah]).

Some WTPs in cold climates already take advantage of this process by modifying the operations of their lagoons or drying beds. One technique is to decant a lagoon down to the residuals interface and allow it to freeze during the winter months. Martel and Diener (1991) report that this technique is not always successful because the residuals do not freeze to the bottom. Core samples taken of one lagoon indicated that very few residuals were frozen by this technique; instead, the core consisted mostly of clear ice, and the solids were pushed downward into the lagoon. Another technique is to pump a shallow layer (20 to 45 cm) of residuals from a storage lagoon into drying beds or ponds that are then allowed to freeze in the winter. This technique works well because the residuals usually freeze completely, but it requires a considerable amount of land and storage volume.

Combination sand drying beds and freeze–thaw beds can also be utilized. In this case, the design must consider the evaporative condition for the drying bed cycle and the freezing and thawing conditions for the freeze–thaw cycle.

Combining the concepts presented in the literature and observations of residuals freezing operations on drying beds and lagoons, a unit operation called a residuals freezing bed was developed (Martel 1989). To maximize residuals dewatering by natural freeze–thaw, a freezing bed includes the following features:

- It is designed to apply residuals in several thin layers rather than a single thick layer. Each layer is applied as soon as the previous layer has frozen, thereby maximizing the total depth of residuals that can be applied.
- The bed is covered to prevent snow and rain from entering. This feature is critical if the bed is to utilize all of the available freezing time in the winter. An open freezing bed would have less capacity because snow accumulations on the surface would slow down the freezing rate. Also, snow removal would be practically impossible if a large snowfall occurred soon after residuals were applied. In this case, the operator would have to delay snow removal until the frozen residuals were thick enough to support snow removal equipment. A covering would also prevent rainfall from rewetting the thawed residuals.
- The sides of the bed housing are left open to allow free air circulation. However, a half-wall or louvered wall is recommended to prevent drifting snow from entering the bed. Also, the roof is made to be transparent so that solar radiation can help thaw and

> dry the residuals in the spring. Incoming solar radiation in the winter is expected to be negligible because of the sun's low azimuth and the likelihood of snow on the roof.

Essentially, a freezing bed consists of an inground containment structure that is waterproofed to prevent groundwater infiltration. A ramp is provided at one end to allow vehicle access for residuals removal and to distribute the incoming residuals evenly within the bed. The opposite end of the bed is equipped with an overflow gate or drain valves to draw off supernatant during thaw. The bottom of the bed is underdrained with wedge-wire screen or sand to allow drainage of the filtrate. Both overflow and filtrate are collected in a sump and pumped back to the plant (Martel 1989).

The freezing bed must be complemented with appropriate residuals storage in a separate lagoon or tankage.

Martel (1989) has developed a mathematical model to develop design criteria for residuals freezing beds. The model calculates the depth of residuals that can be frozen and thawed naturally for a proposed site. Martel indicates that freezing occurs in two phases. During phase 1, liquid residuals will be cooled to the freezing point; during phase 2, liquid residuals at the freezing point will be converted to a frozen solid by loss of the latent heat of fusion. Further cooling of the layer below the freezing point should not be significant because the operational plan should call for immediate application of the next layer as soon as the previous layer has frozen. He concluded that phase 1 cooling time was small relative to the total cooling and freezing time and that for design purposes cooling time could be eliminated from freezing time predictions without serious error.

The model developed by Martel for phase 2 freezing predicts the time needed to freeze a layer of specified thickness. It also provides a comparison model for predicting the time required to thaw residuals of a specified thickness that can be used to determine a rational design for freezing beds.

Dempsey, DeWolfe, and McGeorge (1993) examined eight WTP residuals for their content of trace metals, pH, TOC, total phosphorus (P), total Kjeldahl nitrogen (TKN) concentrations, dithionite-citrate-bicarbonate extractable aluminum (Al) concentration, and sequential fractionation of metals. Five residuals in both the wet and freeze–thaw dewatered forms were selected for combination with two soils. These mixtures were examined in a variety of soils tests. Results were compared with respect to the form of the residual (wet or freeze–thaw) and, where appropriate, to results for the soils or residuals alone. In addition, the soil–residuals mixtures were packed into columns and eluted with a calcium nitrate and sodium nitrate solution.

Several conclusions were drawn by Dempsey, DeWolfe, and McGeorge (1993) regarding wet versus freeze–thaw dewatered residuals. First, the freeze–thaw dewatering process resulted in a statistically significant decrease in TKN concentration versus the wet residual, but a similar decrease was not observed for P. Second, the freeze–thaw dewatering process did not result in a decrease in extractable Al. Third, a significant decrease in plant-available P was observed for soil mixed with residuals compared to the soil alone. Furthermore, the decrease in P was greater for the soil–wet residuals mixture compared to the soil mixed with the freeze–thaw dewatered residuals.

Original supernatants and freezates (i.e., liquid solution after a freeze–thaw cycle) were tested for organic parameters, including TOC, instantaneous THMs, and THMFP. After freezing and thawing, the values for all parameters were affected. As expected, TTHM values were greatly reduced for all the freezates because of the volatile nature of the TTHM constituents. The exposure to air, possibly combined with the effects of the latent heat of solidification, likely caused these reductions. TOC values for all freezates increased, as did the THMFP values for two of the three freezates.

Dempsey, DeWolfe, and McGeorge (1993) found that the concentrations of the metals in the supernatants of residuals were much less than 1 percent of the total concentrations, except for manganese. This indicated that the metals in the raw residuals were predominately associated with the solid phases. There was an increase in the concentrations for some of the metals in the freezates (for all three residuals) that may be attributed to the freeze–thaw process, indicating a release from the solid to the liquid phase. However, after 3 weeks, the metals data revealed that most values returned to near original supernatant levels. Aluminum concentrations in the freezate were not higher than the raw residual supernatant, but the concentrations of lead, copper, and zinc in the freezates were typically greater than in the raw residual supernatants.

Vandermeyden and Cornwell (1998) conducted laboratory freeze–thaw tests on several residuals samples. The objectives of these tests were to quantify the volume and solids reduction levels achievable through the freeze–thaw processes and to investigate if any similarities existed among different sludge types. It was not the intent to simulate the actual freezing process encountered in the field with respect to soil and snow insulation, temperatures, and wind chill. All samples were completely frozen in the laboratory, such as could easily be done by a utility to test the impact of freezing on its residuals.

All tests were conducted on 200-mL samples. The 200-mL residuals samples were placed in disposal 500-mL plastic beakers and frozen for 48 hours at –15°C in a laboratory-type freezer. When the samples were completely frozen, they were removed from the freezer and allowed to thaw at room temperature. During the thawing process, the supernatant above these solids was decanted and measured. The remaining solids were gravity filtered through a Whatman filter paper to simulate drainage with underdrains. The supernatant and drainage volumes were added to determine the overall volume reduction achieved. Drained solids concentrations on the remaining solids were then analyzed to determine the increase in solids concentration. Therefore, the drained solids concentrations achieved would approximate freeze–thaw periods prior to evaporative drying and are analogous to the drainage cycle of a sand drying bed.

Table 4.11 shows the results of the laboratory freeze–thaw tests. The volume reduction due to the freeze–thaw process, followed by draining and decanting, was typically 80 to 90 percent by volume. These residuals are representative of the solids remaining immediately after the thawing process is completed and water has been removed by decanting and draining. Subsequent air drying of the remaining solids would be required in order to achieve the desired final solids concentration.

Two field freeze–thaw tests were also conducted by the researchers. One was performed in Edmonton, Alb., Canada, and the other at the Sturgeon Point WTP located near Buffalo, N.Y. In Buffalo, PACl residuals were tested, and in Edmonton, lime residuals were used.

The City of Edmonton evaluated the use of freeze–thaw lagoons as an alternative to mechanical dewatering. Two 100-ft^2 (9 m^2) freeze–thaw lagoons were excavated for this pilot test, which lasted from the beginning of winter in November through the spring thaw in April. The lagoons were approximately 5 ft (1.5 m) deep and had a ratio of 1:1 side slopes. The side slopes were covered with 10-mil-thick plastic during the pilot test to prevent interference of wall effects. The bottom of the lagoon was natural soil consisting of fairly sandy material that could drain free water as necessary.

Prior to the start of the pilot-plant test, the theoretical freezing depth was calculated with the use of a model developed by Martel (1989). The model predicted a maximum freezing depth

Table 4.11
Laboratory freeze–thaw results

Sludge type	Location	Initial solids volume (mL)	Final solids volume (mL)	Volume reduction (%)	Initial solids concentration (%)	Thawed and drained concentration (%)
Alum	Durham, N.C.	200	24.0	88.0	3.0	24.8
	Hunstville, Ala.	200	29.0	85.5	3.2	31.8
	Bridgeport, Conn.	200	36.0	82.0	2.6	22.2
PACl	Buffalo, N.Y.	200	13.0	93.5	1.5	29.3
	Blacksburg, Va.	200	27.0	86.5	4.0	28.3
	Albany, N.Y.	200	29.0	85.5	1.4	14.3
Lime	St. Louis, Mo.	200	18.0	91.0	3.0	47.5
	Taylorville, Ill.	200	71.0	64.5	15.0	36.5
	Midland, Mich.	200	22.0	89.0	6.0	46.4
	Findlay, Ohio	200	52.0	74.0	10.0	38.6
Ferric	Winchester, Va.	200	28.0	86.0	3.9	57.3
	Boulder City, Nev.	200	38.0	81.0	2.7	24.5
	Indiana, Pa.	200	8.0	96.0	0.9	35.1

Source: Vandermeyden and Cornwell 1998.

of 80 in. (200 cm) if the residuals were applied in 6-in. (15-cm) layers, whereas a onetime bulk application would freeze to approximately 43 in. (110 cm). The city staff believed that a onetime bulk loading would be the preferred method of operation in a full-scale operation because it would eliminate the need for residuals storage. Based on the theoretical freezing depth of 43 in. (110 cm), the two pilot freeze–thaw beds were loaded, one at 36-in. (90-cm) depth and one at 48-in. (120-cm) depth.

The residuals applied to the lagoons were obtained from the plant's gravity thickener and had a total solids concentration of 25 percent. The total solids volume applied to each lagoon was 3,700 gal (1.4×10^4 L) and 5,700 gal (2.2×10^4 L) for the 36- and 48-in. (90- and 120-cm) solids depths, respectively. At the end of the 5-month period, there remained approximately 27 in. (68 cm) of residuals in the lagoon loaded at 36 in. (90 cm), and 36 in. (90 cm) of residuals in the lagoon loaded at 48 in. (120 cm). This resulted in a total volume reduction of 33 to 34 percent for both lagoons. There was no free water present in the lagoons, and no decanting was performed at any time. Consequently, all the water from the residuals had drained through the lagoon bottom. Samples collected in the shallower lagoon at 2-, 15-, and 27-in. (5-, 38-, and 68-cm) depths had total solids concentrations of 58, 55, and 55 percent, respectively. Samples collected in the deeper lagoon at 2-, 18-, and 36-in. (5-, 46-, and 90-cm) depths had total solids concentrations of 57, 54, and 54 percent, respectively. It appeared that both lagoons had frozen over the entire depth, as the 55 to 60 percent solids concentration achieved was comparable to the solids concentration expected for freeze–thawed lime solids as previously shown with the laboratory tests in Table 4.11. Thus, the model appeared to be quite good in predicting the freeze depth.

Table 4.12
Erie County Water Authority—Pilot freeze–thaw testing conditions

Bed no.	Loading (lb/ft^2)	Polymer dose (lb/ton)	Initial applied depth (in.)	Drained depth after 21 days (in.)	Initial solids concentration (%)	Initial volume applied (gal)
1	19.2	0.6	36	12	5.3	6,239
2	20.5	0.0	36	15	5.6	6,239
3	10.0	0.0	18	7	6.3	2,323
4	9.4	0.6	18	7	6.3	2,559
5	21.3	0.0	30	13	7.5	4,875
6	26.3	0.3	30	14	9.3	4,875

NOTES: $lb/ft^2 \times 4.8824 = kg/m^2$; lb/ton × 0.5 = g/kg; in. × 2.54 = cm; gal × 3.785 = L.

Six freeze–thaw beds were loaded at the Sturgeon Point WTP owned and operated by the Erie County Water Authority (ECWA). Three beds were loaded without polymer at solids loading rates of 10 to 21.3 lb/ft^2 (50 to 10^7 kg/m^2). This corresponded to residuals depths of 18 to 36 in. (46 to 90 cm). Based on Martel's model, the calculated freezing depth in Buffalo for a onetime loading is 9 in. (23 cm). It was thought that the applied depths used would be in the 9-in. (23-cm) range after drainage occurred. Three other beds were loaded at comparable depths but with the addition of polymer. Table 4.12 summarizes the initial loading conditions for the six freeze–thaw beds.

The lagoons were equipped with underdrains, which allowed the removal of drainage from the residuals shortly after the application was completed. Typically, a volume reduction of 60 to 75 percent was achieved. As shown in Table 4.12, the residuals depth remaining after 21 days was between 7 and 15 in. (18 and 38 cm), compared to a predicted freezing depth of 9 in. (23 cm). No freezing conditions occurred during these initial 21 days, and, therefore, the volume reduction was mostly attributable to the drainage of the residuals.

When the freeze period is modeled using Martel's freeze–thaw model (Martel 1989), an estimated freezing depth of at least 8 in. (20 cm) should be achieved. Composite samples of the residuals on the bed had a solids concentration of 35.5 to 45.1 percent, as shown in Table 4.13. Table 4.11 previously showed that laboratory freeze–thaw and drainage for this facility had about a 30 percent solids concentration.

It can thus be concluded that the residuals probably froze completely and that the model did a fairly good job in predicting the freeze depth, and may have underestimated the freeze depth. The field solids concentrations were slightly higher than the lab values, probably because evaporation was also taking place in the field.

The area required for a freeze–thaw bed is generally determined by the depth of residuals that can be frozen. For some climates with long freezing periods, the depth that can be thawed can be the controlling depth. Martel (1989) developed equations to allow the calculation of these two

Table 4.13
Erie County Water Authority—Pilot freeze–thaw data

Bed no.	Drained depth (in.)	Predicted freezing depth (in.)	Depth remaining* (in.)	Composite solids remaining (%)
1	12	9	7	43.7
2	15	9	4	45.1
3	7	9	4	42.6
4	7	9	4	48.4
5	13	9	4	35.5
6	14	9	4	37.8

*Remaining depth after 118 days.

depths. (NOTE: In some very cold climates the time for thawing could be limited, and the Martel reference should be consulted to determine the thaw time.) The freezing depth can be found by

$$D(z) = \frac{t(f)(\mathrm{T_f} - T)}{\rho_f F\left(\frac{1}{\mathrm{h}} - \frac{d(z)}{2\mathrm{K}}\right)} \tag{4.26}$$

where $D(z)$ = total depth of sludge that can be frozen (m)
$t(f)$ = the freezing time (hr)
$\mathrm{T_f}$ = freezing point temperature = 0°C
T = average ambient temperature (°C)
ρ_f = density of frozen sludge = 917 kg/m^3
F = latent heat of fusion = 93 W·hr/kg
h = convection coefficient = 7.5 W/m^2 °C
$d(z)$ = the thickness of sludge layer (m)
K = conductivity coefficient = 2.21 W/m °C

Because many of these parameters are known or assumed constants, the equation can be reduced to

$$D(z) = \frac{-t(f)T}{11{,}371 + 19{,}294(d(z))} \tag{4.27}$$

Equation 4.27 would be used when the design calls for multiple layers of sludge to be frozen, with each layer of thickness being *d(z)*. In this case, the utility personnel would apply the layer to the bed, and as soon as one layer had frozen, another would be applied. According to Martel (1989), this application method will increase the total depth of sludge that can be frozen as compared to a onetime bed loading. He reports that the minimum practical depth for *d(z)* is 0.08 m, or about 3 in.

The values for $t(f)$ and T can be attained (for U.S. locations) from the National Oceanic & Atmospheric Administration through records that are generally on file at the National Climate Data Center in Asheville, N.C.

For the case of a onetime bed loading, $D(z)$ would be set equal to $d(z)$. Solving this equation requires use of the quadratic rule, which results in the following expression as determined by Vandermeyden and Cornwell (1998):

$$D(z)\frac{-11{,}371-\sqrt{1.3\times 10^{8}-7.7\times 10^{4}Tt(f)}}{3.9\times 10^{4}} \tag{4.28}$$

In the data presented for freeze–thaw results for ECWA, the number of freeze hours available for the study period was 1,512 hours. The average temperature during the period was –2.1°C. Therefore, the freeze depth using the layer method with a 0.08-m depth for each freeze layer would be found by using Martel’s equation:

$$D(z) = \frac{-(-2.1)(1{,}512)}{11{,}371 + 19{,}294(0.08)} = 0.25 \text{ m or } 9.7 \text{ in.} \tag{4.29}$$

Therefore, about three layers of sludge, each 0.08 m (3 in.) thick, could be frozen. If a onetime loading was used, then the freeze depth would be found by using the following equation:

$$D(z) = \frac{-11{,}371+\sqrt{1.3\times 10^{8}-7.7\times 10^{4}(1{,}512)(-2.1)}}{3.9\times 10^{4}} \tag{4.30}$$

$$D(z) = 0.2 \text{ m or } 8 \text{ in.} \tag{4.31}$$

Therefore, under these climatic conditions, the onetime freeze depth is nearly the same (8 in. [20 cm]) as the layer method (9 in. [23 cm]), and there would be no particular advantage in using the more operator-intensive layered method.

The design elements and operational considerations for freezing beds are extensively covered by Martel (1989) and Cornwell and Koppers (1990). The basis of design for a freezing bed allows shallow layers of residuals to be applied to a bed. Each layer should be allowed to freeze prior to application of a subsequent layer on top. Martel (1989) demonstrated that this procedure maximizes the total cumulative freezing depth of the residuals and thereby minimizes the required freezing bed area and cost. Some daily operator attention would be necessary to ensure that the layer has frozen completely and to apply the next residuals layer.

Another consideration in the design of layered freezing beds is the basis of design with respect to the residuals quantity. The two approaches that could be considered are as follows:

1. Design the layered freezing beds only for the residuals generated during the winter season.
2. Design the layered freezing beds to process the residuals generated during the entire year. This would require storage of the residuals generated during the spring, summer, and fall.

If the residuals must be stored from the spring through the fall prior to being applied to the freezing beds, design considerations should include carefully sizing the residuals storage facilities and evaluating how to pump these residuals to the beds after 9 months of storage.

The beds could be designed to operate as layered freezing beds during the winter months and as sand drying beds during the spring, summer, and fall. A sizing analysis of both the freeze–thaw and sand drying beds would have to be performed to determine which scenario requires the most area. This type of design would eliminate the need for extensive residuals storage that would be necessary for processing all the residuals through a layered freeze–thaw method.

Another viable approach for sizing freezing beds would be to calculate the freezing and thawing depth of only a single bulk loading. This type of design would be similar to a sand drying bed or a dewatering lagoon, except in the winter months when the residuals on the bed are subjected to a freeze–thaw treatment cycle. Although freezing a onetime bulk loading is not as efficient as multiple shallow layers, it would provide for a less operator-intensive operation, and the combined sand drying bed and freeze–thaw bed would eliminate storage facilities.

Regardless of which freeze–thaw design approach is selected, each design should incorporate a sand bottom, underdrains, an inlet with energy dissipation devices, and an effective decant system. Examples previously discussed for sand drying beds could be considered for freeze–thaw beds as well. Polymer addition should be considered to maximize the removal of water from the residuals prior to freezing. This would reduce the residuals depth to be frozen and would allow a higher solids loading rate and depth. After the freeze–thaw cycle, the freezate should be decanted as soon as possible. The remaining residuals must be allowed to air dry in order to achieve their typical coffee-grounds texture. With freeze–thaw treatment, the air drying process is usually much faster than for residuals that have not experienced freeze–thaw treatment.

Cleaning of the freeze–thaw beds could be completed with front-end loaders provided that runways and ramps are installed similar to those in sand drying beds.

REFERENCES

Albertson, G.C. 1995. Optimization of a Diaphragm Filter Press. In *Proc. of the 4th Joint WEF and AWWA Conference: Biosolids and Residuals Management*. Kansas City, Mo.: Water Environment Federation.

ASCE and AWWA (American Society of Civil Engineers and American Water Works Association). 1990. *Water Treatment Plant Design*. New York: McGraw-Hill.

ASCE, AWWA, and USEPA (American Society of Civil Engineers, American Water Works Association, and U.S. Environmental Protection Agency). 1996. *Technology Transfer Handbook: Management of Water Treatment Plant Residuals*. Denver, Colo.: AWWA.

AWWA (American Water Works Association). 1999. *Water Quality and Treatment: A Handbook of Community Water Supplies*. 5th ed. New York: McGraw-Hill.

Cornwell, D.A., and H. Koppers, eds. 1990. *Slib, Schlamm, Sludge*. Denver, Colo.: AwwaRF, AWWA, and KIWA.

Cornwell, D.A., and R. Lee. 1993. *Recycle Stream Effects on Water Treatment*. Denver, Colo.: AwwaRF and AWWA.

Dempsey, B.A., J. DeWolfe, and L. McGeorge. 1993. Evaluation of Chemical and Physical Characteristics of Water Treatment Residuals Before and After Freeze–Thaw Cycle. New Jersey DEPE Report. University Park, Pa.: Pennsylvania State University.

Eckenfelder, W.W., and D.L. Ford. 1970. *Water Pollution Control*. Austin, Texas: Jenkins Publishing.

Harbin, C. 1995. Optimization of a Centrifuge for the Dewatering of a Water Treatment Plant's Residuals. In *Proc. of the 4th Joint WEF and AWWA Conference: Biosolids and Residuals Management*. Kansas City, Mo.: Water Environment Federation.

Innocenti, P. 1988. Techniques for Handling Water Treatment Sludge. *Opflow*, 14(2):1.

Martel, C.J. 1989. Development and Design of Sludge Freezing Beds. *Jour. Environmental Engineering, ASCE*, 115(4):799–808.

Martel, C.J., and C.J. Diener. 1991. A Pilot-Scale Study of Alum Sludge Dewatering in a Freezing Bed. *Jour. AWWA*, 83(12):51–55.

Rolan, A.T. 1980. Determination of Design Loading for Sand Drying Beds. In *Jour. North Carolina Section AWWA and North Carolina Jour. Water Pollution Control Federation*, L5(1):25.

Vandermeyden, C., and D.A. Cornwell. 1998. *Nonmechanical Dewatering of Water Plant Residuals*. Denver, Colo.: AwwaRF and AWWA.

Vesilind, P.A. 1979. *Treatment and Disposal of Wastewater Sludges*. Ann Arbor, Mich.: Ann Arbor Science.

CHAPTER 5
COAGULATION AND LIME SOFTENING RESIDUALS DISPOSAL AND BENEFICIAL USE

Several traditional as well as innovative methods are available for ultimate disposal or utilization of coagulant and lime-softening residuals. Through the 1970s, the most widely practiced method for handling these residuals was direct discharge, without treatment, to a receiving stream or water body. In 1970, there were no mechanical sludge dewatering devices in the United States, indicative of the few options for sludge management being utilized. The 1990 version of *Water Quality and Treatment* (AWWA) presented lagoons as the only method other than direct discharge that was available to handle these residuals. In 1989, the first joint AWWA–WEF (Water Environment Federation) conference on residuals was held. At that conference there were no papers presented on utilities using beneficial application methods. However, by the 2000 version of the same conference, there were entire sessions dedicated to residuals beneficial use. The available disposal and beneficial use options have exploded since the 1990s. In 2000, Cornwell, Mutter, and Vandermeyden reported (from the AWWA database) the following residuals handling options being used, by percent of reporting utilities:

Stream discharge	11 percent
Sanitary sewer	24 percent
Monofill	13 percent
Landfill	20 percent
Land application	25 percent
Other	7 percent

In the sections that follow, the many disposal and beneficial use options available to utilities are discussed, with the full understanding that many more options will likely be developed.

DISCHARGE TO SURFACE WATER

Regulatory

The Water Quality Act of 1965 (PL 89-234; FWPCA, 1965) required states to set standards for interstate waters and gave them authority to order treatment of wastes from WTPs. Some grant money was provided by this law for constructing water treatment projects, but most water treatment projects were assigned a low priority and little attention was given to the operating performance of such plants.

With the passage of the Federal Water Pollution Control Act amendments, commonly known as the Clean Water Act (PL 92-500; FWPCA 1977) and the establishment of USEPA, a more formal procedure for controlling WTP discharges was established. Water supply was formally declared as an industry. The implications of this decision were twofold:

1. Construction grant monies offered to public wastewater plants were not available to WTPs even if they were publicly held.
2. A procedure for promulgating guidelines for discharges from WTPs was established.

The guidance document for the water supply industry divided WTPs into three categories:

1. Plants that use one of the following: coagulation, oxidation for iron and manganese removal, or direct filtration
2. Plants that use chemical softening procedures
3. Plants that use a combination of the procedures listed in the first two categories

For each category, the best practical control technology was defined and allowable pH and TSS limitations were established. The limits established for water plant discharges ranged from 5 to 10.8 lb of solids per million gallons (0.6 to 1.3 mg/L) of water treated depending on plant capacity. Larger plants were held to lower solids discharge levels. This guidance document, however, did not progress beyond the draft phase. It also did not address liquid phase waste discharges.

USEPA has historically not established plans to publish a guidance document on water plant wastes. Therefore, discharge decisions are made either by the regional USEPA offices or by the individual states delegated to write their own permits. It is up to the permit writer to rule on the best available treatment technology for each plant on a case-by-case basis. According to USEPA, the primary criterion for the allowance of direct discharge is to meet established instream water quality standards at the edge of the mixing zone:

> In developing technology based limitations in permits, a controlled release of water clarifier sludge and filter backwash from water treatment plants in a manner which meets water quality standards may in appropriate circumstances be constituted to be technology based controls (Grubbs 1985).

Two key phrases are *controlled release* and *meets water quality standards*.

In 2004, USEPA began the process of developing effluent limitation guidelines for WTP residuals. It is expected that these regulations will be available in the 2007 time frame, and at that time federal guidelines should exist for governing direct discharge.

Water Quality Standards

Instream water quality criteria and standards are developed by individual states (with the use of some federal guidelines). Most states have classified each body of water for a designated use and set instream quality guidelines appropriately. Table 5.1 shows example instream water quality criteria and standards for several selected compounds. (Because standards vary from state to state, only examples are illustrated. The specific agency involved should be contacted.) In addition to meeting instream water quality standards, some states have established maximum allowable concentrations in the discharge. These limits generally apply if they are more stringent than the allowable discharge that will meet the instream water quality criteria. For example, Illinois does not allow a discharge of greater than 15 mg/L fluoride (F). Barium discharge is required to be less than 2 mg/L, even if the 1 mg/L instream standard could be met through dilution. Wisconsin has set maximum discharge levels of radium (soluble) in liquid wastes as follows:

$$\frac{\text{Ra-226}}{30} + \frac{\text{Ra-228}}{30} \leq 1 \text{ pCi/L} \tag{5.1}$$

Table 5.1
Example instream water quality guidelines and standards

	Guidelines— Aquatic life chronic criteria			Example standards
	Fresh water (µg/L)*	Salt water (µg/L)*	Human health (µg/L)*	Stream used for potable water (mg/L)†
Arsenic (dissolved)	72	63	2.2 ng/L	0.05
Barium	NS‡	NS	NS	1.0
Beryllium	130	NS	3.7 ng/L	NS
Cadmium	$e^{1.16(\ln(\text{hardness}))-3.841}$	12	10.0	0.01
Chloride	NS	NS	NS	250
Chromium (hexavalent, dissolved)	7.2	54	NS	NS
(trivalent, active, total)	$e^{0.819(\ln(\text{hardness}))+.537}$	NS	170	NS
(total)	NS	NS	NS	0.05
Copper	2.0	4 (24 hours) 23 (anytime)	NS	1.0
Cyanide (free)	4.2	0.57	20.0	NS
Fluoride	NS	NS	NS	1.4
Hydrogen sulfide	2.0	2.0	NS	NS
Iron (total)	1,000	NS	NS	NS
(soluble)	NS	NS	NS	0.3
Lead	$e^{1.34(\ln(\text{hardness}))-5.245}$	8.6	50	0.05
Manganese (total)	NS	100	NS	NS
(soluble)	NS	NS	NS	0.05
Mercury	0.00057	0.1	146 ng/L	0.002
Nickel (total)	$e^{0.76(\ln(\text{hardness}))+1.06}$	7.1	13.4	NS
Nitrate (as N)	NS	NS	NS	10
Phenol	1.0	1.0	3,500	0.001
Selenium	35	54	10	0.01
Silver	$0.1e^{1.72(\ln(\text{hardness}))-6.52}$	0.023	50	0.05
Sulfate	NS	NS	NS	250
TDS	NS	NS	NS	500
Zinc	47	58	5,000	5.0
Aldrin	0.03	0.003	0.074 ng/L	NS
Chloride	0.0043	0.004	0.46 ng/L	NS
Endrin	0.0023	0.0023	1.0	0.0002
Heptachlor	0.0038	0.0036	0.28 ng/L	NS
Lindane	0.08	0.0016	NS	0.004
Methoxychlor	0.03	0.03	NS	0.10
Toxaphene	0.013	0.0007	0.71 ng/L	0.005
DDT	0.001	0.001	0.024 ng/L	0.1
Chloroform	1,240	NS	0.19	NS
Radioactivity Ra-226+228	NS	NS	NS	5 pCi/L
Gross alpha particle activity (excluding radon/uranium)	NS	NS	NS	15 pCi/L

*Units are in micrograms per liter unless otherwise indicated.

†Units are in milligrams per liter unless otherwise indicated.

‡NS = no standard.

These regulations apply to discharge from a water plant to a storm sewer or to a surface body of water. Several states have set limits on suspended solids levels and chlorine residual in the discharge.

Discharges to U.S. Waters

The direct discharge of water plant wastes (i.e., discharge of untreated waste streams) into receiving bodies of water is no longer a widespread disposal method. Most plants treat the waste for solids removal prior to discharge. However, for discharges into large rivers containing a large suspended solids mass, direct discharge has been found to be acceptable and is still practiced in some areas of the country.

To assess the effects or determine the regulatory compliance of a discharge, it is often necessary to calculate the instream concentration of the pollutant of interest. This is especially true for meeting instream water quality standards. The following equations can be used to estimate these instream concentrations or changes to instream concentration. Caution should be exercised to distinguish between dissolved and nondissolved pollutant concentrations.

If the water supply stream is the same as the water plant waste discharge stream, as shown in Figure 5.1, then the following flux relationships exist for the river (*F*(1)) and the WTP (*F*(2)) (AWWA 1987):

$$F(1) = \frac{d(w_1)}{dt} = Q_1C_1 + Q_WC_W - QC_1 - Q_2C_2 \tag{5.2}$$

$$F(2) = \frac{d(w_2)}{dt} = QC_1 + d(A) - Q_WC_w - (Q - Q_W)C_F \tag{5.3}$$

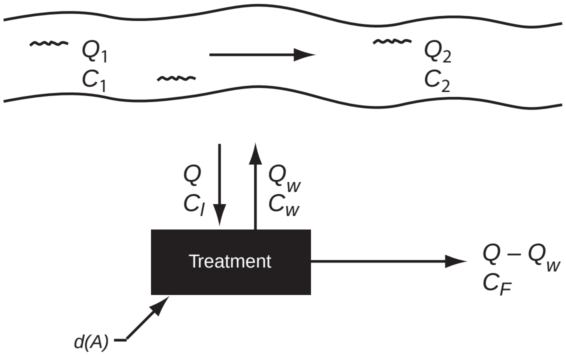
At steady state, and combining the two expressions, results in

$$Q_2C_2 = Q_1C_1 + d(A) - (Q - Q_W)C_F \tag{5.4}$$

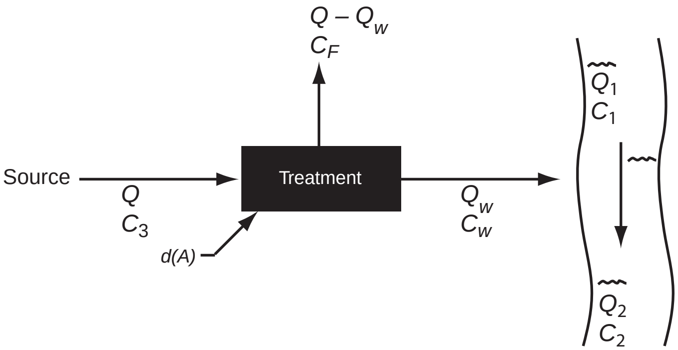
where Q_1,Q_2 = flow in river upstream (1) or downstream (2) of the water plant intake and the sludge discharge
Q = flow into WTP
Q_W = flow of waste into the river
C_1,C_2 = respective pollutant concentrations upstream and downstream of the plant
C_W = concentration of pollutant in the waste stream
C_F = concentration of pollutant in the treated water
$d(A)$ = change in pollutant due to chemical treatment, mass/time
w = mass pollutant concentration
F = flux change, mass concentration with time

or if the net change in mass is desired, then

$$W = Q_2C_2 - Q_1C_1 = d(A) - (Q - Q_W)\ C_F \tag{5.5}$$



(a) Common Source



(b) Different Source

Figure 5.1 Mass balance for discharge of residuals receiving streams

If the discharge stream is not the same as the supply source, as shown in Figure 5.1, then the change in receiving stream concentration upstream and downstream of the discharge is

$$Q_2C_2 - Q_1C_1 = QC_3 - (Q - Q_W)\, C_F + d(A) \qquad (5.6)$$

where C_3 = concentration of pollutant in the supply source

The change in pollutant due to treatment, $d(A)$, is generally the chemical additions (C_A). For IX systems, the chemical additions would be the salt used for regeneration, which is known. For RO plants, chemical addition is minimal. For coagulation and softening plants, chemical additions are often reported as concentration with respect to the influent water plant flow.

$$d(A) = (dC_A)Q \qquad (5.7)$$

$d(A)$ would need to be found for the pollutant of interest. Some common examples are shown below:

Suspended solids:	$d(C_A)$ = 0.44 (alum) + PAC + polymer
Total aluminum (as Al):	$d(C_A)$ = 0.09 (alum)
Total iron (as Fe):	$d(C_A)$ = 1.0 (iron)
Total calcium (as $CaCO_3$):	$d(C_A)$ = 2.0 (Ca removed)

These equations can be used to determine compliance with the instream water quality requirements established by the state.

Benthic Impacts

If the receiving stream or lake has a low velocity and the discharged waste contains settleable solids, the residual discharge will tend to collect in a sludge deposit in the vicinity of the point of discharge. While little work has been published on the benthic impacts of water plant waste deposits, it is likely that the ecosystem in the immediate vicinity of the sludge blanket will be impacted. In addition to the impact of the presence of the blanket itself, anaerobic conditions may develop, resulting in an increase in solubilization of metals—both from within the sludge and the bottom soil, a lowering of the pH, and a release of odors. A related research effort was conducted by Lamb and Bailey (1981) for alum residuals. Their interest was on the effects of using alum to precipitate phosphorus in highly eutrophic lakes. The objective of their study was to determine the acute and chronic effects of aluminum hydroxide to *Tanytarsus dissimilis. T. dissimilis* is a representative of the chironomidae, the family of organisms that occupies a significant portion of the benthic invertebrate community of lakes and an important fish food organism. The acute tests were conducted by adding concentrations of alum to beakers at pH 7.8 of up to 960 mg/L (960 mg/L alum is equivalent to approximately 400 mg/L SS). The results of the acute bioassays indicated no apparent effects.

Mortalities in the chronic assays, however, were recorded at all alum doses. The results show a general increasing of mortality with an increase in alum sludge (aluminum hydroxide) although discrepancies within this conclusion existed. The time to reach 50 percent mortality was shortest at the 480-mg/L alum dose at about 4 days. The mortality time for 80 mg/L and 240 mg/L was not statistically different at about 9 days. The 10-mg/L dose showed 37 percent mortality at 55 days. The major discrepancy was the 960-mg/L dose, which had a 50 percent mortality time of 23 days. The research concluded that there was some chemical toxicity at alum concentrations of 80, 240, and 480 mg/L and at least a stressed condition developed at 960 mg/L. They did not believe that the solid aluminum hydroxide itself was toxic, but rather that some of the species, perhaps the negatively charged aluminates, tended to concentrate toxic materials. They also felt that a heavy floc layer on a lake bottom could inhibit pupae from reaching the surface and deposited eggs from reaching the sediment.

Roberts and Diaz (1985) conducted studies to quantify the effects of alum residuals discharge into a tidal stream in Newport News, Va. The streams studied were shallow with a low velocity. During their work, phytoplankton productivity was measured during an alum residuals discharge event. The authors concluded that a negative impact was associated with the alum residuals discharge. They explained the depression of production by the high suspended solids associated with the discharge. During sampling, light intensity was observed to decline virtually to zero at a depth of only a few millimeters. Thus the authors concluded, “this turbidity effect in itself would argue for the cessation of residuals discharge even in the absence of toxic effects.” Net production of the benthic algae community was also negative during sampling periods, reflecting the low light intensity.

Onsite field investigations were conducted at four locations (Cincinnati, Ohio; Griffin, Ga.; Mobile, Ala.; and Concord, Calif.) to evaluate the effect of waste alum sludges on aquatic macroinvertebrates in receiving water sediments (George et al. 1991).

The macroinvertebrates in the receiving water sediments at all of the utilities appeared to be stressed by the alum sludge discharge. In the Ohio River, the effect was temporal, depending on the transport of the sludge by increased flows during storm events. In regions with slow water velocity, the sludge may settle and cover the sediments. As a result, macroinvertebrate populations may be stressed because of lack of oxygen and carbon sources (primary production) to feed on. Once the sludge blanket is removed, the organisms repopulate the sediments. Based on the macroinvertebrate diversity and density at each site, the organisms appeared stressed by waste alum sludge and other environmental factors not evaluated.

Special Concern Regarding Aluminum: Aluminum Toxicity

Aluminum toxicity itself remains as one of the major concerns regarding the effects of alum residuals discharge. Because aluminum is a ubiquitous element, the question of its toxicity is more difficult to resolve than that of the heavy metals (for which toxicity data were given in Table 5.1). Thus, although the toxicity of soluble aluminum has been demonstrated for a wide spectrum of plants and animals, the overwhelming percentage of aluminum in the world is without measured toxic effect. Therefore, one must be able to distinguish among the forms of aluminum. The chemistry of aluminum in water is essentially that of aluminum hydroxide. It is characterized by being readily amphoteric and forms complex ions with other substances in water and polymerizes. When an aluminum salt of noncomplexing characteristic is placed in water, it undergoes the basic hydrolysis reactions:

$$\begin{gathered}
\mathrm{Al\ (H_2O)^{3+}{}_6 + H_2O = Al\ (H_2O)_5\ OH^{2+} + H_3O^+} \\
\mathrm{Al\ (H_2O)_5\ OH^{2+} + H_2O = Al\ (H_2O)_4\ (OH)^+ + H_3O^+} \\
\mathrm{Al\ (H_2O)_4\ (OH)_2{}^+ + H_2O = Al\ (H_2O)_3\ (OH)_3 + H_3O^+}
\end{gathered} \tag{5.8}$$

and as the pH increases further, the aluminate ion is formed:

$$\mathrm{Al\ (H_2O)_3\ (OH)_3 + H_2O = Al\ (H_2O)_2\ (OH)_4{}^- + H_3O^+} \tag{5.9}$$

which is the apparent limit of hydrolysis as the –2 and –3 species have not been reported. Given equilibrium constants for the reactions in Equations 5.8 and 5.9, one can then plot the solubility of aluminum in the presence of solid aluminum hydroxide as a function of pH. Often, plots of aluminum in equilibrium with crystalline aluminum hydroxide (Al_2O_3) are reported. However, the solubility of aluminum in the presence of colloidal aluminum hydroxide such as would exist in sludges is higher. At pH 5 the equilibrium concentration of soluble aluminum is about 0.135 mg/L in the presence of the solids species of $Al\ (H_2O)_3(OH)_3$. As the solids crystallize further to Al_2O_3 (gibbsite), the solubility decreases. This crystallization to gibbsite may take a year or more. During this time, the pH will drop slightly as bound hydroxide becomes intimately incorporated with the crystal structure. Thus an organism in contact with freshly prepared sludge will be exposed to different soluble aluminum concentrations and different degrees of polymerized or crystallized solid species than an aged sludge. This, of course, further complicates the evaluation of alum sludge toxicity.

Aluminum is also capable of forming strong coordinate bonds with substances other than water. Complexes are formed with inorganic ligands such as fluoride and silicates. Insofar as

Table 5.2
Estimated survival times of brook trout (*Salvelinus fontinalis*, Mitchill)*

pH	Ligands added	Total aluminum concentration (mg/L)	Percent survival after 14 days		Estimated 50 percent survival time (hr)	
			Mean	Range	Mean	Confidence limits
5.1	None	0.02	99	95–100	>500	NA†
	None	0.42	28	20–30	115	88–150
	Fluoride	0.50	45	35–45	202	136–302
	Citrate	~0.5	96	87–100	>500	NA
4.4	None	0.01	95	80–100	>500	NA
	None	0.48	42	27–53	256	213–306
	Fluoride	0.47	71	67–80	472	354–629
	Citrate	~0.5	82	40–100	>500	NA

Source: Adapted from Driscoll et al. 1980.
*Water solution made from synthetic soft water.
†NA = not applicable.

complexing agents are present in water, they will decrease the amount of dissolved aluminum in equilibrium with solid aluminum hydroxide. However, in natural waters, the concentrations of these inorganic ligands is usually not high enough to appreciably effect aluminum solubility. Many organic materials of natural origin are capable of forming dissolved complexes with aluminum such as humic and fulvic acids, polyphenols, sugars, and organic acids. All of these change the form of aluminum present and its solubility.

The following discussion presents some of the data on aluminum toxicity that may be pertinent to alum sludge discharge. Toxicity is discussed in terms of free aluminum ion, solid aluminum hydroxide (which has also been discussed with respect to benthic impacts), aluminate, and soluble aluminum complexes.

Schofield and Trojnar (1980) indicated that levels of aluminum may be the primary factor limiting survival of trout in the Adirondack lakes. They report brook trout mortality at aluminum levels of 0.2 to 0.5 mg/L. However, they did not distinguish between the types of aluminum species present. Driscoll et al. (1980) conducted studies on the toxicity effects of different species of aluminum. They utilized synthetically prepared waters as well as waters obtained from the Adirondack lakes. Brook trout fry were exposed to aluminum as the free ion in soft, synthetic water and complexed with hydroxide, fluoride, or citrate. The results are shown in Table 5.2. Survival of brook trout fry was significantly inhibited at total aluminum concentrations of >0.4 mg/L without the presence of complexing ligands. Fluoride somewhat increased survival, and the addition of citrate significantly increased survival. Driscoll and colleagues (1980) concluded that the inorganic aluminum forms seem to be the major species of concern in direct aluminum toxicity. They then conducted studies on white sucker fry, using natural waters and synthetic solutions.

Driscoll et al. (1980) estimated the 50 percent survival time as a function of aluminum concentration reported as free aluminum ion and total aluminum. A significant decrease in the

Table 5.3
Chronic aluminum intoxication and recovery for trout

	pH 7			pH 8			pH 8.5		pH 9	
	Control	0.52	5.2	Control	0.52	5.2	Control	5.2	Control	5.2
Exposure time (days)	45	45	45	45	45	45	9.25	9.25	4.7	4.7
Weight change during bioassay (%)	182	–3	–26	+192	+76	–18	+50	–32	+84	0
Mortality during bioassay (%)	2	44	58	0	8	77	0	51	0	67
Recovery time (days)	290	290	290	16	16	1	161	161	165	165
Weight gain during recovery relative to control weight gain (%)	100	164	195	100	101	NA*	100	159	100	141
Mortality during recovery (%)	3	5	16	NA	0	83	5	3	2	2

Source: Adapted from Freeman and Everhart 1971, Freeman 1973.
*NA = not applicable.

50 percent survival time occurred near 0.2 mg/L aluminum ion concentration in both the synthetic and natural waters.

Freeman and Everhart (1971) performed a partial chronic bioassay for aluminum using the growth rate of rainbow trout in continuous flow aquariums. They used waters containing 0.05, 0.52, and 5.2 mg/L total aluminum at various pH ranges (Table 5.3). Because of the pH utilized, they primarily studied the effects of aluminum hydroxide and aluminate. Feeding activity diminished within 24 hours, and gill hyperplasia (a swollen, congested condition) was evident within 5 days in many of the trout. The physical condition of all fish continued to deteriorate, with individuals suffering from inability to maintain equilibrium, general listlessness, loss of fright reaction, loss of negative phototaxis, darkening in coloration, and eventual death. Few of the trout that were exposed for the full 45 days survived, even after transfer to uncontaminated water. When exposed only to dissolved aluminum (0.52 mg/L) at the same pH, fish exhibited milder forms of the symptoms described previously, with slower development of hyperplasia and loss of appetite. Mortality was greatly reduced, recovery was rapid and almost complete within 48 hours of transfer to uncontaminated water, and normal weight gain resumed. An aluminum level of 0.05 mg/L had no apparent effect. At pH 8.5, 32 percent of 5.2 mg/L aluminum is dissolved and at pH 9, 10 percent is dissolved. The previously mentioned severe symptoms appeared in fish exposed to these conditions, but they appeared more rapidly. In addition, fecal casts were evident. Exposure was terminated after 222 hours and 113 hours at the two pH conditions, respectively. On transfer of surviving trout to clean water, recovery was rapid in the first case and delayed, but apparently complete, in the second.

Hall et al. (1985) conducted instream studies on the episodic effect of aluminum addition. They artificially added aluminum to streams and monitored biological changes.

Table 5.4
Summary of aluminum toxicity test results*

Sample site pH	Microtox (bacteria)	*Tetrahymena* (protozoa)	*Selenastrum* (algae)	*Ceriodaphnia*	*Pimephales*
Cincinnati, Ohio					
5	0	0	–	0	0
7.4	0	0	0	0	0
8	0	0	+	0	0
Contra Costa, Calif.					
5	0	0	–	0	0
7.5	0	0	–	0	0
8.5	0	0	–	0	0
Evansville, Ind.					
5	0	0	–	NA	NA
7.6	0	0	0	NA	NA
8.5	0	0	–	NA	NA
Omaha, Neb.					
6	NA	0	–	NA	NA
8.3	NA	0	–	NA	NA
9	NA	0	0	NA	NA
Pasco, Wash.					
5	0	0	0	0	0
7.5	0	0	0	0	0
8.5	0	0	–	0	0
Paducah, Ky.					
5	–	0	–	NA	NA
7	0	0	–	NA	NA
8	0	0	–	NA	NA
Griffin, Ga.					
5	–	0	–	NA	NA
6.9	0	0	–	NA	NA
8	0	+	–	NA	NA
Mobile, Ala.					
6.7	0	0	–	NA	NA
8	0	0	–	NA	NA
9	–	0	–	NA	NA
Cookeville, Tenn.					
5	0	0	–	0	0
7.1	0	0	+	0	0
8.5	0	+	+	0	0
Calgary, Alb., Canada					
5	0	0	–	NA	NA
7	0	0	+	NA	NA
8	0	0	–	NA	NA

Source: George et al. 1991.

*0 indicates no inhibition; – indicates inhibitory effect; + indicates stimulatory effect; NA = not analyzed.

To summarize Hall and colleague's conclusions, these field and laboratory results suggest that episodic increases in aluminum can have significant biological and physical as well as chemical consequences in dilute, acidic surface waters. Macrobenthic community structure (e.g., distribution, abundance, and diversity) and function (e.g., trophic interactions and nutrient cycling) may be disrupted by episodic addition of soil-derived aluminum to streams that become acidified during snowmelt. Moreover, the reduction of surface tension in streams or lakes receiving Al may represent a consequence of acidification that could affect aquatic biota and even some terrestrial forms by physically interfering with emergence, egg deposition, feeding, and mating behavior.

George et al. (1991) conducted the most exhaustive study directly on the impacts of alum sludges relative to aluminum toxicity. They evaluated sludges from WTPs on several bioassay species, as shown in Table 5.4. The sludges were subjected to different pH conditions to promote aluminum solubilization.

The algal bioassay using *Selenastrum capricornutum* demonstrated the greatest response to exposure of the alum extracts. No inhibition was observed in either of the chronic toxicity tests or the protozoan acute toxicity test.

Microtox Acute Toxicity Test. At pH 5, sludge extracts from both Paducah, Ky. (15-min EC_{50} = 81 percent sludge extract), and Griffin, Ga. (15-min EC_{50} = 54.8 percent sludge extract; 5-min EC_{50} = 61.9 percent), demonstrated toxicity using the Microtox assay. EC_{50} is the concentration of a test sample causing a 50 percent decrease in light output, and was used as the relative indicator of toxicity. The combination of low pH stress and aluminum species exposure probably caused the toxicity. Extracts obtained from the sludge collected at the Mobile (Ala.) Water Treatment Plant had a 15-min EC_{50} equal to 85 percent of the filtered sludge extract at pH 9. The Mobile sludge extracts contained the highest concentration of total Al and monomerric Al at pH 9. The predominant aluminum species at pH 9 most likely was the aluminate ion. The Microtox test showed no toxic effects to any of the remaining alum sludge extracts.

Protozoan Acute Toxicity Test. None of the sludge extracts produced a toxic response in the protozoan mortality test at either 24 or 48 hours. No decreases in cell numbers were observed in controls or extract-amended treatments over the 48-hour period. Sludge extracts obtained from Cookeville, Tenn., and Griffin, Ga., promoted growth (i.e., cell division) at pH 8. The increased volume of sludge extract in the higher concentrations possibly contributed nutrients to the cells.

Ceriodaphnia *Chronic Toxicity Test.* No toxicity was observed at any of the sludge extract concentrations tested. Mortalities were less than or equal to 20 percent, and young production was acceptable in all sludge concentrations. Analysis of reproduction data revealed no significant difference between Cookeville controls and each sludge concentration. For the three remaining sites, however, control reproduction was too low to provide useful information.

Fathead Minnow Chronic Toxicity Test. Very little mortality in the fathead minnow (*Pimephales*) test populations occurred at each sludge extract concentration for each of the water treatment locations. Average survivorship of minnows in all extract concentrations was well above the 80 percent level required for controls.

Summary and Conclusions. *S. capricornutum* was the only organism that was adversely affected by the alum sludge extracts. This indicates the importance of conducting toxicity tests using a battery of organisms representing various taxa that are important in the aquatic ecosystem. Generally, the aluminum sludge extracts at pH 5 from all 10 locations were the most toxic to algae. The low pH in conjunction with the more soluble aluminum present acted to inhibit growth. The algal growth response for most sampling locations reflected the aluminum chemistry; that is, toxicity was observed at the lowest and highest pH but not a neutral pH.

Aluminum appeared to be directly toxic to algae as opposed to indirectly inhibitory to algae due to phosphate removal. The hardness and the alkalinity of the receiving waters decreased the toxicity in a manner consistent with the protection provided by these parameters against heavy metal toxicity.

Based on the results of this study, the following conclusions were made:

- *S. capricornutum* was the only organism adversely affected by alum sludge extracts.
- Microtox, the protozoan mortality test, and the fathead minnow test showed no adverse response to alum sludge extracts.
- Generally, alum sludge extracts at pH 5 from filtered sludge obtained from all 10 WTPs were most toxic to algae.
- Algal growth response apparently reflected aluminum chemistry, with toxicity occurring at the lowest and highest pH but not at neutral pH.
- Harder receiving water decreased algal toxicity.
- Toxicity testing should employ a battery of test organisms representing the various taxa important in the aquatic system.

Toxicity of Nonaluminum Compounds

Most inorganic ions other than aluminum that could be discharged to a stream as part of a solid waste or a liquid phase waste have established chronic mortality levels and are regulated by state agencies. In Table 5.1, mortality levels can be found for such ions as iron, arsenic, barium, fluoride, nitrate, TDS, and radioactivity. Table 5.1, in conjunction with the mass balance calculations, should be used to assess the discharge toxicity of specific ions.

DISCHARGE TO WASTEWATER TREATMENT PLANTS

Almost unanimously, WTP operators would be happy to discharge their waste to the wastewater plant. In fact, disposal of water plant wastes to the wastewater plant is widely practiced and can be done very successfully. On the other hand, there have been failures with this method.

This section primarily discusses the effects of coagulation process wastes discharged to the wastewater plant. However, many liquid waste streams, including liquids resulting from the dewatering of solid–liquid wastes, are discharged to the sewer. Some of this information is useful for whatever type of waste is being discharged to the sewer.

Regulatory

Prior to the discharging of WTP residuals or any other substance to the sanitary sewer, it must, at a minimum, be classified as nonhazardous by the generator. This is determined using the TCLP test described in chapter 2.

Discharges to the sanitary sewer are subject to USEPA's national pretreatment standards (USEPA 1981) and any additional stringent pretreatment requirements set by the state or local wastewater treatment facility. The requirements imposed by a wastewater treatment facility through a permit, local ordinance, or both, are necessary to enable the facility to achieve compliance with their National Pollutant Discharge Elimination System (NPDES) permit. Pretreatment standards are typically site specific.

Equalization

Sludge from sedimentation basins can be withdrawn on a fairly continuous and uniform basis if the basins are equipped with automatic sludge removal mechanisms. In this case, it may be possible to directly discharge the sludge into the sewer system. Often, however, basins are cleaned on a discontinuous basis and EQ is required prior to discharge. SFBW is produced at very high flow rates for short periods of time, and EQ is often required prior to sewer discharge. Regeneration wastes from IX processes are produced only during the time of media regeneration and EQ may be needed, although the volumes are generally small.

Certain restrictions may be placed on the discharge by the publicly owned treatment works (POTW), such as time of day of discharge, instantaneous flow, maximum flow over a certain period of time, or maximum solids discharge. Figure 5.2 shows an example mass storage diagram approach used for a water plant that had a maximum discharge limitation of solids. During peak solids production periods, storage was required. This particular storage volume was based on storing an average of 2 percent solids concentration. If a higher solids concentration is achieved, then a smaller storage is required. This trade-off usually results in the construction of a basin capable of decanting and thickening. Either a continuous flow or batch fill-and-draw thickener could be used, such as discussed in chapter 4. It may be desirable to have the capability to mix the contents of the EQ/thickener tank, and this should be considered in the design phase. EQ facilities for backwash water or similar high-flow, short-duration productions prior to sewer discharge are similar to backwash EQ facilities discussed in chapter 3.

Consideration should also be given to time of discharge. It may be desirable to discharge during periods of the day when sufficient flow is in the sewer to maintain desired velocities. On the other hand, at certain times the sewer may flow full and a sludge discharge is undesirable. Generally, a velocity of about 2.5 fps should be maintained to prevent sedimentation of hydroxide sludge solids. Lime sludge may have settling velocities much higher than coagulant sludges, and it can be difficult to prevent its deposition in sewer lines. For the discharge of compounds toxic to the biological process, it may be necessary to equalize flows to allow for a continuous discharge or proper dilution.

The City of Cleveland’s (Ohio) Morgan WTP evaluated EQ needed for its residuals based on a tons-per-day sewer limitation. Historical daily operational data were used in order to calculate sludge production using an empirical sludge production equation. The empirical formula used to determine sludge production is

$$S = 8.34\, Q\, (0.44\ \mathrm{Al} + \mathrm{bTu} + A) \tag{5.10}$$

where S = sludge produced (lb/d)
Q = plant flow (mgd)
Al = dry alum dose (mg/L) (17.1 percent Al_2O_3)
b = ratio of TSS and turbidity
Tu = raw water turbidity (ntu)
A = additional chemicals such as polymer, PAC (mg/L)

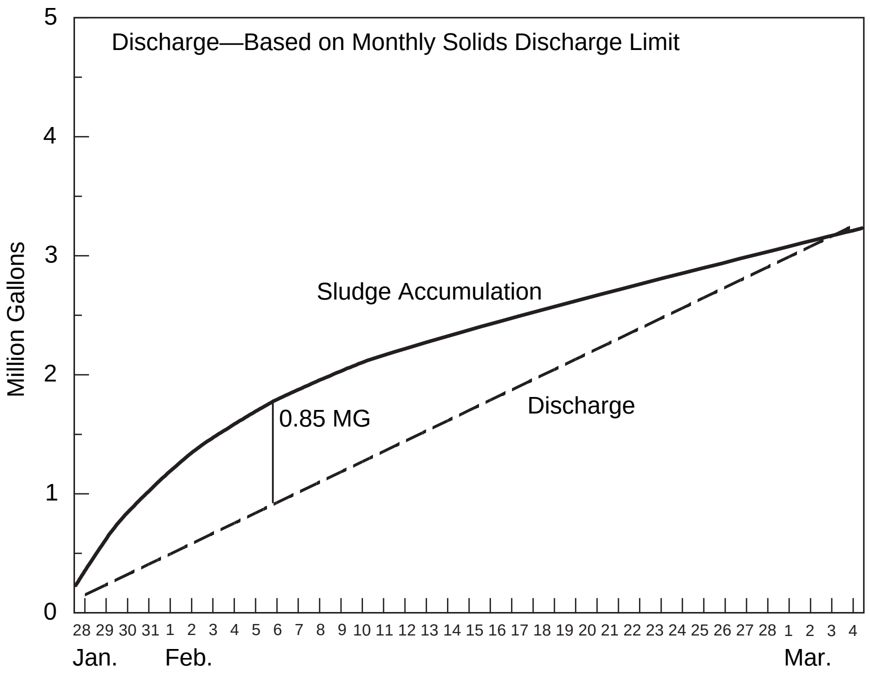


Figure 5.2 Example sludge EQ for sewer discharge

A percentile curve was developed for sludge production quantities in terms of dry tons per day (DT/d). The percentile curve (Figure 5.3) indicated the following calculated sludge production amounts:

50th percentile = 5 DT/d (4,500 kg/d)
90th percentile = 12.4 DT/d (11,200 kg/d)
95th percentile = 17 DT/d (15,000 kg/d)
99th percentile = 30 DT/d (27,000 kg/d)

These data suggest that during periods of peak sludge production (>90th percentile production), the dry solids produced would exceed the allowable sewer discharge release, which is 12.5 DT/d (11,300 kg/d). This peak production period accounts for 10 percent of the days over the 3 years analyzed. To accommodate the increase in sludge volume during peak periods, the Morgan WTP residuals handling facility would need to be able to store the additional amount of alum residuals that could not be released to the sewer.

The tank volume required to store peak residuals at a 2 percent solids concentration was determined by calculating the cumulative sludge production generated in excess of the 12.5 DT/d (11,300 kg/d) sewer discharge limited, presented in Figure 5.4. Based on the historical data, the maximum cumulative amount of sludge that would need to be stored at 2 percent solids concentration is 80 DT (73,000 kg), or 920,000 gal (3.5×10^6 L).

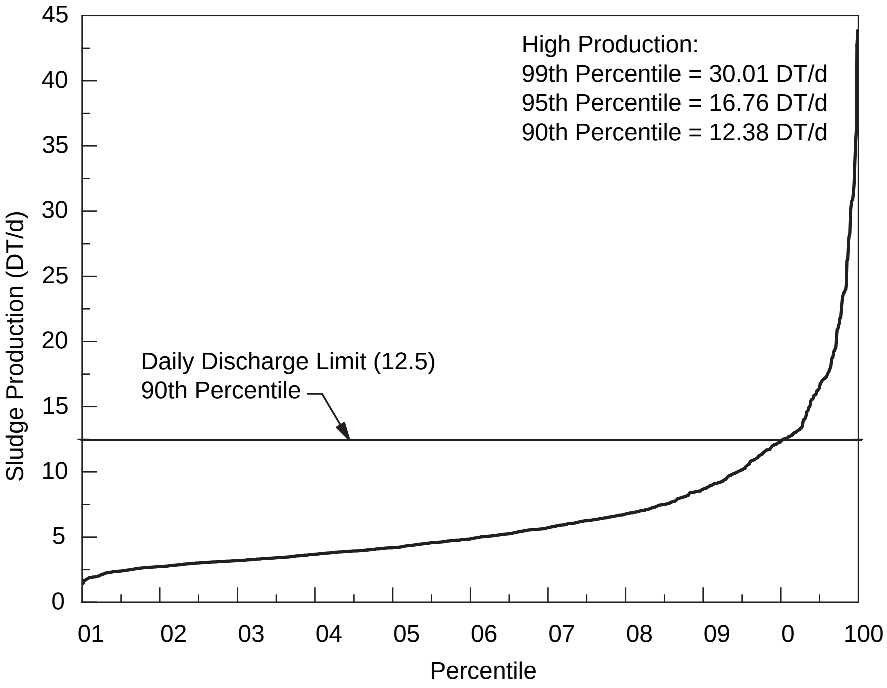


Figure 5.3 Cleveland's Morgan WTP historical sludge production

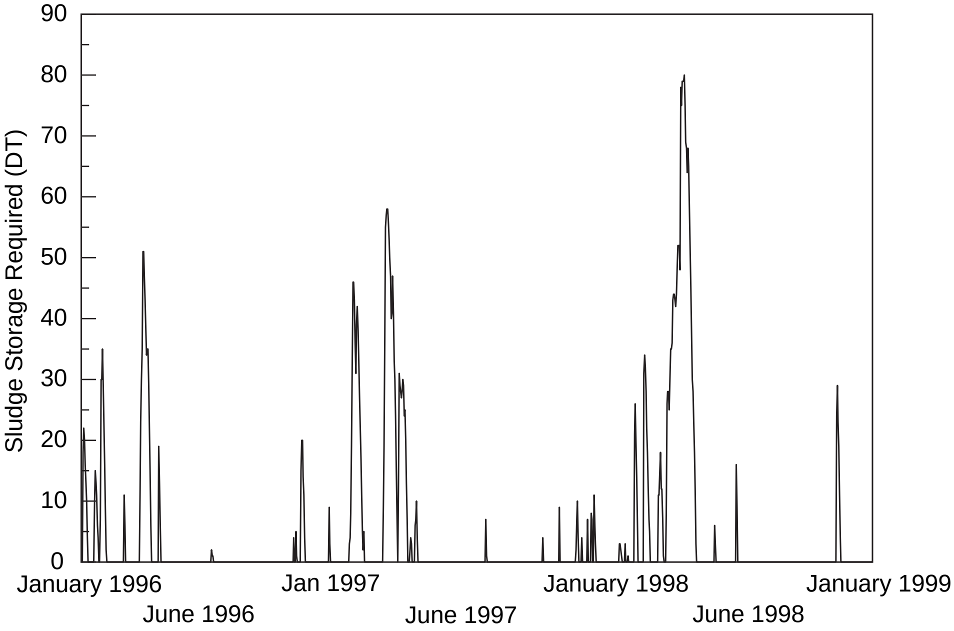


Figure 5.4 Cleveland's Morgan WTP cumulative sludge storage required for sewer discharge

Effects on Biological Processes

Possible effects, either beneficial or detrimental, of water plant wastes on the biological wastewater process are in the areas of toxicity to the biological processes, suspended solids removal or increases, BOD/COD removal or increases, hydrogen sulfide removal, and phosphorus removal. Dissolved solids present in the waste could be available in some form and present in a sufficient concentration to hinder the biological process. Defining the toxic effects of inorganic compounds on the biological wastewater treatment process is not a simple procedure. An initial shock load of a toxic compound can have an inhibitory effect on the biological process. However, with many compounds the microorganisms will adapt and adjust to the presence of the inorganic ion. Therefore, even if a pretreatment standard is being met, it is a good rule of thumb to equalize the discharge according to sewer flow patterns in order to provide a fairly uniform concentration of waste to the biological process.

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. Table 5.5 shows data compiled by Srinath (1978) via a literature review on the threshold concentration for various metals in the activated sludge process. The metals listed in Table 5.5 can be present in liquid waste streams from water plants. However, several other compounds can also be present for which little data on the toxicity to the biological process are available. Crites and Tchobanoglous (1998) also reported the threshold concentration of reported inhibitory effect on heterotrophic organisms, as shown in Table 5.6. Depending upon the type of WTP residual being discharged and the sewer dilution factor, some of these levels could be of concern.

Parkin and Owen (1986) reported inhibitory metals of concern for anaerobic processes, as shown in Table 5.7. It is unlikely that traditional water plant residuals, diluted in a sewer, would reach the levels of concern shown in Table 5.7.

One of the more comprehensive studies on the effects to biological processes related to alum residuals was conducted by Rolan and Brown (1974) in laboratory studies followed later by full-scale work. Rolan and Brown added various concentrations of alum residuals to the raw wastewater and observed the impact on several performance criteria. An increase in alum residuals had a slight detrimental effect on the supernatant suspended solids quality resulting from primary sedimentation. Both turbidity and suspended solids increased by 25 to 40 percent as alum residuals were added at up to 200 mg/L of dry weight solids. Figure 5.5 shows the results for settled suspended solids concentrations from primary clarification. On the other hand, as Figure 5.6 shows, the settled BOD decreased slightly as the alum sludge dose was increased, possibly because of some adsorption of BOD onto the alum floc particles.

The work did find that the addition of alum residuals would lower the phosphorus content of the primary effluent. Figure 5.7 shows that about 1 mg/L of phosphorus was removed by 100 mg/L of alum residuals, probably by sorption onto the aluminum hydroxide matrix. Rolan and Brown (1974) conducted tests on the effects of residuals aging that showed a definite decrease in phosphorus adsorption capacity with an increase in residuals age from 6 hours to 2 months. It was concluded that while some phosphorus removal by the addition of alum residuals occurred, the removal itself was relatively insignificant as a treatment technology.

In later work by Rolan and Matthews (1975), full-scale testing was conducted on the effects of alum residuals on the previously discussed process parameters. Alum residuals were added to a 7-mgd wastewater flow on a daily basis for about 60 days. The alum residuals were measured at a dose that increased the raw wastewater suspended solids concentration by

Table 5.5
Threshold concentration for various metals in the air activated sludge process

Metal ion	Concentration (mg/L)	Type activated sludge experiment
Silver	<0.3	Carbonaceous
Vanadium	10.0	Carbonaceous
Zinc	2.0	Carbonaceous
	5–10	Carbonaceous
	1.0	Nitrification
Nickel	1.0–2.5	Carbonaceous
	1.0	Nitrification
	2.0	Nitrification
Chromium^{+6}	10.0	Carbonaceous
	1.0	Nitrification
	10.0	Nitrification
	1.0	Nitrification
Chromium^{+3}	10.0	Carbonaceous
Lead	10	Carbonaceous
Iron (ferric)	15	Carbonaceous
Copper	1.0–10.0	Carbonaceous
	1.0	Nitrification
	2.0	Nitrification
Cadmium	1.0	Carbonaceous
	5.0	Nitrification

Source: Adapted from Srinath 1978.

100 mg/L. No effects on plant-finished BOD were found. However, final effluent suspended solids values increased from 20 mg/L to 40 mg/L. After stopping alum residuals discharge, suspended solids values returned immediately to the 20-mg/L range. Final effluent color also increased from about 45–50 Pt–Co (platinum–cobalt) units to 70–90 Pt–Co units. On the other hand, total phosphorus in the effluent decreased from 7 mg/L without alum residuals addition to 4 mg/L with the addition.

Salotto, Farrell, and Dean (1973) reported the results of a 200-gpd pilot-plant investigation into the effects of water plant residuals on the activated sludge process. The water plant sludge was from a coagulation/softening process. The sludge contained 27 percent dry weight calcium, or about 67 percent $CaCO_3$ and only 0.75 percent Al, or less than 4 percent aluminum hydroxide species. Therefore, the residuals could be considered lime-softening residuals for comparison purposes with the results of Rolan. The activated sludge process was continually dosed with 200 mg/L dry weight solids of lime residuals. Although the pH of the residuals was 10, the raw wastewater pH was not significantly affected. The pH is one important parameter that would need to be closely monitored when discharging lime residuals to the sewer. No change in COD removal from influent to final effluent was found. Overall, the suspended solids removed was slightly better with the lime residuals addition, although the primary effluent suspended solids was

Table 5.6
Metal inhibitory to heterotrophic organisms

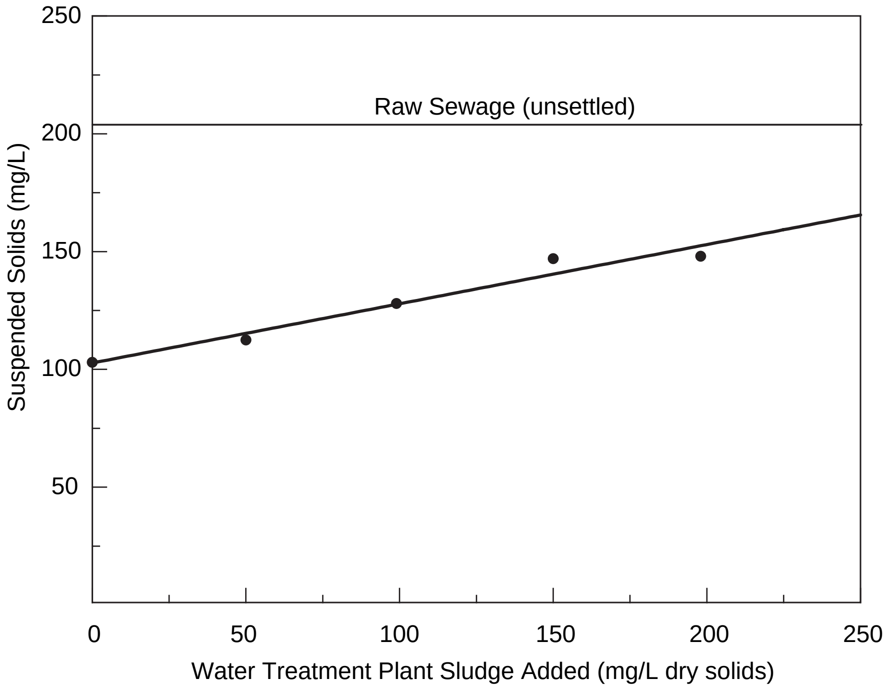
Metal	Symbol	Concentration threshold of inhibitory effect on heterotrophic organisms (mg/L)
Arsenic	As	0.05
Cadmium	Cd	1.0
Chromium	Cr	10*, 1†
Copper	Cu	1.0
Lead	Pb	0.1
Mercury	Hg	0.1
Molybdenum	Mo	ND‡
Nickel	Ni	1.0
Selenium	Se	ND
Zinc	Zn	1.0

Source: Adapted from Crites and Tchobanoglous 1998.
*Total chromium.
†Hexavalent chromium.
‡ND = no data.

Table 5.7
Metals of toxic concern for anaerobic processes

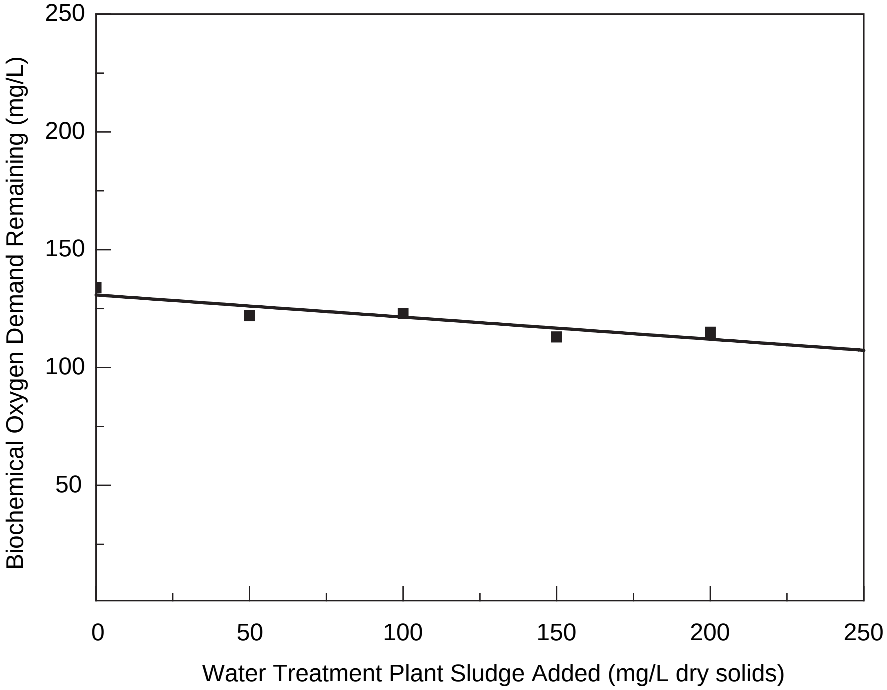
Substance	Moderately inhibitory concentration (mg/L)	Strongly inhibitory concentration (mg/L)
Na^+	3,500–5,500	8,000
K^+	2,500–4,500	12,000
Ca^{2+}	2,500–4,500	8,000
Mg^{2+}	1,000–1,500	3,000
Ammonia-nitrogen NH_4^+	1,500–3,000	3,000
Sulfide, S^{2-}	200	200
Copper, Cu^{2+}	ND*	0.5 (soluble) 50–70 (total)
Chromium, Cr(VI)	ND	3.0 (soluble) 200–250 (total)
Chromium, Cr(III)	ND	2.0 (soluble) 180–420 (total)
Nickel, Ni^{2+}	ND	30.0 (total)
Zinc, Zn^{2+}	ND	1.0 (soluble)

Source: Adapted from Parkin and Owen 1986.
*ND = no data.



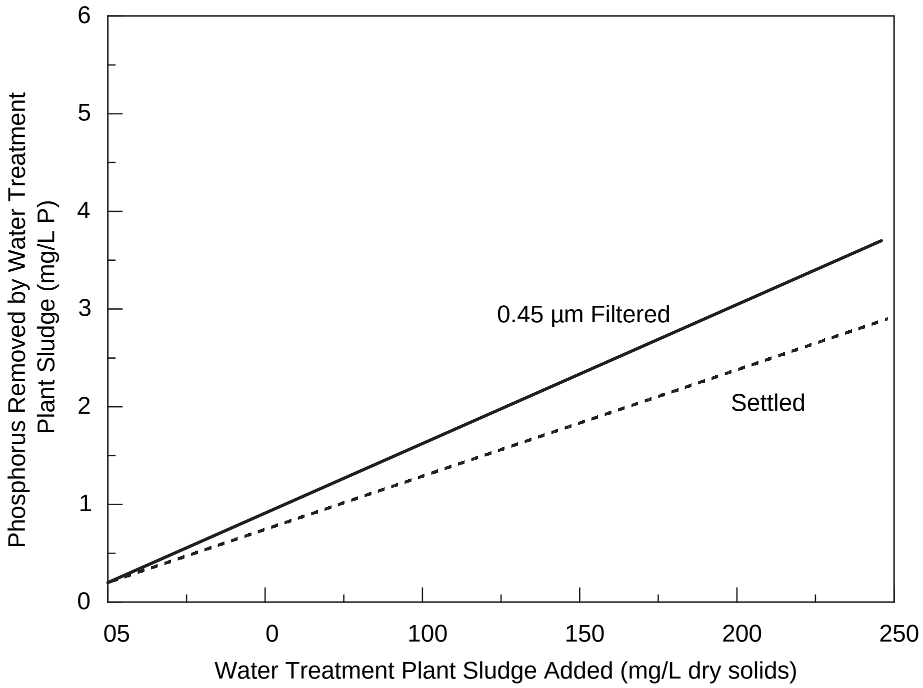
Source: Adapted from Rolan and Brown 1974.

Figure 5.5 Water treatment plant sludge impact on settled wastewater suspended solids



Source: Adapted from Rolan and Brown 1974.

Figure 5.6 Water treatment plant sludge impact on settled wastewater BOD



Source: Adapted from Rolan and Brown 1974.

Figure 5.7 Phosphorus removed versus WTP sludge added to wastewater influent

slightly worse. The researchers explained this based on a carryover of aluminum and magnesium hydroxide from the primary clarifier, which eventually aided settling in the secondary clarifier. No change in phosphorus removal was found with the addition of 200 mg/L lime sludge.

Overall, it can be concluded that if the dosing of water utility 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. If primary clarifiers are present, most of the water plant solids will be removed at that point. No change in overall BOD/COD or suspended solids removal would be expected but should be monitored. If primary clarifiers are not present, then some adverse impacts may result: the activated sludge process will need to operate with a higher mixed liquor suspended solids concentration to maintain the desired mixed liquor volatile suspended solids concentration, and overloading of the secondary clarifiers may occur, resulting in solids carryover.

With alum (and possibly iron) residuals, some additional phosphorus removal may occur, depending upon the residuals dose. However, as a phosphorus removal process to achieve or help achieve normal phosphorus effluent goals, the residuals are of little or no benefit.

If a water utility waste that contains suspended solids is transferred to the POTW, then additional sludge handling is necessary at the receiving facility. Considerations must be given to the unit processes of clarification, digestion, final dewatering, and disposal.

Rolan and Brown (1974) conducted sludge volume tests to determine the additional primary clarifier sludge volume that would be expected after the addition of water plant alum sludge to raw wastewater. The volume of the settled combined sludges was greater than if they had been settled separately. This increase in combined volume may be due to additional removal of fine solids caused by enmeshment within the water plant residuals. Overall, the addition of 200 mg/L of alum residuals solids approximately doubled the volume of primary clarifier residuals.

Digester sizing and performance must consider the additional volume and dry weight of sludge as well as the decrease in percent volatile matter. In the full-scale tests by Rolan and Brown (1974), raw sludge volume to the digesters increased by about 50 percent with the addition of 100 mg/L alum residuals. Percent solids to the digester increased from about 2.5 to 3.1 percent, whereas digested percent solids decreased from 4.5 to 4.2 percent. The percent volatile matter in the raw sludge decreased from 68 to 55 percent.

The City of Philadelphia evaluated the impact that the discharge of alum sludge to the sewer could have on its anaerobic digesters. It was assumed that the alum sludge did not contain volatile solids as would be anaerobically decomposed. (Note that in a traditional volatile solids test, alum sludges will show a volatile solids content. However, this is generally due to evaporating water of hydration rather than destroying organic matter.)

Within the range of normal digestor loadings, it can be reasonably assumed that the volatile solids destruction will decrease proportional to a decrease in detection time and influent volatile solids percent (WEF and ASCE 1998). Table 5.8 shows the results of the analyses. The field-measured baseline condition was 21,000 lb/d (9,500 kg/d) of alum sludge discharge with a corresponding 51 percent volatile solids reduction. The minimum allowed volatile solids reduction to meet USEPA class B standards is 38 percent, and, therefore, a maximum discharge of about 50,000 lb/d (22,700 kg/d) was established. A similar analysis could be done by a WTP utility working in conjunction with the wastewater plant in order to establish allowable discharge limits.

LANDFILL AND MONOFILL

Regulatory

Although there are no federal laws in place that directly govern the handling and disposal of WTP sludges in landfills, applicable regulations can be found in the following bodies of legislation: the CWA, the RCRA, and the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA). The CWA deals with water quality by placing limits on direct discharge. The RCRA and CERCLA affect land disposal of solid wastes. Additional regulations apply in cases where the sludge contains radioactivity.

No federal mandates are currently in place that would prohibit the disposal of WTP residuals deemed nonhazardous in a municipal solid waste landfill (MSWLF). Disposal of nonhazardous wastes in MSWLFs is governed at the federal level by RCRA regulations. It should be noted, however, that more stringent individual state regulations disallowing disposal of WTP sludge in MSWLFs would override RCRA rules.

The toxicity characteristics used to identify hazardous wastes, those subject to regulation under Subtitle C of the RCRA, were revised in a rule promulgated by USEPA in March of 1990. If a sludge fails the TCLP test, it must be treated as a hazardous waste. Several specific constituents that could be present and potentially leach from WTP sludge are quantified in the TCLP test. The list of measured constituents along with corresponding regulatory threshold levels is shown in Table 5.9. In addition to serving to determine whether a waste is hazardous, the TCLP test is used in regulating many other types of residuals disposal or utilization. As discussed in chapter 2, residuals seldom fail the TCLP test.

Table 5.8
Predicted impact of WTP residuals on Philadelphia anaerobic digesters

WTP residuals discharges (lb/d)	Digester solids loading (lb/d)	Influent volatile solids (%)	Detention time (days)	Estimated volatile solids reduction (%)
21,000*	221,000	73	22.9	51
30,000	230,000	70	22.0	46
40,000	240,000	67	21.0	43
50,000	250,000	65	20.2	40
60,000	260,000	62	19.5	36
90,000	290,000	56	17.5	32

Note: lb/d × 0.4536 = kg/d.
*Field-measured value. All digester performance values associated with higher WTP residuals discharges were theoretically estimated.

The three other indicators of a hazardous material according to the RCRA are ignitability, reactivity, and corrosivity. Hazardous wastes that are capable of causing a fire during transport, storage, or disposal are defined as ignitable. Several properties indicate reactivity in a waste, including violent reaction with water leading to the formation of potentially explosive mixtures. Corrosivity as indicated by pH or the capability to corrode steel is another identifying characteristic used in the RCRA to classify a waste as hazardous. Official lists of designated hazardous wastes have been published by USEPA in the Code of Federal Regulations, Section 40 (40 CFR Part 261, Subpart D; USEPA 1983a).

Although the establishment of governing criteria for the land application of WTP sludges was authorized under the CWA, the landfilling of dewatered residuals produced at WTPs is regulated by the RCRA and other solid waste disposal requirements. The RCRA focuses on the following five elements:

1. Classification of hazardous wastes
2. “Cradle-to-grave” manifest system
3. Standards to be followed by generators, treaters, disposers, and storers of hazardous wastes
4. Enforcement of established standards
5. Authorization of states to obtain primacy for implementation of the regulations

Subtitle C of the RCRA concerns the disposal of hazardous wastes, and Subtitle D establishes a framework in which all levels of government can work cooperatively to effectively control the management of nonhazardous solid wastes. The protection of land, water (both surface and ground supplies), and air from contamination by solid waste was the original goal of the RCRA.

As stated under the RCRA, the basic criteria for determining whether a waste should be classified as hazardous or nonhazardous are ignitability, corrosivity, reactivity, and toxicity. Sludge determined to be hazardous or that which contains polychlorinated biphenyls (PCBs) in

Table 5.9
TCLP contaminants and regulatory levels

Contaminant	Regulatory threshold level (mg/L)
Arsenic	5.0
Barium	100.0
Cadmium	1.0
Chromium	5.0
Lead	5.0
Mercury	0.2
Selenium	1.0
Silver	5.0
Endrin	0.02
Lindane	0.4
Methoxychlor	10.0
Toxaphene	0.5
2,4-D	10.0
2,4,5-TP	1.0
Chlordane	0.03
Heptachlor (and its epoxide)	0.008
Benzene	0.5
Carbon tetrachloride	0.5
Chlorobenzene	100.0
Chloroform	6.0
1,2-Dichloroethane	0.5
1,1-Dichloroethylene	0.7
Methyl ethyl ketone	200.0
PCE	0.7
TCE	0.5
Vinyl chloride	0.2
1,4-Dichlorobenzene	7.5
Hexachlorobenzene	0.13
Hexachlorobutadiene	0.5
2,4-Dinitrotoluene	0.13
Hexachloroethane	3.0
Nitrobenzene	2.0
Pyridine	5.0
o-Cresol	200.0
m-Cresol	200.0
p-Cresol	200.0
Pentachlorophenol	100.0
2,4,5-Trichlorophenol	400.0
2,4,6-Trichlorophenol	2.0

Source: USEPA 1983a.

concentrations >50 ppm is regulated under Subtitle C of the RCRA and the Toxic Substances Control Act (TSCA). The uniform hazardous waste manifest system developed by USEPA makes possible the tracking of hazardous waste from its generation to the point of ultimate disposal. If manifests are correctly processed, the generator of the hazardous waste, who is ultimately responsible for the waste disposal, can reliably track the waste from the "cradle" to the "grave." To ensure that a hazardous waste can be monitored from its generation to its disposal, each RCRA hazardous waste generator must obtain an identification number from USEPA.

Subtitle D of the RCRA governs the management of nonhazardous solid waste, the category under which the bulk of WTP waste falls. USEPA does not have the authority to enforce the criteria through their regulatory programs. In Part 257, "Criteria for Classification of Solid Waste Disposal Facilities and Practices," specific criteria are enumerated and the conditions are described under which violation of these criteria would pose a potential threat of adverse impacts on health or the environment. The following rules regarding floodplains, endangered species, surface water, groundwater, application to land used for the production of food-chain crops, disease, air, and safety must be adhered to in order to ensure that the potential for adverse effects on health and the environment is minimized.

Restricting the flow of the base flood, reducing the storage capacity of the floodplain, or causing a washout of solid waste would constitute a violation of the floodplain criteria established in the regulations. In addition, facilities are prohibited from causing or contributing to the demise of any endangered or threatened species or the critical habitat thereof. Causing a discharge of pollutants that is in violation of the CWA's NPDES or of a federally approved areawide or statewide water quality management plan would be accompanied by a reasonable probability of adverse effects on health or the environment and hence would breach the rules. Contamination of an underground drinking water source beyond the solid waste boundary or an alternative boundary specified by the state would constitute a violation of the rules as well. Contaminants under Part 257 of the CFR include the 10 inorganics, 4 chlorinated hydrocarbons, and 2 chlorophenoxyls that appear in the National Primary Drinking Water Regulations.

To remain in compliance with the regulations, the population of disease vectors onsite must be minimized through the periodic applications of cover material or some equally effective technique. Protection of air quality is ensured through prohibition of open burning of waste. Safety standards that limit explosive gases (methane) and address fires, bird hazards to aircraft, and site access are contained in the regulations as well.

Specific constraints are placed on the application of solid waste to land used for the production of food-chain crops with regard to cadmium and PCBs. In general, minimum pH requirements must be satisfied. The maximum allowable application rate of sludge with respect to cadmium is 0.4 lb/acre (0.5 kg/ha). Maximum cumulative application rates are set according to the pH of background soil coupled with soil cation exchange capacity, with the stipulation that the pH (if <6.5) must be adjusted to and maintained at ≥6.5 wherever food-chain crops are grown. Sludge having PCB concentrations ≥10 mg/kg applied to land used for producing animal feed must be incorporated into the soil, and sludge having a PCB concentration >50 mg/kg must be disposed of in a hazardous waste landfill.

Revisions to "Criteria for Classification of Solid Waste," set forth in final form in 40 CFR Parts 257 and 258 (USEPA 1991), were proposed in response to the 1984 Hazardous and Solid Waste Amendments to the RCRA. Reorganization of Part 257 to exclude MSWLFs resulted in the creation of Part 258. Only MSWLFs that will be receiving waste on or after

October 9, 1993, are subject to all of Part 258 requirements. Although WTP sludge is defined as a solid waste in the regulations, the focus of the rules is the location, design, operation, cleanup, and closure of MSWLFs. However, because disposal of WTP wastes in MSWLFs and construction of sludge monofills are viable ultimate disposal options, and there are no federal regulations in place that directly apply to WTP waste disposal, Parts 257 and 258 currently offer the best federal guidelines.

Siting restrictions regarding wetlands, fault areas, unstable areas, and seismic impact zones are included in Part 258. According to the regulations, placement of an MSWLF in a wetland must not result in "significant degradation" as defined in Section 404(b) of the CWA. A ban on locating new landfills within 200 ft (61 m) of certain faults, along with design specifications to ensure resistance of horizontal acceleration in seismic impact zones and structural stability in unstable areas, is also included in the proposed regulations.

Subpart C of 40 CFR Part 258 (USEPA 1991), addressed required daily operating criteria with regard to cover material requirements, disease vector control, explosive gas control, air quality, access restrictions, run-on/runoff control systems, surface water, liquids restrictions, record-keeping requirements, closure and postclosure criteria, financial assurance, and the exclusion of receipt of hazardous wastes. Although procedures to detect and prevent disposal of hazardous wastes and wastes containing PCBs are outlined in this section, the TSCA makes provisions for the disposal of limited categories of PCB materials in MSWLFs.

Design criteria for liner and leachate collection systems, based on such considerations as hydrogeology, climate, leachate characteristics, groundwater quality, proximity of groundwater users, final cover requirements, and overall groundwater carcinogenic risk levels are detailed in Subpart D of 40 CFR Part 258 (USEPA 1991). Two basic design options are outlined in the rules—one required only in states without USEPA-approved programs and the other available in states with approved programs. The latter is site specific and subject to state approval and must ensure that drinking water maximum contaminant levels (MCLs) will not be exceeded in the uppermost groundwater aquifer at the point of compliance (often the solid waste disposal area boundary). The former is required in states without USEPA approval and involves the installation of a protective composite liner system consisting of a flexible membrane liner and a compacted soil component. The flexible membrane liner component provides a highly impermeable layer to maximize leachate collection and removal, and the lower soil component serves as a backup in the event of liner failure (40 CFR Part 258 [USEPA 1991]). A provision in the regulations does allow states without approved programs to petition to use the performance standard approach instead of the composite liner approach.

Groundwater monitoring procedures and corrective action measures are included in Subpart E of 40 CFR Part 258 (USEPA 1991). These requirements may be suspended, however, upon demonstration to the state that no potential exists through the postclosure period for migration of hazardous constituents to the uppermost aquifer. In addition to requiring the installation of a network of monitoring wells at new and existing MSWLFs, the proposed rules outline sampling and analysis requirements. A two-phased monitoring approach has been proposed in which initiation of Phase II monitoring is triggered by a change in groundwater chemistry or detection of certain Phase I parameters at statistically significant levels above background. Phase I monitoring includes sampling semiannually for the following parameters: antimony, arsenic, barium, beryllium, cadmium, chromium, cobalt, copper, lead, nickel, selenium, silver, thallium, vanadium, zinc, and several VOCs. Phase II monitoring consists of an expanded list of hazardous contaminants (40 CFR Part 258 [USEPA 1991]). In the event that any of the Phase II parameters is detected at statistically significant levels, the regulations dictate that the measures level be compared to the state-specified groundwater protection level, either an SDWA MCL or some health-based concentration limit that triggers the assessment of corrective measures.

Cornwell et al. (1992) conducted a survey of state agency landfill and monofill requirements. Common classifications of WTP sludge by states included special or solid waste (majority of states), industrial waste, municipal waste, commercial waste, and waste by-products of a WTP. As in classification, there is considerable variability among states regarding disposal practices. At the time of the survey, nearly all states were using the TCLP test to determine whether a sludge is hazardous.

In general, it was apparent that state regulations vary tremendously. Some states have few regulations and limited knowledge regarding how and where their WTP wastes are disposed of, whereas others are much more highly regulated, with clear procedures in place for dealing with WTP wastes.

Several clear divisions emerge among the various state regulations. Possibly the most prevalent dividing line occurs between states that have large numbers of WTPs producing sludge and those that have only a few. States that have experienced problems with WTP sludge have procedures in place to deal with the waste, whereas those generating small amounts of waste, such as those whose main supply is from groundwater aquifers, seem to have little concern about it and consequently have few regulations governing its disposal. States having small or few WTPs that are able to discharge wastes to large receiving waters within their borders with little discernible impact usually have little regulation of WTP disposal practices.

State responses to survey questions dealing with landfilling of sludges in MSWLFs and sludge monofilling are summarized in Table 5.10. Landfill requirements vary from state to state, and sludge-only landfills are often permitted on a case-by-case basis. For example, although New Jersey regulations did not prohibit the disposal of WTP wastes in MSWLFs, there were no MSWLFs in the state.

Solids content of a sludge is the criterion most widely used to determine whether it will be accepted for disposal in an MSWLF. Another commonly used gauge is the amount of free water contained in a sludge, normally measured using the paint filter liquids test. Specific mixing ratios of sludge to refuse are also required in some states. No states prohibit the use of sludge monofills. Requirements were reported to be the same as or similar to those for a new MSWLF in the majority of states.

Among states requiring a minimum percent solids content, Nebraska and California requirements were found to be the most stringent at 70 and 50 percent, respectively. In several other states, such as New York and South Carolina, sludge having a solids content >20 percent was found to be acceptable for disposal. At the other end of the spectrum, several states were found to have no minimum sludge solids concentration criteria. Found throughout various USEPA regions, states falling into this category included Colorado, Florida, Maryland, Michigan, and Rhode Island. A majority of states prohibit the deposit of WTP sludge containing free liquid; several of these specify the paint filter liquid test. Most states require no specific mixing ratio of WTP sludge to refuse, although some suggest a 1:3 or 1:4 ratio to maintain workability. A ratio of 1:10 is required by West Virginia.

Many states require the use of the paint filter liquid test to define “no free liquid.” The paint filter liquid test (USEPA Method 9095 in USEPA 1998) uses a 100-g sample placed in a funnel with a 6-mesh filter. In order to pass the test, no liquid can pass through the filter in a 5-minute time period, and the wastes would be classified as containing no free-flowing liquids.

Table 5.10
Summary of state requirements for landfilling of WTP sludge

Standard	Percentage of states where standard was in place at time of survey
Solids content	
2%–20%	8
20%–30%	18
30%–40%	4
>40%	6
Numerical limit not specified	60
Free water	
No free water	56
Paint filter liquid test specified	34
Minimum sludge-to-refuse ratio (applies to codisposal only)	18
Monofill requirements essentially the same as codisposal (often case by case as well)	76

Source: Cornwell et al. 1992.
NOTE: All 50 states were surveyed, but two states did not provide data.

Common threads in the regulatory framework for WTP sludge disposal at the state level emerged through comparison of responses to several survey questions. In addition to categorization of WTP sludge and methods of determining whether a waste is hazardous, the formality of each state's current regulatory structure, in terms of the procedures in place and their enforcement, was given a relative ranking by the interviewer. At the time of the survey, the degree of formality exhibited by most in-place state regulatory structures was low. With regard to this parameter, for only 7 out of 50 states (California, Iowa, Michigan, Minnesota, New Jersey, New York, and Pennsylvania) were the systems in place deemed very formal. At the other end of the spectrum, the regulatory structures of Georgia, Hawaii, and Maine were labeled informal.

Another survey regarding state regulatory approaches for WTP wastes was documented in the paper titled "State Regulatory Approaches for Water Plant Wastes" (AWWA 1991). The Water Treatment Waste Disposal Committee found that codisposal in MSWLFs, in spite of rapidly diminishing available capacity, was the most frequently employed disposal alternative. Next to codisposal, land application and direct discharge were found to be most prevalent. Although most states regulate WTP wastes based on NPDES standards and solid waste and sewage sludge regulations, these wastes are not necessarily a major concern to state regulatory agencies, many of which are waiting for the development of USEPA regulations for WTP sludges before developing their own.

Municipal Solid Waste Landfills

For those utilities dewatering their solid–liquid waste streams, landfilling of the resultant solids is a commonly used method for final disposal. In the case where the sludge is disposed of in a municipal landfill, the utility often has little to do except determine the requirements for using the landfill. In some cases, the landfill must be approved to accept industrial wastes in order to dispose of water plant residuals at the site. The principal constraint on using the landfill is usually the allowable solids concentration. While some landfill owners or state regulations will set a specific solids concentration as previously described, usually the requirements are more qualitative. The requirements may be stated as “no free water,” “must behave as a solid,” or “must be handleable by earth-moving equipment.”

Monofill Description and Design

Types of Monofills

The two major types of sludge monofilling methods are trench filling and area filling. Trench filling can be further subdivided into narrow trench and wide trench monofilling techniques. The three basic types of area filling include the area fill mound, area fill layer, and diked containment methods. Method selection is determined principally by sludge solids content, sludge stability, site hydrogeology (location of groundwater and bedrock), ground slope, and land availability. The following sections describe specific aspects of the various sludge monofilling methods.

In trench landfills, sludge is placed entirely below the original ground surface. Trench depth is dependent typically on the depth to groundwater and bedrock; sufficient soil buffers between the sludge and substrata must be maintained. Trench depth is a function of sidewall stability and equipment limitations as well. The trenching method encompasses both narrow and wide trench-type disposal areas, which range in width from several feet to 50 ft (15.2 m).

Narrow trenches are generally employed for sludges with low solids concentrations that cannot support any type of heavy equipment. Wide trenches are used for sludges when solids concentrations are sufficient to achieve the necessary shear strength to support heavy equipment. Trenching is a convenient way to operate a landfill because trucks can unload sludge from firm ground above the trench while a hydraulic excavator located outside the trench or a tracked dozer operating inside the trench places and compacts the sludge. Trenches are also relatively quick and easy to construct, thus minimizing construction costs.

The planning and design of sludge disposal trenches involve determination of the following parameters in order to predict the acreage needed for a long-term disposal plan:

- Cover thickness
- Excavation depth
- Length
- Orientation
- Sludge fill depth
- Spacing
- Width

Final cover thickness depends on the trench width and type of equipment (land based or sludge based) to be employed in final cover operations. It should be noted that a daily cover for odor control is typically not required for WTP sludges. Factors influencing excavation depth include location of groundwater and bedrock, soil permeability, soil cation exchange capacity, equipment limitations, and sidewall stability. Trench length is limited by sludge solids content and ground slopes; trenches must be discontinuous dikes constructed to contain sludge with a low solids content in a sloping area. For optimal land utilization, trenches should be oriented parallel to one another. Sidewall stability, in addition to ultimately controlling excavation depth, determines trench spacing. In general, for every foot of trench depth, 1.0 to 1.5 ft (0.3 to 0.5 m) of space should be provided between trenches (USEPA 1978). Spacing should not hinder vehicular access or preclude stockpiling of trench spoil. Sludge solids content and equipment limitations are both important considerations in appropriate trench width determinations.

The selection of narrow trenches <10 ft (<3.05 m) in width versus wide trenches is primarily based on the anticipated sludge solids concentration and available property size. Narrow trenches are generally operated with equipment such as hydraulic excavators located on firm ground above the trench. The primary advantage of the narrow trench system is its ability to provide a feasible means of ultimate disposal for relatively wet sludges. On the other hand, substantial land requirements and poor space utilization dictated by the number of trenches and required amount of ground between trenches are significant disadvantages of this system. In addition, if a synthetic liner is required for each individual trench, the construction cost of the trench would increase sharply because of the relatively small scope of the work and high mobilization costs for a lining contractor.

Wide trenches, generally classified as up to 50 ft (15.2 m) in width, are applicable for adequately dewatered sludges capable of supporting heavy equipment such as tracked dozers or similar types of tracked vehicles. The main advantage of wide trenches as opposed to narrow trenches is the better land utilization, because fewer numbers of wide trenches are required to handle the same volume of sludge. Wide trenches also provide better access for installing liners. A disadvantage of wide trenches relates to the fact that sludges, which are typically dewatered sufficiently to prevent free flowing, must be unloaded directly on the trench floor, thus requiring access for trucks.

Unlike the trench landfilling technique, where sludge is placed belowground, in area filling techniques, sludge is placed above the original ground surface. Area filling may be accomplished in one of three ways:

1. Area mound, where sludge is mixed with soil such that it becomes stable enough to be stacked in mounds
2. Area layer, where sludge is spread evenly in layers over a large tract of land
3. Diked containment, where earthen dikes are constructed aboveground to form a containment structure into which the sludge can be disposed

Although solids content is not limited in area fill landfills, the requirement that sludge must be capable of supporting heavy equipment because of the lack of sidewall containment necessitates reasonably good sludge stability and bearing capacity. These characteristics are typically achieved through good dewatering, dewatering followed by air drying, or mixing sludge with bulking agents. A combination of these methods can also be employed. Areas with high water tables and those with bedrock close to the surface are particularly amenable to area fill methods of sludge monofilling. Liners are therefore more likely to be required, but their installation is easier than the installation of liners in trenches. A general overview of the three types of area filling techniques is presented in the following paragraphs.

The area fill mound technique is a disposal method used for wastewater sludges. For wastewater sludge, a soil bulking agent is generally mixed with sludge to enhance stability and increase bearing capacity to the degree required based on the sludge depth and the weight of the equipment. After being piled in mounds approximately 6 ft (1.83 m) high, the sludge and soil mixture is covered with at least 3 ft (0.91 m) of soil cover material (more if additional lifts are to be piled on top of the first mound). This disposal method allows for good land utilization and reasonably high application rates. On the negative side, the tendency of mounds to slump, particularly under high rainfall conditions, and the resulting need for mound readjustment introduce higher labor and equipment requirements. This monofilling method should be adaptable to WTP sludges; however, it should be evaluated based on the sludge shear strength, size of the monofill, and so forth.

In the area fill layer disposal method, sludge is spread in 6- to 12-in. (15.2- to 30.5-cm) layers. This provides additional air drying of the sludge and allows higher solids concentration and shear strengths to be achieved. This method seems favorable for coagulant sludges, which are typically difficult to dewater. The layering method eliminates the need for a separate air drying area outside the monofill, provided the monofill cell is large enough. The area fill layering technique usually results in relatively stable fill areas when completed and therefore requires less extensive equipment and labor efforts for maintenance than the area fill mound technique.

In the diked containment disposal method, earthen dikes are constructed aboveground to form a containment structure into which the sludge can be disposed. Containment areas may sometimes be placed at the top of a slope, which can provide part of the containment structure itself. Access roads are constructed on top of the dikes so that sludge haul trucks can unload sludge directly into the disposal cell.

The sludge can be disposed inside the diked containment area in either a layering or a mounding technique, although the layering technique seems to be the preferred method. Access should be provided into the disposal cell itself for tracked equipment and for trucks delivering the sludge.

When the diked containment disposal method is used, available land is developed to its greatest potential. Large cells can be constructed and long-term construction cost savings achieved. The diked containment method allows the highest sludge loading rate per acre (due to greatest available storage volume) of the three types of area filling by providing the stability necessary to increase sludge depth. Because of high sludge loading rates per acre, a liner and leachate collection system may be necessary to prevent moisture from being squeezed into the surrounding dikes or subsurface soil.

One final but essential aspect of monofill planning and design involves effective stormwater management. Surface grades of 2 to 5 percent should be maintained to promote runoff, preclude ponding, and limit flow velocities, thereby minimizing soil erosion. Stormwater collection should be utilized to route upstream stormwater flow around the monofill. Sediment ponds and other erosion control measures should be employed as necessary.

All monofill sites should be provided with an all-weather road that provides access from a public road. Gravel roads should be considered the minimum standard. Road slopes should generally be no steeper than 7 to 10 percent for uphill and downhill grades, respectively, to be accessible to fully loaded vehicles. Sludge could be delivered to the working area by way of temporary roads. The need for buildings and utilities is dictated by landfill size, but some consideration should be given to a truck wash station, equipment storage, and operations facilities. Necessary limited access can be achieved through installation of gates and peripheral fencing, depending on the relative isolation of the site.

Utilization of Sludge Physical Characteristics for Monofill Design

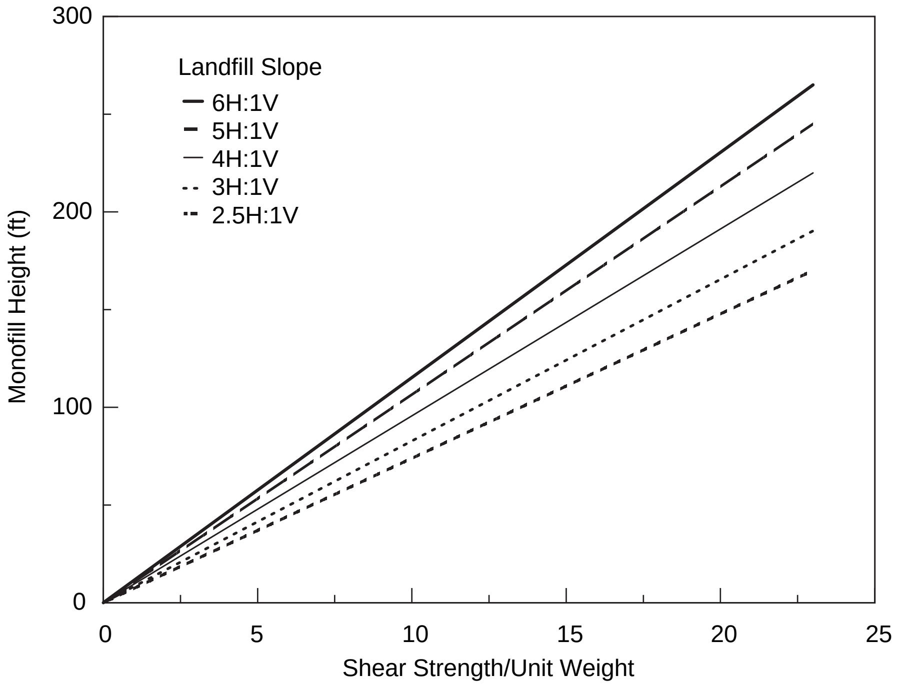
The sludge physical data discussed in chapter 2 showed shear strength values for determining sludge stability. These data allow the determination of the required minimum solids concentration that would result in a shear strength adequate to (1) create stable side slopes to prevent slope failure and (2) support the heavy earth-moving equipment utilized in monofill operations. Once the required minimum solids concentration is determined, the monofill size can be established based on sludge generation rates, density, and disposal depth. It should be noted that both the slope stability and the bearing capacity analysis must be performed in order to determine which requirement will govern the monofill design.

Slope Stability Analysis. Sludge monofills operated with an area fill technique require a slope stability analysis to prevent slumping or sliding of the side slopes. The sludge mass on a sloping surface is subject to numerous shearing stresses because gravitational force tends to pull the upper parts of the sludge mass downward. Provided that the shear strength of the sludge is greater than the highest internal stress, the side slope would remain stable. However, if the sludge shear strength becomes less than the internal stress, even for brief periods of time, slope failure occurs to a point where the internal stress again becomes less than the sludge shear strength.

In planning a sludge monofill, it would be useful to determine the required sludge solids concentration and corresponding shear strength in order to maintain a stable side slope condition. A procedure was developed by Cornwell et al. (1992) to utilize the results from physical characterization tests to obtain the required minimum solids concentration and shear strength. The slope analysis methodology assumed a uniform side slope with a constant angle from the toe of the monofill to the top of the slope, and a monofill supported on a firm level ground. Using the method of limiting equilibrium, the maximum monofill height with a stable slope was computed for different levels of shear strength. Figure 5.8 shows the relationship between the monofill height and the ratio of shear strength to wet unit weight for five slope levels. The five slope levels ranged from 6H:1V (where H is horizontal, V is vertical) (16.7 percent slope, 9.5° slope angle) to 2.5H:1V (40 percent slope, 21.8° slope angle). The results shown in Figure 5.8 incorporate a safety factor of 1.2 for planning purposes.

By utilizing Figure 5.8, the minimum solids concentration required for maintaining a stable slope can be determined. The procedure of determination involves an iterative process and can be accomplished as follows:

1. Select the desired slope angle and landfill height.
2. Find the required ratio of shear strength to wet unit weight from Figure 5.8.
3. Assume a wet unit weight of sludge, and compute the shear strength.
4. Find solids concentration for the computed shear strength from soil data such as that shown in Figure 2.18.
5. Check the wet unit weight for the solids concentration using data such as that shown in Figure 2.14 corrected to wet weight by Equation 2-18.
6. Repeat steps 3 through 5 until the assumed and calculated unit weights balance.



Source: Cornwell et al. 1992.

Figure 5.8 Monofill height versus the ratio of shear strength to wet unit weight of the sludge

This procedure was applied to alum sludge 1 in chapter 2 as an example using these methods. The side slope was set at 3H:1V, and a 20-ft (6.1-m) monofill height was considered. The results of this analysis are shown in Table 5.11.

For the sludge, utilizing data from Table 5.11, a solids concentration in the range of 25 percent would be required for a 20-ft-high monofill.

The shear strength required to ensure a stable side slope may not be sufficient to prevent heavy equipment from settling into the sludge. Thus, an equipment-bearing capacity analysis must be conducted to determine whether the shear strength required to maintain a stable slope or the shear strength required to support heavy equipment should govern in the design of a monofill.

Equipment-Bearing Capacity Analysis. The objective of the equipment-bearing capacity analysis is to determine the minimum sludge shear strength necessary to support various types of heavy equipment commonly used in the operation of a sludge monofill. The resulting sludge shear strength is then compared with the slope stability data to establish the governing shear strength.

The bearing capacity analysis considered the bearing capacity failure of the sludge under the drive wheel (or track) of the equipment; in other words, equipment settlement into the sludge. The failure condition considered the equipment static weight on a level surface and utilized the general bearing capacity equation. The bearing capacity equation (5.11), developed by Terzaghi and Meyerhof (in Spangler and Handy 1982) incorporates dimensionless bearing capacity factors designated N_c, N_q, N_γ:

$$q_\mu = cN_c + qN_q + 0.5B\gamma N\gamma \qquad (5.11)$$

Table 5.11
Slope stability analysis example

Trial No. 1	
Required slope	3H:1V
Monofill height [ft (m)]	20 (6.1)
Shear strength/wet unit weight	2.7
Assumed wet unit weight [lb/ft^3 (kg/m^3)]	50 (850)
Shear strength [lb/ft^2 (kN/m^2)]	135 (6.5)
Shear strength (psi)	0.9
Solids concentration (%)	20
Calculated wet unit weight [lb/ft^3 (kg/m^3)]	74 (1,258)
Assumed wet unit weight [lb/ft^3 (kg/m^3)]	74 (1,258)
Trial No. 2	
Shear strength [lb/ft^2 (kN/m^2)]	199.8 (9.6)
Solids concentration (%)	26
Calculated wet unit weight [lb/ft^3 (kg/m^3)]	74 (1,258)

Source: Cornwell et al. 1992.

where q_μ = bearing capacity in force per unit area
c = undrained shear strength
q = surcharge loading
B = width
γ = soil unit weight

The three N-bearing capacity factors are functions of the internal friction angle Φ and the assumed shape of the failure zone. For a failure analysis of clay in an undrained shear condition, $\Phi = 0$. This yields values of $N_c = 6.16$, $N_q = 1.0$, and $N_\gamma = 0$, based on Terzaghi and Meyerhof (in Spangler and Handy 1982) standardized tables. Also, $q = 0$, assuming no surcharge pressure exists around the equipment wheel. Thus, by substituting these values, the bearing capacity equation yields

$$q_\mu = 6.16c \tag{5.12}$$

Factors of safety are commonly applied in this type of analysis, and by incorporating a safety factor of 3.0, the allowable bearing pressure is

$$q_a = \frac{q}{\mathrm{F}} = \frac{6.16c}{3.0} = 2.05c \tag{5.13}$$

where q_a = allowable bearing pressure
F = safety factor

Table 5.12
Specifications and ground pressure for monofill equipment

		Track width		Track length		Static weight		Ground pressure	
Type	Horsepower	(in.)	(cm)	(ft, in.)	(m)	(lb)	(kg)	(psi)	(kN/m^2)
Crawler dozer	67	25	63.5	6 ft, 9 in.	2.1	17,170	7,788	4.25	29.07
Crawler dozer	90	30	76.2	8 ft, 7 in.	2.6	25,822	11,713	4.20	28.73
Crawler dozer	120	34	86.4	10 ft, 3 in.	3.1	34,782	15,777	4.20	28.73
Crawler dozer	165	36	91.4	10 ft, 8.3 in.	3.3	43,355	19,666	4.70	32.14
Dump truck (empty)	250	NA*	NA	NA	NA	20,000	9,072	25	170.98
Dump truck (full)	250	NA	NA	NA	NA	38,900	17,645	50	341.96
Excavator	10	18	45.7	11 ft, 0 in.	3.4	28,000	12,701	5.89	40.28

Source: Cornwell et al. 1992.
*NA = not applicable.

The pressure (q) exerted by the equipment on the sludge therefore cannot exceed the allowable bearing pressure. Calculating the induced pressure and solving for the undrained shear strength as shown in Equation 5.14 yields the shear strength required for the sludge to support the equipment with a factor of safety of 3.0.

$$c = \frac{q_a}{2.05} \tag{5.14}$$

Various types of equipment applicable to monofill operation are available. Typically, crawler dozers would be utilized for spreading and stacking sludge in an area mound operation. The crawler dozers can be equipped with extra wide and long tracks to minimize the ground pressure exerted by the equipment's static weight. Hydraulic excavators would also be applicable to sludge monofilling in either an area mound technique or a trench disposal method. Empty and fully loaded tandem dump trucks were included in the analysis to compare their shear strength requirements with those of the tracked equipment. Table 5.12 summarizes the equipment specifications considered in the bearing capacity analysis and the related ground pressure for the individual types of equipment.

As shown in Table 5.12, the crawler dozers exert relatively low ground pressure in comparison to the dump trucks. Crawler dozers with extended tracks for low ground pressure would be quite suitable for monofilling operations. Proceeding with dump trucks onto a sludge monofill would require careful consideration and, possibly, special unloading areas.

Table 5.13
Required shear strength and solids concentration for various types of heavy equipment

	Ground pressure		Minimum shear strength		Approximate sludge concentration (%)		
Type	(psi)	(kN/m^2)	(psi)	(kN/m^2)	No. 1	No. 2	No. 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

Source: Cornwell et al. 1992.
NOTE: psi × 6.9 = kPa.

The required sludge solids concentrations for a specific sludge can be determined for the individual types of heavy equipment by substituting the calculated ground pressure in the equation $c = q_a/2.05$. The resultant c equates to the minimum sludge shear strength required to support the equipment. Based on the triaxial compression or cone penetration data, the corresponding sludge solids concentration can be determined. Table 5.13 summarizes these results for the three sludges shown in Figure 2.18. As shown in the table, the tracks on the crawler dozers and hydraulic excavator provide low minimum shear strength requirements and, consequently, the laboratory data yield manageable sludge solids concentrations for these types of equipment. For alum sludge no. 1 and no 2, the sludge solids concentration should be at least around 30 and 25 percent, respectively, to support the tracked equipment. Empty and fully loaded tandem dump trucks typically required at least 7 percent higher solids concentrations compared with the tracked equipment even though the minimum required shear strength was significantly higher. The shear strength data, however, exhibit a nonlinear relationship with respect to the solids concentration, and hence a significant increase in required shear strength does not necessitate a similar increase in solids concentration.

When the required minimum sludge solids concentration is determined, a decision can be made as to how the solids concentration can be achieved. Options to be considered include the level of sludge dewatering, subsequent air drying requirements, and use of bulking agents. As the data in Table 5.13 indicate, the solids requirements are site specific. This fact, coupled with site-specific evaporation rates for natural drying and additive levels for bulking agents, precludes generalized methods of predicting the minimum solids concentration for a particular utility. Rather, a certain degree of preliminary laboratory work would be required in the feasibility phase of a disposal plan.

Monofill Permitting Process

Design requirements for sludge monofills vary from state to state; however, in the majority of instances, the general design criteria for sludge monofills mirror those that are imposed on

MSWLF design. It is likely that strong similarities would also be found in the permitting processes, particularly with regard to the required components of an application package.

In addition to site acceptability and detailed design, permit application approval for most landfills involves submittal to reviewing authorities of the following standard information:

- Soils and hydrogeological analyses
- Operational plan
- Erosion and sedimentation control plan
- Groundwater monitoring plan

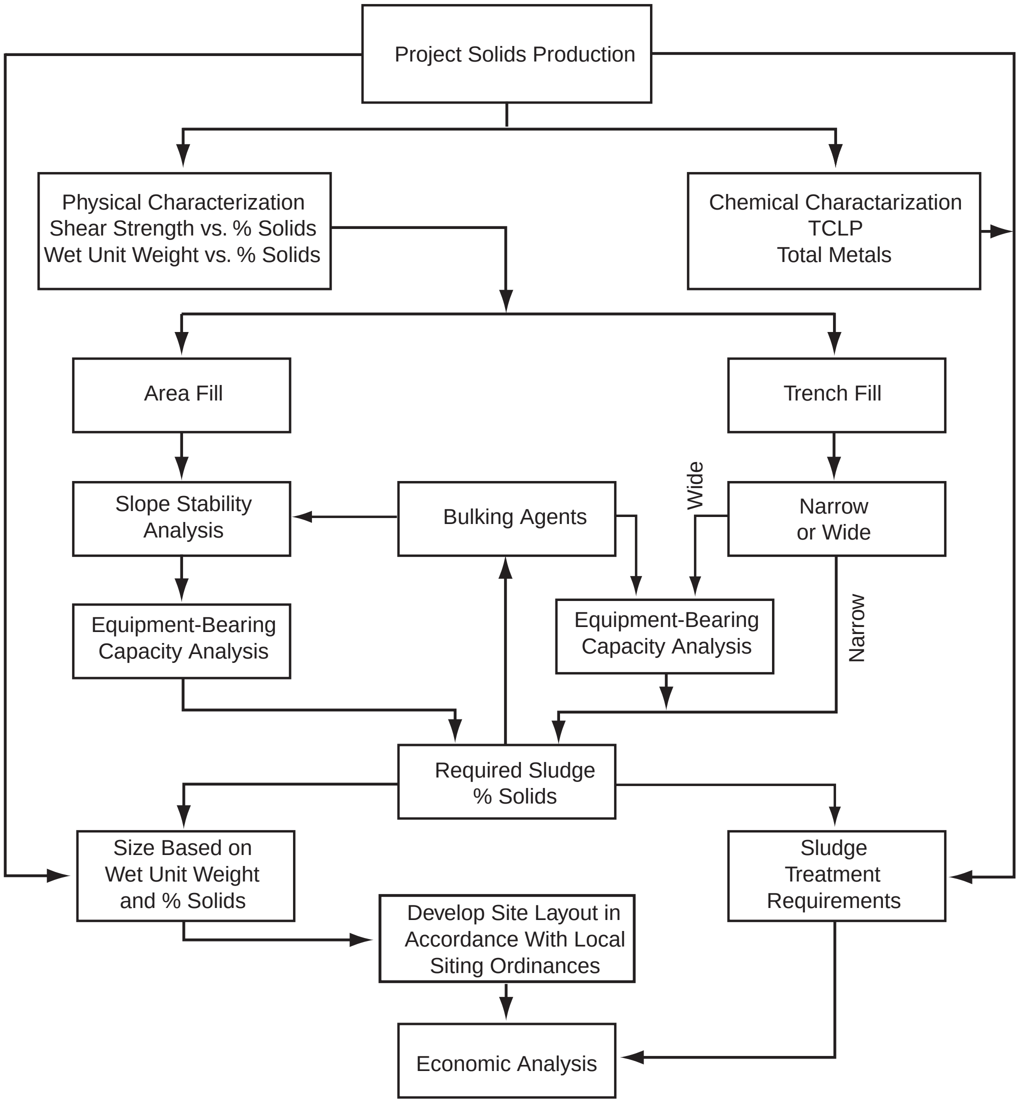
Because most of the concern regarding land disposal of WTP sludge stems from concern about groundwater contamination, more detailed information regarding the characteristics of the waste to be landfilled, such as background metals concentrations (determined by a total metals analysis) and results of leaching tests, may be required as part of the permit application (see Cornwell et al. 1992, Hsieh and Raghu 1997).

Figure 5.9 presents a schematic diagram that highlights the essential planning considerations for disposal of WTP sludge in dedicated monofills. The diagram depicts the determining factors and the effects of each decision and stage in the planning process. For example, an integral parameter dictated largely by slope stability and equipment-bearing capacity analyses, the required sludge solids content impacts both monofill size and sludge treatment, which in turn influence economics. As indicated on the figure, sludge treatment requirements and project economics are affected not only by the quantity of sludge produced but also by the sludge's physical and chemical characteristics. In summary, Figure 5.9 presents a logical path to be followed in planning a sludge monofill and defines the interrelationships of the principal design considerations.

BENEFICIAL USE

Many WTPs use a variety of beneficial use methods for managing coagulant- and lime-based residuals. Elimination of surface water discharge, stricter landfilling regulations, higher landfill tipping fees, and restrictions on sewer discharge have forced water utilities to spend more time marketing residuals in order to locate and develop alternatives to disposal. Research and marketing efforts over the years have resulted in the development of many new and innovative techniques for beneficial use of residuals. Beneficial use markets discussed in the next section include

- Land application
- Cement manufacturing
- Brick making
- Turf farming
- Composting (with yard waste or biosolids)
- Commercial topsoil and potting soil production
- Road subgrade
- Forest land application
- Citrus grove application
- Nutrient control
- Landfill cover



Source: Cornwell et al. 1992.

Figure 5.9 Considerations for sludge monofill design

- Land reclamation
- Hydrogen sulfide (H_2S) binding
- Flue gas desulfurization

This list contains a broad array of residuals uses that could potentially provide benefit to several commercial markets such as agriculture, industrial manufacturing, forestry, solid waste handling, land reuse, and pollution control. For each of the applications listed, residuals could be used to supplement or even replace raw materials normally used. In some cases, coagulant residuals have proven to be equal to or even more valuable than the natural or commercial products replaced. Table 5.14 is a list compiled by Cornwell, Mutter, and Vandermeyden (2000) of utilities that were identified as protectors of beneficial use of WTP residuals.

Table 5.14
Utility beneficial use matrix

Utility	Residuals type	Co-use with biosolids	Composting with yard waste	Turf farming	Topsoil blending	Landfill cover	Land application[*]	Brick making	Cement manufacturing	Nutrient control	Road subgrade	Land reclamation
Metropolitan Water, Los Angeles, Calif.	Alum/ferric		X						X			
Tampa, Fla.	Ferric						X					
Elizabethtown, N.J.	Alum				X		X					
ECWA, Buffalo, N.Y.	PACl			X	X							
Newport News, Va.	Alum						X					
Myrtle Beach, S.C.	Alum			X								
Charlottesville, Va.	Alum/PACl	X				X						
Durham, N.C.	Alum							X				
Cobb County–Marietta, Ga.	Alum/lime						X					
New Jersey Water Supply Authority, Clinton, N.J.	Alum				X		X					
Santa Clara, Calif.	Alum					X		X				
Wales, United Kingdom	Alum							X				
Greenwich, Conn.	Alum	X	X									
Danville, Pa.	Alum									X		
Englewood, Colo.	Alum				X		X					
Tulsa, Okla.	Alum								X	X		
Boynton Beach, Fla.	Lime										X	
Bradford, Pa.	Alum											X
Cary/Apex, N.C.	Alum							X				
Earthgrow, Inc.[†]	Alum				X							

Source: Cornwell, Mutter, and Vandermeyden 2000.

*Land application includes citrus groves, agricultural land, forest land, and hay fields.

†Commercial user of WTP residuals.

Agricultural Land Application

Agricultural land application of water treatment residuals is the most commonly practiced beneficial use method. It is reported in AWWA's WATER:\STATS database that at least 25 percent of the utilities responding perform land application of residuals. The specific type of land application was not reported by WATER:\STATS, but it would be reasonable to assume that the majority of the residuals are applied to agronomic soils. A detailed description of the processes involved for land application of water residuals are outlined in *Land Application of Water Treatment Sludges* (Elliott et al. 1990). The Awwa Research Foundation (AwwaRF) land application report provides a very good source of technical information concerning the principles and design associated with land-applying residuals. Implementation logistics and residuals quality requirements are also summarized in the report.

Other valuable land application references include the following:

- "Groundwater and Crop Growth Issues Associated With Water Treatment Residuals" (Knocke et al. 1991)
- "Agronomic Effects of Land Application of Water Treatment Sludges" (Elliott and Dempsey 1991)
- "An Assessment of Cropland Application of Alum Sludge" (Mutter 1994)

Several land application research studies conducted using coagulant residuals have demonstrated neutral or slightly positive impacts on crop growth (Lin and Green 1990, Geertsema et al. 1994). Some of the benefits associated with the addition of coagulant residuals to agronomic soils include

- Improvement to soil structure
- Soil pH adjustment
- Addition of trace minerals
- Increased moisture holding capacity
- Soil aeration

Some negative effects on soil characteristics have also been documented. Research studies have shown that some coagulant residuals have a tendency to bind plant-available phosphorus in soils (Elliott and Dempsey 1991, Knocke et al. 1991). Also, aluminum phytotoxicity could also be a problem if the soil pH is not maintained ≥6.5 (Elliott and Dempsey 1991).

Elliott et al. (1990) provides a complete instructional guide for a utility interested in pursuing a land application program. The report provides detailed information on the following topics:

- Description of major components found in coagulant residuals
- Residuals effects on soil phosphorus availability
- Heavy metals and residuals toxicity
- Residuals effects on physical properties and soil pH

Land application implementation logistics include

- Residuals application design
- Site selection
- Agricultural methods
- Storage of residuals
- Application rates
- Monitoring and reporting

Land application of residuals can be performed using either liquid or cake solids residuals. The liquid or solid material could be effectively land applied at any solids concentration found to be economically feasible by a utility and acceptable to the landowner. The amount of dewatering required is based primarily on hauling distances to the application site, storage facilities required, residuals water value, and landowner preference. Solids concentrations for liquid residuals applications from coagulation processes range from 0.5 to 10 percent, whereas cake residuals applications require a solids concentration of greater than 15 percent. Lime sludges can be applied as a liquid up to 10 percent solids concentration and as a solid if the solids concentration is greater than 40 percent.

Liquid residuals applications, where feasible, can provide several advantages. Liquid applications only require gravity sedimentation and thickening, thereby eliminating the need for dewatering facilities and equipment. Liquid applications to agronomic soils can be applied throughout the growing season depending on the type of crop produced and the application technique used. Applications throughout the growing season, if feasible, may provide an additional water value for crop growth. Residuals applied in a liquid form tend to provide a more even application to agronomic soils, create less soil clumping, and are incorporated into the natural soil more rapidly. A disadvantage of land-applying liquid residuals is the increased volume of residuals, which directly impacts the handling and transportation costs. Liquid applications are only economically attractive when application sites are within close proximity to the WTP, or if relatively small quantities of residuals are generated.

Land application of dewatered residuals requires dewatering to a solids concentration that can be handled by front-end loaders, transported by dump trucks, and spread onto farmland using manure spreading–type equipment. The volume reduction, however, significantly reduces transportation and handling costs. Cake solids are not typically applied during the growing seasons because of the potential for physical crop damage during spreading. Therefore, a residuals storage facility may be required to stockpile residuals until land application is possible. Uneven distribution and soil clumping are also potential problems using cake solids application.

In order to increase the value of coagulant residuals for agricultural use, several contractors and utilities have developed processes that combine residuals with other beneficial agricultural products. Residuals amendments include lime addition, fertilizers, biosolids, and finished compost materials. Any of these products could effectively increase the agronomic value of the water treatment residuals. Fertilizers (N, P, K), compost, or biosolids could be blended with residuals prior to or during the land application process. Blending the residuals with any of these amendments at the proper ratios increases the residuals value and, as a result, makes marketing of residuals to farmers an easier task.

The chemical and physical quality of coagulant residuals and effects on agronomic soils are outlined by Elliott et al. (1990). They suggest several chemical and physical parameters that should be analyzed for residuals characterization. A list of important residuals quality parameters that should be investigated prior to land-applying residuals is presented in Table 5.15. Regulatory agencies responsible for granting land application permits may require a utility to test for some or all of these parameters as part of the permit application process. Subsequent testing may be required on an annual or semiannual basis for permit compliance. Many of the states regulate residuals beneficial use on a case-by-case basis depending on the type of use and quality of the residuals. Therefore, no exact list of parameters will apply to every utility.

Application Rate Determination

Elliott et al. (1990) prepared a procedure for determining the allowable land application ratio for coagulant residuals. Their procedure is summarized in the following paragraphs, some of which is also applicable to lime residuals.

Nitrogen. Compared to sewage sludge, large quantities of residuals can be land applied on an annual basis without risk of nutrient overloading. A conservative approach is to determine the amount of residuals needed to supply 20 percent of the crop's N requirement. Twenty percent was chosen because of the uncertainty regarding what fraction of the total N in residuals is available to the crop in the first growing season (referred to as the first-year mineralization rate). A mineralization rate of 10 percent was found to be reasonable by Elliott et al. (1990). If the actual mineralization rate is less than 10 percent, a substantial crop yield will still be realized because 80 percent of the N need will be supplied with commercial fertilizer. If the actual mineralization rate is greater than 10 percent, N pollution will be limited because only 20 percent of the crop need is being satisfied by the residual.

pH Adjustment. The quality of agricultural limestone and other liming agents is strictly controlled by state agricultural authorities. Factors considered are calcium carbonate equivalence (or acid neutralizing capacity), fineness, and moisture content. With the exception of lime-softening sludges, most water treatment sludges are not of high enough quality to be sold and marketed as liming agents. However, many sludges have the ability to favorably alter pH, and alteration of soil pH should be considered when determining an application rate.

Soil pH, soil pH buffer capacity, and sludge calcium carbonate equivalent (CCE) must be known before an application rate can be determined. These analyses can be obtained from the state soil testing laboratory and many commercial laboratories. Soil pH alone only indicates if a site needs liming. It does not indicate the amount of liming agent needed. The amount of lime required to achieve neutralization to pH 7.0 is determined by reacting a soil sample with a buffer solution whose initial pH is 7.0. The exchangeable acidity in the soil reacts with the buffer solution and causes a decrease in pH. The laboratory will specify liming requirements in the soil test results in units of CCE of standard ground limestone (90 percent CCE) per acre. The sludge application rate (dry tons per acre) is determined by dividing the liming requirement by the sludge CCE expressed as a decimal. Table 5.16 shows application rates for materials with a CCE of 70 percent or greater.

It is generally not advantageous to apply more than 8,000 lb of CCE per acre (800 kg/m^2) at one time. In fact, because the goal is to apply sludge annually, it is best to achieve small, incremental changes in pH and not completely neutralize a soil in one application. With careful pH

Table 5.15
Important residuals quality parameters for land-applying coagulant residuals

Parameters	Units
Physical tests	
Solids concentration	%
Color	NA*
Texture	NA
Soil aggregation	NA
Moisture content	%
Grain size analysis (clay/silt/sand)	%
Specific weights	lb/ft^3 (kg/m^3)
Chemical tests	
Nutrients	
TKN	lb/ton (mg/kg)
Total phosphorus	lb/ton (mg/kg)
Potassium	lb/ton (mg/kg)
Ammonia–nitrogen	lb/ton (mg/kg)
Nitrate–nitrite (as N)	lb/ton (mg/kg)
Calcium	lb/ton (mg/kg)
CCE	%
Metals	
Total metals	lb/ton (mg/kg)
TCLP metals	mg/L
Radionuclides	
Gross alpha	pCi/g
Gross beta	pCi/g
Ra-226	pCi/g
Organics	
TOC	lb/ton (mg/kg)
Toxicity	
Phytotoxicity—Microtox test	EC_{50}
Other tests	
Total coliform	no./gram
pH	NA

Source: Cornwell, Mutter, and Vandermeyden 2000.
*NA = not applicable.

Table 5.16
Liming material conversions

Pounds per acre of CCE on soil test	Percent CCE of liming material								Divide total into the following no. of applications
	70	75	80	85	90	95	100	105	
	Actual recommendation (lb/acre)								
1,000	1,400	1,300	1,200	1,200	1,100	1,100	1,000	1,000	
2,000	2,900	2,700	2,500	2,400	2,200	2,100	2,000	1,900	
3,000	4,300	4,000	3,700	3,500	3,300	3,200	3,000	2,900	
4,000	5,700	5,300	5,000	4,700	4,400	4,200	4,000	3,800	1
5,000	7,100	6,700	6,200	5,900	5,600	5,300	5,000	4,800	
6,000	8,600	8,000	7,500	7,100	6,700	6,300	6,000	5,700	
7,000	10,000	9,300	8,700	8,200	7,800	7,400	7,000	6,700	
8,000	11,400	10,700	10,000	9,400	8,900	8,400	8,000	7,600	
9,000	12,900	12,000	11,200	10,600	10,000	9,500	9,000	8,600	
10,000	14,300	13,300	12,500	11,800	11,100	10,500	10,000	9,500	
11,000	15,700	14,700	13,700	12,900	12,200	11,600	11,000	10,500	
12,000	17,100	16,000	15,000	14,100	13,300	12,600	12,000	11,400	
13,000	18,600	17,300	16,200	15,300	14,400	13,200	13,000	12,400	2
14,000	20,000	18,700	17,500	16,500	15,600	14,700	14,000	13,300	
15,000	21,400	20,000	18,700	17,600	16,700	15,800	15,000	14,300	
16,000	22,900	21,300	20,000	18,800	17,800	16,800	16,000	15,200	
17,000	24,300	22,700	21,200	20,000	18,900	17,900	17,000	16,200	
18,000	25,700	24,000	22,500	21,200	20,000	18,900	18,000	17,100	3
19,000	27,100	25,300	23,700	22,400	21,100	20,000	19,000	18,100	
20,000	28,600	26,700	25,000	23,500	22,200	21,100	20,000	19,000	

Source: Elliott et al. 1990.
NOTE: lb × 0.4536 = kg.

monitoring, it will be possible to determine an annual decrease in pH due to calcium leaching and adjust the sludge application accordingly.

Heavy Metals. Most residuals do not contain excessive concentrations of heavy metals. For this reason, metals will probably not control the annual application rate of sludge. The federal government has mandated that Cd loadings be less than 0.5 kg Cd/ha-year (0.45 lb Cd/acre-year). Additionally, many states limit the annual loading of Cu, Cr, Pb, Hg, Ni, and Zn. Using the rates determined for fertility or soil pH adjustment, the annual loading rate of each metal should be calculated to see if any of these limits are exceeded. If so, the sludge application rate will be dictated by the metal that results in the lowest application of sludge per acre.

Though not usually affecting annual application rate, metal buildup in the soil will determine how many years a site can be used for sludge disposal, given that USEPA guidelines and the regulations in many states stipulate a lifetime loading limit for each metal.

Table 5.17
Recommended cumulative metal limits for cropland

	Soil cation exchange capacity (meq/100 g)		
	<5	5 to 15	>15
Metal	kg/ha (lb/acre)		
Lead	560 (500)	1,120 (1,000)	2,240 (2,000)
Zinc	280 (250)	560 (500)	1,120 (1,000)
Copper	140 (125)	280 (250)	560 (500)
Nickel	140 (125)	280 (250)	560 (500)
Cadmium	5 (4.4)	10 (8.9)	20 (17.8)

Source: USEPA 1983b.

Wet Weight Basis. Dividing the dry ton application rate by the total solids content (expressed as a decimal) of the sludge gives the wet ton application rate.

If the sludge is a liquid or slurry, volume and depth of application should also be determined. Calculations are shown in the following equations; the sludge density of 62.5 lb/ft^3 (1,000 kg/m^3) can be adjusted using the density method from chapter 2 if the solids concentration is significant.

$$\frac{\text{wet ton}}{\text{acre}} \times \frac{2{,}000\ \text{lb}}{\text{ton}} \times \frac{\text{ft}^3}{62.5\ \text{lb}} = \frac{\text{ft}^2}{\text{acre}} \tag{5.15}$$

$$\frac{\text{ft}^2}{\text{acre}} \times \frac{\text{acre}}{43{,}560\ \text{ft}^2} \times \frac{12\ \text{in.}}{\text{ft}} = \text{in. depth} \tag{5.16}$$

The depth of application should be checked against the site infiltration rate and moisture holding capacity given the soil survey to see if the soil can absorb the hydraulic loading. If hydraulic capacity is exceeded, two or more partial applications may be necessary.

Site Life. Application of sludge should stop when the allowable lifetime loading of metals has been reached. The values given in Table 5.17 are recommendations by USEPA. Most states have adopted these, or more restrictive, metal limits. These values are for the total metals applied. Determine the total metals added at the initial application rate and divide the allowable loading by this number. This will give the estimated site life in years. Because residuals do not contain high concentrations of heavy metals, sites can be used for many years based on metal-loading criteria.

Loading Rate Calculation Example. An alum sludge removed from drying beds will be incorporated into a silt loam soil on which corn is to be grown for grain. The analysis of the alum sludge is as follows:

Solids content	15%
Total nitrogen	0.5%
Phosphorus	0.2%

Potassium	0.1%
Cadmium	2 ppm
Copper	200 ppm
Chromium	100 ppm
Nickel	80 ppm
Lead	150 ppm
Zinc	500 ppm

Fertilizer recommendations for 125 bu/acre of corn yield are 160 lb N/acre (16 kg/m^2), 50 lb P_2O_5/acre (5 kg/m^2), and 38 lb K_2O/acre (3.8 kg/m^2). The initial soil pH is 5.0, its cation exchange capacity is 10 meq/100 g, and the alum sludge has a CCE of 10 percent. Soil test results indicate 7,000 lb/acre of CCE required for this soil.

1. Determine the residual application rate necessary to satisfy 20 percent of the crop's N requirement (160 lb N/acre [72 kg N/acre]). The WTP must supply 32 (160 × 20 percent) lb (15 kg) of available N. Assuming a 10 percent first-year mineralization rate, the available N per ton of sludge is

   $$0.1 \times \frac{10 \text{ lb total N}}{\text{ton}} = 1 \text{ lb available N per ton} \tag{5.17}$$

   Thus, 32 tons per acre (7.0 kg/m^2) will supply 32 lb available N per acre (3.2 kg/m^2).
2. Check to ensure that the N application rate calculated in step 1 will not (a) result in excessive liming of the soil or (b) exceed the federal maximum Cd loading (0.45 lb Cd/acre-year)
   a. Because the residual has a 10 percent CCE, 10 tons will be required to supply 1 ton of CCE. Liming of the soil requires 7,000 lb (3,150 kg) of CCE; therefore, 35 tons (32,000 kg) of residuals would satisfy the liming requirement. An application rate of 32 tons/acre (3,100 kg/m^2) will not result in overliming.
   b. The residuals have 2 ppm Cd, which corresponds to 0.004 lb Cd per ton of residual. At 32 tons of residuals per acre (3,100 kg/m^2), the Cd loading would be 0.128 lb Cd/acre. Assuming a once-per-year application, this loading is well below the federal limit of 0.45 lb (0.045 kg/m^2) Cd/acre-year.
3. Determine supplemental P, K, and limestone that must be applied.
   a. Prior to crop planting, soil sampling should be accomplished to ensure that adequate plant-available P exists. If P deficiency is indicated, follow the P fertilizer recommendation of the soil testing laboratory. Since WTP residuals may exert a P demand, this should be accounted for.
   b. The residual has 0.1 percent K, which corresponds to 2 lb K/ton:

$$\frac{2 \text{ lb K}}{\text{ton}} \times \frac{1.2 \text{ lb } K_2O}{\text{lb K}} = \frac{2.4 \text{ lb } K_2O}{\text{ton residual}} \tag{5.18}$$

Thus, 77 lb K_2O (2.4×32) will be applied per acre (7.7 kg/m^2). Because the soil test called for only 38 lb K_2O per acre (3.8 kg/m^2), no supplemental K is required.

4. The useful life of the field as a residual application site can be calculated. At 32 tons/acre (7.04 kg/m^2), the annual Zn application rate would be

$$500 \text{ ppm} \times \frac{0.002 \text{ lb/ton}}{\text{ppm}} \times \frac{32 \text{ tons}}{\text{acre}} = 32 \text{ lb Zn/acre} \tag{5.19}$$

According to Table 5.17, a total of 500 lb Zn per acre is allowed to be applied to this soil. Thus, the number of years the site can be used is

$$\frac{500 \text{ lb Zn/acre}}{32 \text{ lb Zn/acre-year}} = 15.6 \text{ years} \tag{5.20}$$

This same calculation can be performed for the other metals. The actual estimated site life will be the shortest time predicted for any of the metals.

Land application of softening sludge has been practiced for years. Farmers were allowed to cart away dewatered softening sludge from a plant in Ohio as early as the 1950s (Reeves 1980).

In farming regions, the application of nitrogen fertilizers causes a reduction in soil pH. If optimum pH conditions do not exist, crop yields will be reduced. Therefore, farmers must apply sufficient quantities of calcium carbonate as a means of counteracting the fertilizer applications. For each 45.4 kg (100 lb) of ammonia fertilizer, 1.4 to 1.8 kg (3 to 4 lb) of limestone must be applied (Russell 1980).

Several state agencies have evaluated the neutralizing power of softening sludges versus commercial limestone. In 1969, the Ohio Department of Health reported that the total neutralizing power of lime sludge is greater than that of marketed liming materials (Ohio Department of Health 1969). To bring the soil pH into the desirable range, 2,721 kg/4,047 m^2 (3 tons/acre) lime, or about 9,070 kg (10 tons) of lime sludge at a 30 percent solids concentration, are required. Subsequent lime applications are required to maintain the desired pH.

In Illinois, a CCE test performed on several softening sludges indicated that the softening sludges were superior to agricultural limestones available locally. Because softening sludges contain a high quantity of calcium carbonates and offer a high degree of neutralization, this resource should be used for soil conditioning when it is practical. The addition of softening sludge also increases the porosity of tight soils, making them more workable for agricultural purposes (Russell 1980).

Cement Manufacturing

Water treatment residuals have successfully been used as an ingredient for the production of portland cement. cement (commonly referred to as hydraulic cement) is a material that has the property of hardening under water and is the primary bonding agent in concrete and masonry. The name “portland” was chosen because the inventors thought that cement resembled a building stone quarried from the Isle of Portland off the coast of England. More than 95 percent of the cement produced in the United States is portland cement (USGS 1997). Portland cement is a principal material used for concrete construction, and because of its high rate of use, there is a continuously increasing demand for it. Uses for cement include ready-mix concrete, block, pipe, precast slabs, road construction, and building materials.

The natural materials used for cement production include limestone, shale, and clay. The critical elements supplied by the raw materials for cement production are calcium, silica, aluminum, and iron. Limestone provides approximately 70 to 80 percent of the raw material required for cement production but only contains low concentrations of aluminum, iron, and silica. In order to supplement the required elements, cement plants add other materials such as clay, shale, iron ore, and bauxite (USGS 1997).

Water treatment residuals generated using lime softening or coagulants commonly contain some or all of the key elements that cement plants add during their manufacturing process. Residuals could potentially increase concentrations of these critical elements, which would reduce the total volume of supplemental materials a cement manufacturer would have to purchase and add. A reduction in the volume of raw materials required could provide a cost savings to the manufacturer.

The recommended location for residuals addition is during crushing and prehomogenization of raw materials. Incorporation of the residuals at this stage of the process ensures complete grinding and mixing of the residuals along with other raw materials. A storage facility is required for stockpiling residuals at the cement plant prior to use. The residuals are blended into the other raw materials at a desired ratio based on the quality and quantity of the residuals. Demonstration tests are necessary to determine the optimal mix ratio prior to full-scale production. Other application points may also be possible; however, because of the limited use of residuals for cement manufacturing, no established procedures exist.

Storage facilities and application equipment required for adding residuals into the cement-making process may or may not preexist at the cement plant. A covered concrete storage pad, front-end loader, conveyor system, and feed hopper could be required for residuals incorporation. Most cement plants will most likely require some process modifications but should not require additional equipment. Cement manufacturing is a year-round process that utilizes a large supply of raw materials; therefore, only short-term storage of residuals at the cement plant would be necessary.

Brick Manufacturing

Brick manufacturing is an ancient art that involves many techniques; and the manufacturing equipment used for brick making has resulted in the development of a modern industry. Computer-controlled kiln designs and a better knowledge of the raw materials used for brick making have resulted in increased production and better-quality bricks. Today's brick manufacturing facilities in the United States produce billions of bricks per year. Manufacturers are capable of providing a wide variety of colors, textures, and shapes desired by consumers. Bricks are used for a wide variety of purposes; however, the majority of bricks produced are used for commercial, residential, and industrial construction.

There are some striking similarities between the physical and chemical compositions of materials used for brick making and coagulant residuals. Alum, ferric, and PACl residuals have chemical and physical properties similar to the natural clays and shales used for brick production. Coagulant residuals consist of clays, silt, and sand removed from the water during treatment along with organic matter and other chemical compounds formed during chemical coagulation. Those residuals with a high clay content are optimal for brick making. Lime, on the other hand, is detrimental to brick quality, and, therefore, residuals containing significant concentrations of lime are not acceptable for brick-making applications (Ceratec 1998).

The raw materials used for brick manufacturing include clay and shale. Clay is the most abundant mineral material on earth; however, not all clays have the same properties. The natural clay used for brick production must have specific properties in order to produce good-quality bricks.

Several water utilities, along with the assistance of brick makers, have performed demonstration testing using residuals as an ingredient for brick making. Utility experience suggests that success or failure of brick making as a beneficial use alternative is dependent on several factors, including the following (Rolan 1976, Migneault 1989):

- Proximity of brick-making plant to the WTP
- Residuals chemical and physical qualities
- Coagulants and other chemicals used during the treatment process
- Acceptance of the residuals by brick manufacturers
- Impacts on normal operations

The use of water treatment residuals in brick production could benefit the brick industry in several ways. Water treatment plants generate tons of residuals on a daily basis that could be used to offset some of the enormous volume of natural materials used for brick production. The added volume of residuals could potentially decrease the volume of natural clay and shale, thereby increasing the life of the quarry used by the brick manufacturer for their raw material.

Studies conducted by brick manufacturers have demonstrated that good-quality bricks can by produced using water plant residuals. No significant impacts on the structural properties of brick were noted, in tests conducted by the City of Durham, when up to 15 percent alum residuals were added to natural clay for brick production (Rolan 1976). Several manufacturers have also found that residuals containing significant concentrations of iron hydroxide or barium produce a desirable red color in finished bricks (Rolan 1976, Migneault 1989, Owen 1998).

Quarry application of the residuals is the easiest method for blending residuals with the natural raw materials. Dewatered residuals are transported from the water plant to the quarry and either stockpiled or dumped directly into the quarry. Residuals that are stockpiled are later blended with other materials prior to delivery to the plant. Residuals dumped directly into the quarry are ripped into the natural clays by mining machinery. The blending process is accomplished using front-end loaders, augers, or pugmill equipment. Residuals blending rates are typically <10 percent of the total volume of materials used on a daily basis for brick making. Overall, the amount of residuals that could be supplied by most WTPs is only a very small percentage of the huge total volume of raw material used for brick manufacturing. For example, a typical brick-manufacturing plant uses on average approximately 121,000 DT (109,800 metric tons) of raw materials annually. With a typical blend rate of 5 percent, up to 6,000 DT/year (5,450 metric tons/year) of residuals could be supplied annually. By comparison, a fairly large WTP that treats 50 mgd (190,000 m^3/d) of raw water with an average turbidity of 15 ntu using 30 mg/L of alum would only generate around 3,000 DT/year (2,720 metric tons) of residuals or 2.5 percent of the volume of materials used by a typical brick plant.

Blending of residuals at the brick plant requires a more technical approach toward introducing the materials into the existing plant processes. Brick plant blending would most likely involve more operator attention and control than quarry applications of residuals (Migneault 1989). Dewatered residuals from the treatment plant are transported directly to the brick manufacturing facility and stockpiled. Additional facilities at the brick plant may be necessary for storage and handling of residuals prior to incorporation into the brick-making process. Additional facilities and/or equipment could include a covered concrete storage pad, conveyers, feed hoppers, and front-end loaders. The optimal locations for introduction of residuals into the brick-making process are during the "preparation" stage of the process (Maury 1998, Parrish 1998). Residuals are added to the plant mullers and crushing equipment along with the natural clay and shale rock for materials blending. After addition of residuals into the other raw materials, the remainder of the brick-making process remains unchanged.

Turf Farming

Since the early 1980s, the turf farming industry has doubled the total number of acres used for turf grass production. The 1992 Census of Agriculture (USDA 1992) listed a total of 218,160 production acres (88,290 ha) in the United States versus 124,600 acres (50,425 ha) in 1982. The majority of turf grass produced is used for residential landscaping. Other turf grass uses include commercial building landscaping, golf courses, sports fields, parks and cemeteries, and roadside landscaping.

The typical turf grass production farm during 1997 averaged 350 acres (141 ha) with a total production acreage of 67 percent of the farm (235 acres [95 ha]). Turf grass sod production, similar to other agriculture crops, requires a significant investment by the farmer that must be recovered through production and sales in order to maintain a profitable operation. Turf farmers compete with nature, other methods of turf establishment, turf producers, and other forms of ground cover. Intensive management is required to produce quality turf grass.

Coagulant residuals can be applied to turf fields in a liquid or solid form. The extent of dewatering required prior to application is dependent on transportation, dewatering costs, economics, and preference by the turf farmer.

Liquid residuals can be land applied during field preparation or during the turf growth stage. Liquid residuals applied to soil during field preparation should be thickened at up to at least 3 percent solids concentration and sprayed evenly over the application area to prevent excessive wetting of the soil. Caution must be used such that the residuals do not increase the soil moisture concentration to levels unacceptable for cultivation and seeding. Liquid residuals could also be applied to the fields while the turf grass is growing. Residuals applied directly onto turf grass must have a very low solids concentration to prevent residuals from blocking photosynthesis by coating the grass blades (Wooley 1996). Liquid applications to turf grass could provide an even distribution of residuals with minimal clumping, as well as a water value for irrigation.

Cake solids residuals application is only possible during field preparation and residuals should not be applied directly to the turf grass. Residuals should be dewatered to a cake solids concentration of at least 20 percent. Residuals are typically surface applied using manure-spreading equipment and then cultivated into the natural soil. Dewatered residuals application methods decrease hauling costs to the farm but could result in uneven distribution and soil clumping.

Liquid or solid residuals application rates to soils similar to other land application practices are usually based on cumulative nutrient and/or heavy metals loading rates. Demonstration studies could be performed to determine the optimal loading rates for good turf growth while minimizing metals accumulation in the soils. Adsorption of plant-available nutrients and heavy metals loading rates should be assessed prior to full-scale applications.

The Erie County Water Authority (ECWA) near Buffalo, N.Y., operates two WTPs that treat municipal drinking water using conventional methods. The Sturgeon Point Water Treatment Plant (90 mgd [341,000 m^3/d]) treats water from Lake Erie whereas the Van de Water Treatment Plant (50 mgd [189,000 m^3/d]) treats water from the Niagara River. Raw water quality is generally very good, with turbidity typically <10 ntu. Both plants use PACl as the primary coagulant. PAC is fed seasonally for removal of taste and odor.

Combined PACl residuals generated by both plants average 5,500 to 9,100 lb/d (2,500 to 4,100 kg/d) of dry solids. Residuals chemical characteristics are very similar to the native soil; however, the residuals contain higher concentrations of aluminum, manganese, TKN, and copper.

Residuals handling at the Sturgeon Point Water Treatment Plant consists of a 1.7-acre (0.7-ha) storage lagoon. The lagoons are cleaned every 5 years and freeze–thaw dewatered followed by air drying. The final residuals cake is approximately 60 percent solids concentration. The Van de Water Treatment Plant uses a residuals holding tank, a residuals thickener, a retention tank with lime addition, and a plate-and-frame filter press for residuals dewatering. The final residuals cake concentration is approximately 35 percent.

ECWA conducted a residuals management study to determine the feasibility of residuals disposal of landfilling by employing a beneficial use alternative. A long list of potential beneficial use alternatives were reviewed by ECWA. The following three alternatives were considered to be the most favorable:

1. Turf farm application
2. Topsoil blending
3. Lightweight concrete blending

ECWA's criteria for selecting a beneficial use program focused on locating a user(s) that would buy or accept residuals at no cost, develop several different beneficial use options, and establish long-term contracts that would ensure project success. Beneficial use disposal was determined to be the most economical alternative when compared with disposal to sanitary landfills or monofills. ECWA concluded that turf farming and topsoil blending were the most promising options for residuals reuse.

ECWA located a turf farm within close proximity to the Buffalo area that was interested in testing residuals for use as a soil substitute. The turf farm owner agreed to allow ECWA perform a demonstration study at the turf farm. The 1-year demonstration project involved adding residuals to nine plots each for both Sturgeon Point and Van de Water residuals along with two control (no residual addition) plots. Application rates used were 1.5, 3, and 6 percent, which were equivalent to 13.7, 27.4, and 54.8 DT/acre (30.63, 61.3, and 122.8 metric tons/ha), respectively.

Statistical analysis of the soil and soil–water data revealed that the test plots demonstrated no significant differences compared to the control plots. The readily available phosphorus concentration was not shown to have decreased by the addition of residuals. Visual observations also revealed no significant differences in turf appearance using a residuals loading rate up to 3 percent. The 6 percent loading rate (which would be an unusually high application rate) did cause a noticeable strain on turf growth; however, it was inconclusive as to whether or not this was a result of residuals addition to the test plot.

Overall, a 10 to 20 percent yield increase was achieved using residuals application rates of 1.5 and 3 percent. Data collected from the demonstration study suggested that turf farming was a viable option. Based on the successful findings from the demonstration study, ECWA received a Beneficial Use Development Permit from the New York State Department of Environmental Conservation. This permit allowed ECWA to begin marketing residuals to turf farmers and topsoil producers.

Composting

Composting is a natural biological process that accelerates the decomposition of organic solid waste into a soil-like material. Composting operations used for recycling solid waste, yard waste, bark fines, and sawdust are becoming an increasingly popular alternative to landfilling. Some communities recycle as much as 60 percent of their solid waste by composting.

Composting operations have successfully used wastewater biosolids for years as an additive to compost piles. During organic decomposition, heat is generated, which destroys pathogens and effectively sanitizes biosolids into a material that is safe for reuse. The use of biosolids in composting generates a valuable recycled fertilizer material.

Work has been conducted using residuals as an ingredient in compost piles along with yard waste, solid waste, bark, and biosolids. Addition of residuals has been shown to benefit the composting process by providing moisture, trace minerals, pH adjustment, and by serving as a bulking agent. Co-composting using blends of residuals and biosolids has been demonstrated to benefit the composting process and final product by diluting heavy metals concentrations that are regulated by many states for land application reuse.

Finished compost material can provide a valuable and environmentally safe soil amendment for agricultural or commercial soil applications. Many municipal composting facilities supply finished compost materials to end users at little or no cost, whereas commercial composting operations use their finished compost to produce topsoil and potting soil blends that are bagged and sold commercially.

Water treatment residuals have been used successfully as a bulking agent for composting. Fine compost materials such as grasses or ground leaves require a bulking agent to increase pore space for aeration and moisture distribution. For compost operations that receive very dry materials, residuals can also effectively provide moisture to the pile, which is critical for organic decomposition.

The blending ratios of residuals mixed into a compost pile is dependent on the type of materials being composted. Demonstration studies would be necessary to determine the optimal residuals-to-compost mix. Residuals could be blended into the compost blend prior to or during windrow formation. Pugmills, blending augers, windrow machines, and front-end loaders are frequently used for blending the various compost materials.

Residuals delivered from the water plant should be dewatered to at least 15 percent solids concentration for use in composting. Most compost facilities have existing equipment that is capable of handling a semidry or dry material. The amount of dewatering required is dependent on the other compost materials used and should be determined on a case-by-case basis. Extremely wet residuals are not recommended for composting applications because of handling and storage problems.

Residuals may require stockpiling at the WTP or compost facility prior to blending and windrow formation. Composting can be performed year-round; however, short-term storage will be required for periods of inclement weather or for periods when other compost ingredients are not available (i.e., grass, leaves). If residuals are to be stored at the composting site, additional storage facilities may be required. A concrete storage pad with stormwater drainage collection should be used to contain residuals and residuals leachate.

Topsoil and Potting Soil Production

Manufacturing of commercially sold soil products is a very large industry. Artificial or screened soil products such as mulch, potting soil, topsoil, and manure are available at commercial lawn and garden centers. There is a high seasonal demand for these products for residential and commercial landscaping and horticultural applications.

In order to manufacture commercial soil products, various raw materials are required. Typical ingredients used for potting soils and topsoils include perlite (aeration), crushed limestone (pH conditioning), sand (weight), bentonite clay (bulking agent), peat mosses, bark fines, and fertilizer (N and P). These ingredients are transported to a manufacturing facility where ingredients are blended and bagged for sale. Raw material costs and transportation costs are both major economic considerations for manufacturers.

Use of water treatment residuals as an ingredient for production of various commercial soil products has become an increasingly popular option for utilities. Residuals have been demonstrated to be an effective substitute for some raw materials commonly used for soil production including perlite, limestone, sand, and bentonite clay. Residuals are valuable to soil manufacturers primarily for use as a bulking agent or weight additive in their products. Most of the ingredients used in soil production are very lightweight. To increase product weight, manufacturers add heavier materials such as sand or clay. Without the added weight, a 40-lb (18-kg) bag of potting soil would be extremely large and bulky, and difficult to handle by consumers. Use of residuals could effectively reduce the amount of other materials normally used and, as a result, decrease materials and transportation costs.

Manufacturers also realize the value of using recycled materials for product sales and marketing. Products advertised as a “recycled material” could provide a manufacturer with a significant marketing advantage over competitors.

Water treatment residuals used as an ingredient for manufacturing commercial soil products is handled and blended using the same equipment, machinery, and techniques as used for the other ingredients. Additional storage space and conveyors may be necessary for addition of residuals. The amount of residuals added to a soil blend is typically only a small percentage of the entire mix. Blend ratios are a function of the consistency, quality, and availability of the residuals. Ratios must be determined by the manufacturer through demonstration testing.

Many commercial soil production facilities include a composting operation that generates finished compost products for use as ingredients in potting soils and topsoils. Residuals could be incorporated with other raw materials during the composting process instead of as a direct addition to soil products.

Road Subgrade

Lime-softening residuals have been used successfully for years as a subgrade material for constructing foundations for roads and parking lots. In certain regions of the country, the soil structure is very unstable and is not capable of supporting pavement without the addition of rock and soil materials. Road construction requires a hard base to prevent potholes and cracking of concrete or asphalt. Lime-softening residuals used are typically blended with various types of rock, gravel, or soil materials to form a hard, stable road base.

Use of lime-softening residuals for this application could significantly decrease the volume of other materials normally used for construction. Water treatment plants are typically willing to give away lime residuals at no cost; however, most require that the user provide all of the labor and equipment required for removal and transportation. Lime residuals used as a subgrade material generally benefits both parties involved and provides a reuse application that is safe as well as beneficial.

The process used for road construction is very simple. Contractors, or other users, collect the residuals from the treatment plant as needed and deliver it to either the road construction site or blending facility. Residuals are mixed with shell rock, gravel, or other materials using some form of mechanical blending (pugmill, auger, front-end loader, etc.). The mixed subgrade material is then applied evenly over the road base and compacted. After the material dries and hardens, it forms a stable road base that is capable of supporting pavement. No extra equipment is typically required by the road contractor for residuals use.

Forest Land Application

Many water utilities own and manage watershed properties that protect their raw water supply sources. Watershed property is typically protected forest land that is used as a buffer to prevent contamination of a drinking water supply. Management of watershed includes planting trees, forest management, control of stormwater runoff, and harvesting trees for profit. An extensive forested watershed could also be valuable for land-applying residuals generated by WTPs.

The principals used for forest application are similar to agricultural land application practices. Metals and nutrient loading rates, spreading methods, and compliance monitoring need to be closely evaluated before initiating a forest application program. Forest application programs require extensive operator management and specialized heavy equipment for spreading residuals onto forest land.

To develop a forest land application program, the first step is to determine the quantity and quality of residuals that will be applied on a daily or yearly basis. This knowledge will help a utility determine the total acreage that would be required for maintaining a long-term program. The proposed land for residuals applications should be surveyed to determine which tracts of the land are suitable for this form of land application. Regulatory agencies that will potentially oversee the residuals application program will have specific criteria for selecting land application sites. Factors such as land slope, proximity to surface water, property lines, power lines, and so forth must be considered when siting for residuals application. After identifying which tracts can be used, adequate roads must be constructed to allow spreader vehicles to reach the application sites.

Forest applications during inclement weather or at other times may not be possible; therefore, a residuals storage facility is required at the forest application site or WTP for short-term storage. Dewatered residuals would need to be delivered and stockpiled at this facility until land spreading is possible.

Application rates are usually based on heavy metals and nutrient accumulation in the forest litter. Demonstration studies may be required to determine the effects on forest soils, groundwater, and plant species in order to determine a safe loading rate. During full-scale operation, a monitoring program would be necessary to track soil characteristics and groundwater quality over time. Monitoring is critical for determining how frequently residuals can be applied and ultimately how much residuals can be spread on each tract of land.

The Newport News Department of Public Utilities (Waterworks) operates two WTPs that produce approximately 50 mgd (189,250 m^3/d). The treatment processes include conventional treatment at one facility and upflow clarification at the other. Both plants use aluminum sulfate (alum) as the primary coagulant at an average dosage of 50 to 60 mg/L. PAC is fed during the warm-water months (as needed) for taste and odor removal. The quality of raw water obtained from the Waterworks reservoirs is generally good; however, there is a high concentration of organics and color. Frequent applications of copper sulfate are used during warm-weather months to control algae in the reservoirs. These applications are responsible for the high copper concentration found in the residuals produced at each plant.

Alum residuals produced by the Lee Hall plant are piped to a common centrifuge facility located at the Harwood's Mill Water Treatment Plant. The centrifuge produces a residuals cake concentration of approximately 16 to 18 percent. The residuals are then trucked to a holding site prior to forest application.

The Waterworks owns and manages approximately 8,000 acres (3,237 ha) of watershed property. Studies determined that 2,000 acres (809 ha) of the total watershed was suitable for land-applying alum residuals. The Virginia State Water Control Board provided Waterworks with an interim land application permit to perform demonstration studies using alum residuals. Demonstration studies were conducted by the Waterworks along with Virginia Polytechnic Institute and State University (Geertsema et al. 1994) to determine the impacts of alum residuals on Loblolly pines. Both greenhouse and field studies were conducted to assess impacts on soil, soil–water, and plant growth. Results from these studies concluded that low-rate residuals land application was a viable beneficial use.

Information obtained from the demonstration study was necessary for securing a land application permit granted by the Virginia Department of Environmental Quality (DEQ). After receiving a permit in 1994, Waterworks began full-scale application to stands of Loblolly pines. Residuals are hauled to two covered storage facilities located on Waterworks property for storage and are land applied using an ag-chem application vehicle. The Waterworks DEQ permit specified the buffer widths and boundaries used to design the land application sites. Proper distances were maintained from roads, tributaries, surface water, and power poles as required by the permit. Copper, nitrogen, and aluminum loading rates are frequently monitored to maintain levels as regulated by DEQ permit. The land application program has been and continues to be an effective and accepted management alternative.

Citrus Grove Application

Citrus crops grown in southern U.S. states include oranges, grapefruit, and many other citrus species. Soils in these regions of the United States are typically low in iron, which is vital for the growth of citrus crops. Citrus farmers in these regions frequently apply agricultural iron amendments to grove soils to satisfy the trees' demands for this element. The use of ferric sulfate ($Fe_2(SO_4)_3$) as a primary coagulant generates a residual that is rich in iron humate. Many utilities in these regions have switched from using alum coagulants to a high-purity ferric sulfate to increase residuals value and make the residuals more marketable for land application. High-purity ferric sulfate has a reduced concentration of heavy metals that would otherwise limit citrus grove application rates. Ferric residuals used on citrus crop soils have been demonstrated to be as effective as the other commercial iron products typically used.

Dewatered ferric residuals are usually delivered to the citrus farm by utility trucks or by contractors and surface applied to the citrus grove. Application rates should be determined based on the chemical quality of residuals and should be verified through demonstration testing. The application process is similar to applications using other commercial iron products. Monitoring of heavy metals accumulation in citrus grove soil should be routinely performed to prevent soil contamination.

Land application of residuals to nutrient-contaminated soils is used to bind available soluble phosphorus and nitrogen. Demonstration studies should be conducted to determine a particular residual's potential for nutrient reduction. Application rates can then be established to achieve a desired reduction of nutrient concentrations in soils.

Buffer strips around feed lots, poultry houses, and cropland could be constructed to minimize nutrient runoff into adjacent waterways. Buffer strip application rates would also need to be determined through demonstration testing.

Blending of residuals with animal waste products for use as agricultural fertilizer could serve to lower the nutrient concentrations to nonpolluting levels while still providing a fertilizer value to crops. Residuals could be blended to animal waste products prior to or during application.

Landfill Cover

For years, dewatered residuals have been transported to sanitary landfills for final disposal. Development of alternative disposal practices and increased tipping fees have significantly reduced the volume of residuals landfilled. Some utilities have developed agreements with MSWLFs to use residuals as a daily cover material instead of just burying the material. Landfills require a large volume of fill material for covering up the different cells of garbage on a daily basis. This material is generally supplied from borrow pits or from previous landfill excavation. By blending residuals with other daily fill materials, residuals qualities are diluted and the volume of fill material is increased. Because of the residuals value as a fill material, some landfills will lower tipping fees for accepting residuals, thereby making it more economical to utilities. Many utilities still view landfills as the safest location for disposal of solid wastes, limiting the potential for future liability due to environmental contamination caused by residuals.

Dewatered residuals are delivered to the landfill and stockpiled for later use. Depending on local or state regulations, a concrete storage pad with drainage collection may or may not be required. Residuals are transported by the WTP, and a tipping fee is normally charged depending on the agreement with the landfill. The landfill may use the residuals directly as cover material or blend with other fill material. Front-end loaders and dump trucks that typically exist at landfills are capable of handling and applying residuals.

Land Reclamation

Land reclamation is the process of improving the ability of environmentally damaged land to support plant growth. Damaged land could include abandoned strip mines, quarries, borrow pits, and so forth. The goal of land reclamation is to add fertile soil back to areas that are not capable of supporting vegetation. Strip mine operations remove the topsoil from areas and leave behind sites that are incapable of supporting plant life and are susceptible to erosion. Recently, residuals have been used successfully as a material for reclaiming damaged land. Water treatment residuals are typically low in nutrient content; however, the material provides a good base of clay, sand, silt, and trace minerals that can support growth. Nutrients can be added along with residuals using fertilizers, composted materials, biosolids, or other nutrient-rich materials.

The procedures used for land reclamation are very similar to methods used for land application. Dewatered residuals, however, are applied to the reclamation area at much higher loading rates than for agricultural land application. The optimal loading rates should be determined through demonstration testing using test and control plots located at the reclamation site. To improve the soil fertility for supporting plant growth, nutrient-rich materials could be blended along with residuals or added after residuals application. Materials such as grass, leaves, compost, other yard wastes, or even biosolids could be added to provide valuable nutrients to the soil.

Land reclamation is completed by introducing different species of grasses, crops, or trees to the application site. Different species of grasses should be used for early restoration followed by shrubs, bushes, and trees. Grasses will provide a quick form of ground cover that will stabilize the soil and prevent erosion.

Sheff, Rogers, and Murrell (1997) reported results on using lime residuals in land reclamation. They reported that lime residuals aftermixed with suitable soils and compacted can render the land suitable for commercial, residential, or recreational uses. They focused on reclaiming an abandoned gravel pit, two sand mines, and closure of previous lime sludge lagoons.

Abandoned Gravel Pit—Jackson, Mich.

This reclamation project was performed at a site that had become a nuisance to nearby residential housing in Jackson, Mich. The original area permitted for land reclamation was 24 acres and was an ideal location because it was within 2 miles of the Jackson water treatment facility. Initially, the project included topographic surveying to determine quantities of lime and soil that could be placed at the site.

Geologically, the site consisted of sand extending down to 15 ft (4.6 m) below the base of the sand mine where a clay till material was encountered. This clay till material is believed to extend to 45 ft (14 m) and directly overlay the local bedrock which is used as a water supply source. The water table in the base of the sand mine was within 1 foot of or at the ground surface. Several feet of engineered fill were required as part of the reclaiming process to provide adequate separation for placement of the lime–soil residual.

This project accepted 61,000 yd^3 (46,000 m^3) of lime. The project is scheduled for an end use of residential housing. No impact has been reported on the adjacent water supply wells west of the land reclamation site.

Active Sand Mine—Shiawassee County, Mich.

This reclamation process was performed at an active sand mine where the active face was progressing in one direction while the land reclamation process proceeded forward to slowly ameliorate the effects of the sand mining operation. This facility was permitted to accept 450,000 yd (34,000 m^3) of lime. Lime was accepted from three sources, including the Lansing Board of Water & Light, the East Lansing Meridian Township Sewer and Water Authority, and the City of Owosso.

Of note are the interesting problems associated with the varying lime qualities and their impacts on the analytical results. In the case of the East Lansing and Owosso sources, the initial analytical testing indicated extremely elevated levels of phosphorus and barium. In both cases, the municipalities initiated programs to assess the source of the phosphorus and barium within the lime residual.

Initially focusing on phosphorus, soil classifications were reviewed and onsite tests were performed in order to classify soils at depths below the proposed fill area to assess phosphorus uptake. Calculations of total phosphorus contained in the permitted yardage were made and compared to the phosphorus uptake for the residual soils at the site.

This exercise of the soil phosphorus uptake potential and sources of phosphorus within the water treatment system proved enlightening; however, in the end, it was unnecessary because of analytical interferences associated with the analytical techniques utilized and matrix interference. Specifically, the initial analytical laboratory utilized ICP in order to determine the phosphorus levels within the lime samples. Review of the analytical results, and testing and retesting by two other laboratories, indicated that the calcium matrix of the lime residual emitted a wavelength identical to those given off by phosphorus and barium. Therefore, in subsequent applications of the process, all chemical analyses were performed utilizing a graphite furnace as opposed to ICP.

Operational Sand Mine—Ypsilanti, Mich.

This 50-acre (220,000-m^2) site was originally permitted for 360,000 yd^3 (272,000 m^3) of lime. The site accepted lime from the Ypsilanti Community Utility Authority and the City of Ann Arbor. Geologically and hydrogeologically, the site is located 0.5 mi (800 m) north and vertically 50 ft (15 m) above the Huron River on the north side of the City of Ypsilanti.

The site soils generally consist of sand overlying clays at depths of 40 ft (12 m). Locally, private wells are utilized for water supply. The site's end use will be residential, possibly with a golf course as the centerpiece of the development.

Four In-place Closures—Ypsilanti Township, Mich.

The reclamation process was utilized to close in-place three lime storage lagoons for the Ypsilanti Community Utility Authority at their Bridge Road WTP and one at their Catharine Street location. In this process, more than 57,000 yd^3 (43,000 m^3) of lime were mixed with soil and replaced at a cost less than an in-place closure covering with a membrane or geofabric. The closure will allow the site to be used for recreational purposes because of the substantial increases in bearing capacity. The site provided substantial engineering hurdles, including the fact that it was located on the downstream side of a dam along the Huron River.

The following conclusions are based on the findings of works of Sheff, Rogers, and Murrell (1997):

- Reclamation can be performed utilizing residual lime material. The work reported that more than 550,000 yd^3 (416,000 m^3) of lime had been utilized to reclaim approximately 170 acres (7.5×10^5 m^2) of land within the state of Michigan.
- From a geotechnical standpoint, maximum allowable bearing capacities of 2,500 lb/ft^2 (12,500 kg/m^2) can be achieved utilizing up to 60 percent lime mixtures.
- Bearing capacities in excess of 3,500 lb/ft^2 (17,500 kg/m^2) are predicted based on soil–lime proportions.
- The resulting lime–soil mixture generally has a low permeability.
- Long-term stability of the lime–soil mixture in a low pH environment is not known.
- This reclamation process has proven to be a viable, cost-effective alternative to landfilling and land application of lime residuals.
- No impacts to groundwater have been observed and, based on the testing performed, are not expected to be associated with these reclamations.

Hydrogen Sulfide Binding

Utility experiences using iron coagulant residuals for control of hydrogen sulfide (H_2S) were documented in *Slib, Schlamm, Sludge* (Cornwell and Koppers 1990). Several experiments, along with some full-scale applications, were performed using iron residuals from WTPs in place of conventional methods using H_2S binding such as ferric chloride addition and biogas washing. Hydrogen sulfide gas is generated from the decomposition of wastewater and causes severe corrosion of concrete and metals. Hydrogen sulfide also has a strong, offensive odor at concentrations as low as 0.1 ppm. Ferric chloride has historically been used to bind sulfides to prevent the formation of H_2S. Researchers in the Netherlands have demonstrated that WTP iron residuals can be equally effective as, or more effective than, ferric chloride for sulfide binding. Four different applications are presented in *Slib, Schlamm, Sludge* that detail experiences using iron residuals for H_2S binding in the Netherlands. These applications are as follows:

1. Sulfide binding in sewer pipes
2. Hydrogen sulfide control in WWTPs
3. Sulfide binding during anaerobic liquid manure processing
4. Sulfide binding in tannery wastewater treatment

Experimentation in the Netherlands using iron residuals for sulfide binding in sewers proved to be very successful. Iron residuals were metered into a sewer system at a dose rate of 31 mg/L. The study demonstrated that the residuals were capable of reducing H_2S by up to 95 percent (Baltussen 1985). Several WTPs in the Netherlands using ferric coagulants discharge backwash wastewater directly into sewers primarily for residuals disposal, as well as for the added benefit of sulfide binding. Reduction of H_2S in sewer pipes significantly reduces corrosion.

The Dutch have also used iron residuals in place of other H_2S reduction methods for reducing odor and corrosion. A regional wastewater authority in the Netherlands, De Dommel/De Aa, conducted a study involving use of iron residuals in the sludge digestion process. The study resulted in the following conclusions (Cornwell and Koppers 1990):

- Iron residuals lowered the H_2S concentration from >600 mg/L to <50 mg/L.
- No adverse effects on the digestion process were noted.
- Use of iron residuals was more cost-effective than other means of sulfide control.

A second study was performed by West-Overijsel, which is a regional wastewater authority in the Netherlands. The West-Overijsel treatment plant used liquid iron residuals (7 percent solids) from a groundwater treatment process for sulfide control. Conclusions from this study were as follows (Cornwell and Koppers 1990):

- Iron residuals were able to retain H_2S levels to <500 mg/L.
- No adverse effects on the wastewater digestion process were noted.
- Use of ferric chloride was eliminated, which decreases the chloride concentration in the WWTP biosolids generated.
- Costs were decreased 50 percent using iron residuals.

Livestock production is a very large industry in the Netherlands, and, as a result, manure wastes are a significant environmental problem. There are several liquid manure processing plants that handle wastes from pig farms. These plants convert pig manure into a dry granular fertilizer for agricultural use. Research demonstrated that the addition of iron residuals into the manure digestion process effectively decreases H_2S production. Excessive H_2S concentrations generated during this process cause corrosion to gas engines (Cornwell and Koppers 1990).

Iron residuals have also been used to reduce sulfide odors in tannery wastewater treatment processes. Sulfides are used in the treatment of animal hides, and, as a result, H_2S is formed in the wastewater EQ basin. The iron residuals were shown to have reduced H_2S production to a limited extent. Further research is being conducted to improve the methods for using iron residuals in this process (Cornwell and Koppers 1990).

Flue Gas Desulfurization

The beneficial use of softening residuals for flue gas desulfurization (FGD) at coal-fired plants is a viable technology. Coal-fired electric utilities typically use finely ground limestone in the FGD process to lower sulfur dioxide (SO_2) emissions produced by burning coal.

Wet-limestone FGD is the most common process used in the United States for removal of SO_2 from power plant flue gas emissions. According to the 1992 U.S. Department of Energy “Utility FGD Survey,” of the 157 utility FGD systems operating in January 1990, 77 used some form of the wet-limestone process (Shannon et al. 1997). With the promulgation of the Clean Air Act, the number of power plants with an FGD system will sharply increase and create the marketing opportunity for softening plant residuals.

In the wet-limestone FGD process, flue gas containing SO_2 is passed through scrubbers where it comes into contact with a spray of slurry containing finely ground limestone in water. The contact between the flue gas and the slurry spray cools and saturates the flue gas and results in adsorption of the SO_2 into the slurry liquid. The slurry falls to the bottom of the scrubber and drains into a reaction tank. Chemical reactions between the calcium carbonate from the limestone and the absorbed sulfuric dioxide result in the precipitation of solid particles of calcium sulfite and calcium sulfate within the reaction tank. Fresh limestone slurry is added to the reaction tank as necessary to maintain the operating pH of the scrubber system. The slurry from the reaction tank is recirculated back to the scrubber by means of a system of process pumps and piping. A sidestream of slurry is withdrawn to allow for separation and disposal of the calcium sulfite and calcium sulfate solids.

Fine grinding of the limestone is important because it increases the surface area of the particles, thus allowing more surface area for more efficient removal of SO_2. The scrubbing process is an acid–base reaction between the acidic flue gas and the alkaline limestone.

The following equations outline the general process that occurs in the wet-limestone scrubbing process:

$$SO_2 + H_2O \rightarrow H_2SO_{3(aq)} \tag{5.21}$$

$$H_2SO_{3(aq)} \rightarrow H^+_{(aq)} + HSO_3^-{}_{(aq)}$$

$$CaCO_{3(aq)} + H^+_{(aq)} \rightarrow Ca^{+2}_{(aq)} + HCO_3^-{}_{(aq)}$$

$$Ca^{+2}_{(aq)} + HCO_3^-{}_{(aq)} + HSO_3^-{}_{(aq)} \rightarrow CaSO_{3(aq)} + CO_2 + H_2O$$

When the flue gas contacts the limestone slurry spray in the scrubber tower, SO_2 is transferred from the gas phase to the liquid phase. The acidity generated by this reaction dissolves the finely ground limestone particles. The dissolved calcium then reacts with the sulfur species to form calcium sulfite ($CaSO_3$), which precipitates as a hemihydrate, $CaSO_3 \cdot \frac{1}{2}H_2O$. The continuing reaction of calcium and sulfite allows more SO_2 to be removed as the flue gas passes through successive contact zones in the scrubber tower.

Because calcium sulfite hemihydrate is difficult to dewater, many scrubber systems blow air into the slurry holding vessels to produce the sulfate ion (SO_4^{-2}):

$$HSO_3^- + \tfrac{1}{2}O_2 \rightarrow SO_4^{-2} + H^+ \tag{5.22}$$

The sulfate reacts with calcium to form a gypsum precipitate, $CaSO_4 \cdot 2H_2O$. The gypsum precipitate is much easier to dewater and handle than calcium sulfite hemihydrate. Moreover, if oxidation is complete in the system, the gypsum may be of high enough quality for use as wallboard material.

In one respect, an FGD system is similar to a clarifier in that it generates a precipitate containing unwanted compounds. Whereas the clarifier removes compounds already dissolved or suspended in liquid, the FGD system must extract a gaseous pollutant.

Two of the common difficulties in scrubber operation are inefficient limestone utilization and low SO_2 removal. In systems that are improperly designed or use low-quality limestone, a significant portion of the limestone passes through the scrubber undissolved. Not only does this increase the amount of raw limestone needed by the system, but it may also cause SO_2 emissions limits to be exceeded.

Limestone quality can have a dramatic impact on the dissolution properties of the limestone. Typical limestones are composed of calcium carbonate and magnesium carbonate ($MgCO_3$) with lesser quantities of inerts such as silicon oxide compounds. Small amounts of iron and manganese carbonates may also be present. Limestones containing 95 percent or greater calcium carbonate are usually considered desirable in FGD systems because high-purity limestone is

Table 5.18
Overall composition of residuals and limestone samples

		Residuals			
	Beachville limestone*	Water District No. 1 of Johnson County, Kan.	City of Lawrence, Kan.†	City of Topeka, Kan.	LEC limestone (raw)
Ca (mmol/g)	9.68	8.42	8.10	9.00	8.06
Mg (mmol/g)	0.14	0.88	0.53	0.35	0.35
CO_3 (mmol/g)	9.80	8.55	8.13	8.80	8.63
$CaCO_3$ (wt %)	96.9	84.3	81.1	90.1	80.7
$MgCO_3$ (wt %)	1.2	7.4	4.5	3.0	2.9
Inerts (wt %)	1.2	3.7	9.7	2.5	12.5
Salts (wt %)	NA‡	2.4	2.8	1.3	NA
Closures					
Weight (%)	–0.7	–2.2	–2.0	–3.2	–3.9
Molar (%)	0.1	4.2	3.0	3.0	–1.3
Slurry pH§	NA	9.45	9.25	9.80	NA

Source: Shannon et al. 1997.
*Beachville limestone fineness: 90 percent passing 325 mesh.
†Lawrence residuals were from the Clinton Water Treatment Plant (Lawrence, Kan.).
‡NA = not available.
§Reagent $CaCO_3$ pH = 8.55.

highly reactive. Likewise, magnesium carbonate, in its pure form, is also reactive in acidic solutions. The problem with limestones containing significant quantities of $MgCO_3$ is that the magnesium and calcium usually coexist with carbonate in the same crystal lattice. This product, known as dolomite, is relatively unreactive in FGD systems. Scrubbers using dolomite typically show reduced limestone utilization and reduced SO_2 removal efficiencies.

Overall composition results are shown in Table 5.18. This table shows calcium, magnesium, carbonate, and acid-inerts concentrations for a control limestone (Beachville), the three residuals samples, and the current Lawrence Energy Center (LEC) limestone, all located in Kansas. In addition, the total weight percent of soluble salts other than calcium and magnesium is listed for the three residuals. Finally, Table 5.18 shows the pH of a water slurry of the three residuals samples.

Significant results from Table 5.18 include the following:

- $CaCO_3$ concentrations of the residuals varied from 81.1 to 90.1 percent. Lawrence had the lowest $CaCO_3$ concentration due to a relatively high level of inerts (9.7 percent). The inerts for the residuals samples are composed primarily of silicon, aluminum, iron, and carbon.
- The LEC limestone contained 80.7 percent $CaCO_3$. This is lower than any of the residuals and is the result of a high concentration of impurities. Specifically, the LEC

limestone contained 12.5 percent acid inerts, which is a high level of inerts for an FGD limestone.

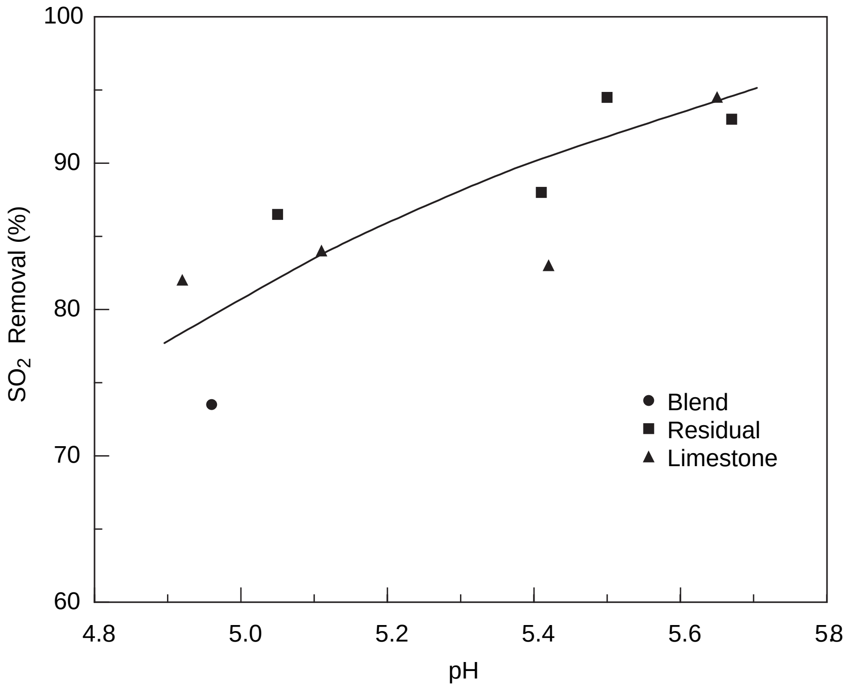
- $MgCO_3$ concentrations in the residuals varied from 3.0 to 7.4 percent. As shown later, most of this magnesium is soluble. The LEC limestone has a lower magnesium concentration of 2.9 percent.
- The carbonate concentration of both limestones (standard Beachville limestone and LEC limestone) and the three residuals was measured. For the limestones, the carbonate concentration was approximately equal to the sum of calcium and magnesium on a molar basis, as expected. The carbonate concentrations for the residuals samples, however, were approximately equal to the calcium concentration but well below the sum of calcium and magnesium. These results suggest that calcium is present as calcium carbonate and magnesium is present as magnesium hydroxide in the WTP residuals.
- The pH of a water slurry of the three residuals was measured. The results showed that the pH of the residuals was above that for reagent-grade calcium carbonate (8.55) but below that of lime (12.5). This confirms the presence of magnesium hydroxide.

SO_2 removal efficiency is obviously one of the most important performance indicators for an FGD system. Because the WTP softening residuals are more reactive than limestone, it was anticipated that the SO_2 removal performance would improve.

Figure 5.10 (Shannon et al. 1997) shows SO_2 removal as a function of scrubber slurry pH. This figure shows that the SO_2 removal achieved at a given pH is relatively independent of the reagent used. Similar results have been observed in other FGD systems when the reagent type or characteristics were changed. For example, the relationship between SO_2 removal and pH in other systems stays the same even with limestones having differing reactivities and/or particle size distributions. One conclusion from these results is that the pH control set point required to achieve a given SO_2 removal level is independent of reagent type.

Results from the work by Shannon et al. (1997) are summarized below:

- The residuals were shown to be more reactive and dissolved faster than the limestone, resulting in greater reagent utilizations for a given pH.
- The SO_2 removal achieved with the residuals (at a given reagent utilization) was better than that achieved with limestone.
- Screening analyses of the residuals and limestone showed substantial differences in particle size distributions. Residuals were finer than ground limestone with 91.7 percent passing a 325-mesh sieve, while limestone had only 66.6 percent passing a 325-mesh sieve.
- The scrubber slurry cake formation rate—the rate at which a visibly dry cake forms on the dewatering filter media—was lower than what is typical for FGD systems both when using limestone and when using residuals. This is attributed to the scrubber design that utilizes higher concentrations of fly ash.
- Aluminum fluoride blinding, a phenomenon that inhibits calcium carbonate dissolution in scrubber slurry, was observed in all of the tests, regardless of whether residuals were present. The aluminum fluoride blinding is attributed to the high concentrations of fly ash. Therefore, it was not possible to determine whether residuals



Source: Shannon et al. 1997.

Figure 5.10 SO_2 removal versus scrubber slurry pH, LEC

would cause aluminum fluoride blinding in systems that do not have high fly ash concentrations.

- The existing limestone handling system at LEC needs to be modified before the plant could switch to using residuals as makeup reagent.

CHEMICAL RECOVERY AND REUSE

Actual practice of the recovery of chemicals from water plant residuals has centered around the production of lime from lime-softening residuals and the recovery of alum or iron from coagulant residuals. The objective of chemical recovery is generally a combination of producing the recovered chemical at a price less than the commercial price (thus representing a chemical cost savings to the plant) and at reducing the quantity of waste product requiring treatment or disposal (thus saving on residuals handling costs). Processes for recovering chemicals from both types of residuals have and are being utilized; however, each has found only limited application.

Reclamation of Lime From Lime-Softening Residuals

Lime recovery is accomplished by the process of recalcination. In the basic process of recalcination, the lime residuals—consisting of primarily $CaCO_3$—are dewatered and burned, producing calcium oxide (quicklime).

In 1938, in a discussion of a paper by W.W. Aultman (as reported by Wertz 1960) on reuse of lime in softening, Charles P. Hoover of Columbus, Ohio, called attention to problems associated with the likely success of recalcination for many water suppliers. He pointed out the problems for producing an acceptable product for plants with high magnesium or suspended particles. He added that Miami (Fla.), with the largest softening plant in the country using well water, had the best chance to accomplish recalcination of lime at a reasonable cost. Early in 1938, Aultman conducted classical studies on the recalcining of lime for the Metropolitan Water District of Southern California.

In 1941, A.P. Black, who was employed as a consultant for the Miami (Fla.) Water Plant, first suggested to the Miami Water and Sewer Board that they should consider recalcination. A plant was eventually completed in December 1948, which was the first full-scale recalcination plant. In this process, the residuals were first thickened in a circular gravity thickener with residuals scrapers to about a 20 percent solids concentration. This slurry was dewatered by centrifugation to a 66 percent solids concentration. The dewatered cake was fed to a rotary kiln where the cake was heated to 2,100°F (1,149°C). The stack gases contained about 25 percent carbon dioxide, which was sent to the recarbonation basin of the softening process for pH reduction.

Every mole of lime added to remove a mole of calcium carbonate hardness produces two moles of $CaCO_3$ sludge:

$$CaO + Ca(HCO_3)_2 = 2CaCO_3 + H_2O \tag{5.23}$$

In recalcination, this calcium carbonate is then dewatered and heated to produce lime:

$$CaCO_3 + \text{heat} = CaO + CO_2 \tag{5.24}$$

Therefore, for every original mole of CaO added to the softening process, theoretically, two moles of CaO are produced in recalcination. In actual practice, only about 20 percent excess lime is produced.

As recalcination has been applied, various alternatives have been utilized, as shown in Figure 5.11. One of the problems that had inhibited more widespread use of recalcination was that impurities in the residuals either made the recovery of lime inefficient or the resulting product was not of high quality. These contaminants, which are not volatilized during calcination, will increase with recycle and reuse, causing problems both in the slaking process and in efficient calcination. The primary impurity present in groundwaters affecting calcination is magnesium and sometimes silica. Surface waters will also have suspended solids and coagulant hydroxides if the latter are used in the treatment process.

The first step for many plants practicing recalcination is a purification process, as shown in Figure 5.11. The most common method of eliminating impurities from the calcium carbonate residuals is one- or two-stage centrifugation. Centrifugation uses the specific gravity difference between the calcium carbonate and the impurity to make the separation. Primarily, this procedure has been used to separate magnesium hydroxide from the calcium carbonate, although at least one study separated silt by this procedure. In both cases, the calcium carbonate is heavier and moves to the wall of the centrifuge whereas the magnesium hydroxide or silt has lower specific gravity and is lost in the centrate. The primary disadvantage of this method is that some calcium carbonate is also lost in the centrate, depending upon the amount of impurity present and the required degree of classification. Table 5.19 shows data for the separation achieved for one such residuals sample. This particular residuals sample was fairly low in impurities with an initial $CaCO_3$ purity of about 85 percent. A 30 percent reduction in magnesium was achieved, a 42 percent reduction in iron, and a 78 percent reduction in aluminum. The loss of $CaCO_3$ to the

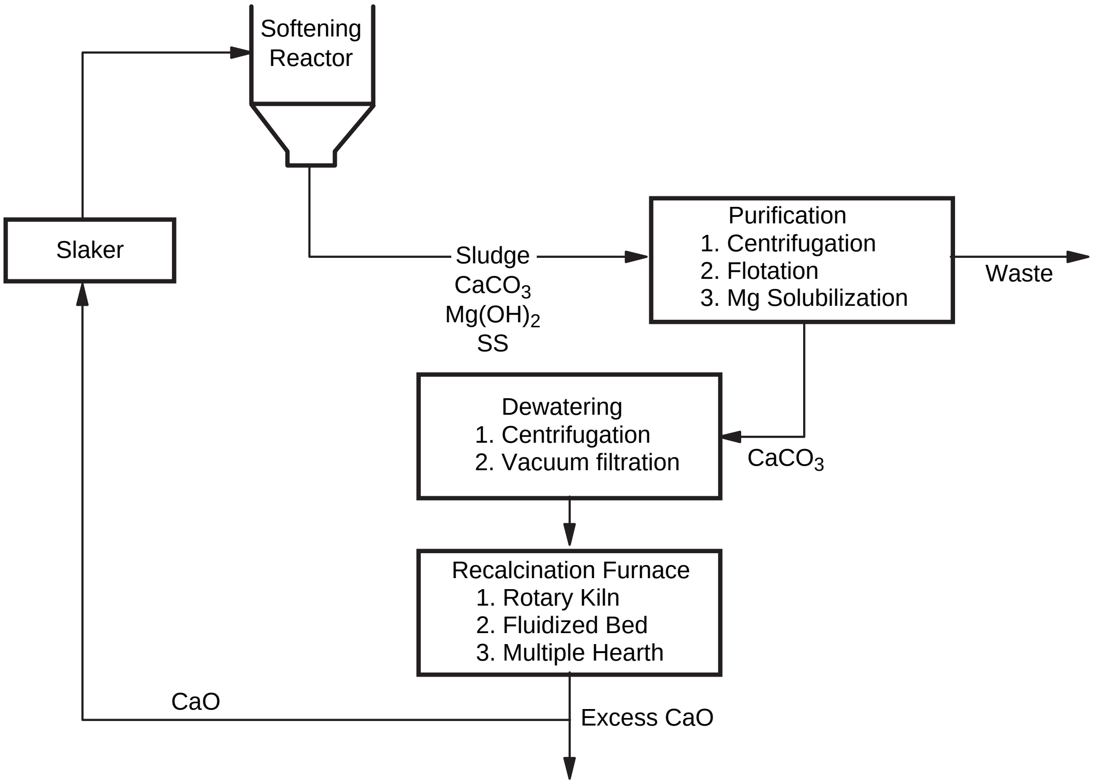


Figure 5.11 Process options for lime recovery from lime-softening sludge

Table 5.19
Effectiveness of centrifugation in removing impurities from $CaCO_3$ sludge

Parameter	Feed to centrifuge (%)	Cake from centrifuge (%)	Reduction in cake (%)
Moisture	85.4	33	NA*
Solids concentration	14.6	67	NA
Magnesium	1.9	1.3	30
Iron (as Fe_2O_3)	0.85	0.49	42
Aluminum (as Al_2O_3)	8.0	1.8	78

Source: Adapted from Sheen and Lammers 1944.
*NA = not applicable.

centrate was only about 10 percent, again reflective of the relatively pure original residuals sample. The purified cake for the example shown is equivalent to about 96 percent $CaCO_3$. It has been estimated that at least a 91 percent grade of $CaCO_3$ is needed to be suitable for feed to the recalcination step.

When the magnesium content is higher, a greater degree of separation is needed and more of the $CaCO_3$ is lost. In these cases, it may be appropriate to remove the magnesium by selective dissolution. This is accomplished by mixing the CO_2 from the recalcination stack gases with the residuals. For example, when the plant in Lansing, Mich., practiced recalcination, it lowered the pH of the sludge to 9.0 using the CO_2 and thereby reduced the magnesium oxide content of the cake from 3.5 percent to 1.8 percent. Solubilization takes place by way of the reaction:

$$Mg(OH)_2 + 2CO_2 = Mg(HCO_3)_3 \tag{5.25}$$

The separated magnesium carbonate can then be precipitated with lime, dewatered, and disposed of. Alternately, the liquid could be heated to 35 to 45°C using heat recovery from the recalcination process to produce magnesium trihydrate, which is usable as a coagulant:

$$Mg(HCO_3)_2 + 2H_2O + \text{heat} = MgCO_3{\cdot}3H_2O + 2CO_2 \tag{5.26}$$

Following purification (if needed) and dewatering of the calcium carbonate, the cake is flash-dried and burned. Available furnace types include a rotary kiln, flash calciner, fluidized bed, and multiple hearth.

The economics of applying the process primarily depends upon the cost of fuel necessary to calcinate the sludge. The fuel consumption is in the range of 8,500,000 to 12,000,000 Btu per ton of CaO produced. Number 2 fuel oil has a heat value of 141,000 Btu/gal (11 kW·hr), so that 60 to 90 gal (230 to 340 L) of No. 2 fuel oil are required per ton of CaO produced.

An additional factor that should be considered in the economics of lime recalcination is disposal costs. Particularly those plants that have radium in the lime sludge may find disposal very difficult and recalcination a viable alternative. However, caution should be exercised as data are not available on the fate of radium in the calcination process.

The removal mechanism for radium in lime softening is unknown. If the radium is in some way associated with the magnesium hydroxide, then it may be removed from the calcium carbonate during the purification step. However, if it exists as a precipitate such as radium carbonate ($RaCO_3$) (in which case it has a high specific gravity) or is sorbed with the $CaCO_3$ itself, then it will remain in the cake during centrifugation and hence be calcined. Based on physical data, the radium could vaporize during calcination. Radium has a boiling point of 2,060°F (1,130°C), which is very close to the 2,100°F (1,150°C) used for calcination. A plant considering recalcination with radium in the residuals should consider the fate of radium, both to eliminate a buildup and to ensure proper disposal of the waste streams.

The City of Dayton Division of Water Supply and Treatment facilities supply drinking water to approximately 440,000 people in Montgomery County, Ohio. Source water is supplied by wells in the Miami and Mad River well fields, which have been classified as GWUDI by the Ohio Environmental Protection Agency. Wells pump water from the aquifer to two water treatment facilities: the Ottawa Water Treatment Plant and the Miami Water Treatment Plant. Each of these is a precipitative softening plant, and each plant has a rated capacity of 96 mgd (360 ML/d). Lime (calcium oxide), fluoride, and chlorine are used for water treatment, with rapid sand filtration being the final step in the treatment process. In 2003, Dayton's two water plants treated and pumped 26 bil gal (9.8×10^{10} L) of water, which is an average greater than 71 mgd (270 ML/d).

Dayton's well water is considered very "hard" because of the natural minerals. With a hardness of 345 mg/L, the lime precipitative process is needed to reduce the hardness to 150 mg/L before pumping to customers. During the water treatment process, hydrated lime (calcium hydroxide) reacts with hardness causing minerals in the well water to produce calcium carbonate and magnesium hydroxide residuals that settle to the bottom of the sedimentation basins. These residuals are pumped to a Lime Recovery Facility (LRF). Ottawa WTP residuals are pumped directly to the adjoining LRF. Miami WTP residuals are first pumped to two EQ tanks.

The wasted calcium carbonate/magnesium hydroxide lime-softening residuals are pumped to an LRF. The LRF is located next to the Ottawa Water Treatment Plant, and its purpose is to recover the lime used at the two WTPs. To prepare water softening residuals for recovery at the LFR, carbon dioxide that is produced by the lime recalcination process selectively dissolves and separates magnesium from the residuals. After carbonation, the result is relatively pure calcium carbonate, which is dewatered in centrifuges and pumped into a rotary lime kiln. During the recalcination process, temperatures in excess of 2,000°F (1,100°C) convert the calcium carbonate into calcium oxide and carbon dioxide. The carbon dioxide is used for carbonation of lime-softening residuals and pH adjustment of drinking water at the adjacent WTP. The final product of the recalcination process is calcium oxide (lime).

The lime kiln produces calcium oxide in pebble form. This material is conveyed pneumatically from the LRF to the adjacent Ottawa WTP and is trucked to the Miami WTP. At the water plants, this lime is mixed with water in slaking machines to create calcium hydroxide for water softening. The excess lime is sold to a broker for $50 per ton, which further reduces operating costs.

Reject water from the LRF centrifuges is sent to Dayton's WWTP by way of the sanitary sewer. The reject water, which contains calcium carbonate, conditions the wastewater sludge and helps to improve that operation. When the LRF is out of service, the lime-softening residuals from both WTPs are pumped to a lagoon. Solids, primarily calcium carbonate and magnesium hydroxide, are dredged from the lagoon and applied to farmland for pH adjustment of soil.

Iron and Alum Recovery

Methods for recovery of coagulants from WTP residuals have been investigated since the turn of the century. Most of the studies have focused on alum recovery, although methods have also been investigated to recover iron.

As outlined by Roberts and Roddy (1960), the earliest attempt to reclaim alum residuals was made by Jewel, who in 1903 patented a process for water treatment and for reclaiming the coagulant by reacting the aluminum hydroxide with sulfuric acid. Mathis, in 1923, was issued a patent for basically the same process as developed by Jewel. Black Laboratories, of Orlando, Fla., in 1951 suggested the use of an alum sludge recovery process to utilize the SO_2 gas from boiler stacks as a source of sulfuric acid. Some of the first reported alum recovery in the water treatment field was by Palin (1954). Palin's work was conducted at the Whittle Dene Waterworks of Newcastle, England. In his first set of experiments, filter wash water was treated with 0.05 and 0.1 percent (by volume) sulfuric acid. Chloride was added to oxidize the color present in the dilute recovered alum solution. This recovered alum was then used in conjunction with commercial-grade alum in order to determine the amount of commercial-grade alum needed to coagulate the raw water and lower the color of the finished water to 10 Hazen. Palin found that in treating raw water 28 ppm commercial alum was needed, whereas treating raw water plus 3 percent by volume recovered alum, only 11 ppm commercial alum was needed. Despite this large reduction in alum dosage, the cost of acid used was higher than the cost of alum saved. Palin reported superior results when the residuals were charred at 400°C before acid treatment. It was found that 1 ton (907 kg) of oven-dried sludge would yield 2 tons (1,814 kg) of aluminum sulfate cake (14 percent Al_2O_3) upon addition of about 0.9 tons (816 kg) of 98 percent H_2SO_4.

In Tampa, Fla., Roberts and Roddy (1960) studied the recovery of alum in both pilot- and full-scale processes. The alum residuals were thickened by settling for 3 hours. The solids content reached 1 percent in pilot-plant scale and as high as 2 percent in full-scale operation. The residuals samples were reacted with enough sulfuric acid to convert the aluminum hydroxide to aluminum sulfate. The amount of acid used varied depending upon the alkalinity of the raw water. The pH range for complete aluminum dissolution was between 1.5 and 2.5 for highly alkaline and less alkaline waters, respectively. After the reclaimed alum was recycled 10 times, there was no reported reduction in finished water quality. It was estimated that chemical costs could be reduced by 70 percent using the acid recovery method. However, the process was never utilized because of subsequent concerns for handling the large volume of dilute alum and the recycle of organics that originate in the highly colored raw water.

In 1969, Isaac and Vahidi studied alum recovery for a method of residuals disposal. They tested both alkaline and acid methods of aluminum recovery. They found that aluminum recovery with caustic soda was not very satisfactory. It was also observed that organic matter bound with the aluminum hydroxide, especially organic color, was much more soluble in alkali than in acid. It was therefore decided to use the acidic method for aluminum recovery. Aluminum was recovered from fresh residuals and from anaerobically digested sludge. Tests were then conducted to determine the volume occupied by the residuals after acid treatment. At a pH of 2.5, corresponding to 79 percent aluminum recovery, a 74 percent volume reduction of residuals was obtained. Recovered alum was usually about 75 percent as efficient as fresh alum in reducing color, although one test resulted in an efficiency of 89 percent. The researchers concluded that the pH should be lowered to about 3.0 for a recovery of about 60 to 65 percent of the aluminum because the organic color was not dissolved to an excessive extent at this pH.

In laboratory experiments, Webster (1966) found that if sulfuric acid was added to alum residuals to depress the pH value to about 2.4, a clustering effect of the floc particles took place with extremely rapid settling of the insoluble matter. The supernatant liquor contained the alum, representing about 80 percent recovery. A pilot plant for alum recovery was then constructed. Good coagulation was not obtained with recycled alum that had been recovered at a pH of <3.0. Webster concluded that the alum reduced the pH of the raw water below the range for acceptable color removal. Therefore, the pH of the sludge was reduced to 3.5 for alum recovery and reuse. No detrimental effects resulted from continued recycling of the alum recovered at a pH of 3.5.

Streicher (from AwwaRF 1969) conducted pilot tests to determine the usefulness of acid recovery of aluminum followed by filter pressing the remaining residuals. The pH was reduced to between 1.5 and 2.5 by sulfuric acid. He found that when the ratio of $Al(OH)_3$ to other suspended matter in the residuals was high, considerably less than stoichiometric amounts of sulfuric acid were required. If the ratio was low, more than stoichiometric amounts of acid were needed. Acid treatment resulted in reduction of the residuals volume to <10 percent of the original volume, and the solids concentration of the settled residuals reached 20 percent. The alum recovery was 80 to 93 percent. With the use of a filter press, the remaining residuals were concentrated to 40 to 50 percent solids concentration.

Fulton (1974) described an alum recovery system scheduled to be put into operation in 1974 at Jersey City, N.J. The process consisted of thickening, acid addition, and filter pressing of the resulting residuals. The acid recovery could be bypassed and only the filter press used if necessary. For a 100-mgd (380-ML/d) plant, the savings was estimated at $4.60 per million gallons when alum recovery was used. An alum recovery of 90 percent was estimated. The process has not been utilized.

Table 5.20
Examples of residuals characteristics applicable to alum recovery

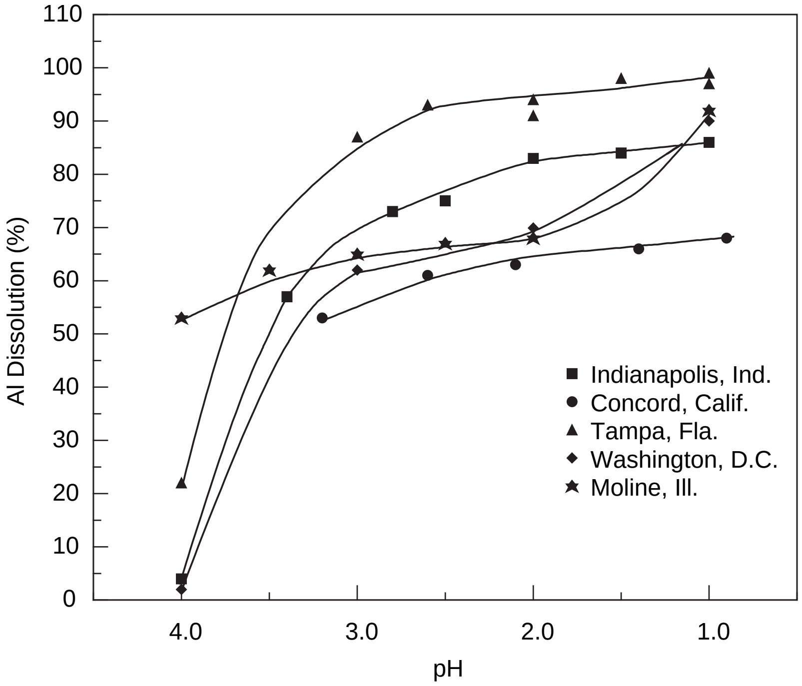
Parameter	Washington, D.C. (%)	Indianapolis, Ind. (%)	Concord, Calif. (%)	Tampa, Fla. (%)	Moline, Ill. (%)
Dissolved inorganic solids	9	26	36	61	7
Nondissolvable inorganic solids	49	52	18	6	79
Dissolvable organic solids	26	12	18	25	2
Nondissolvable organic solids	16	9	28	8	13
Solids concentration	12.1	1.7	1.7	1.6	1.7

Source: Adapted from Cornwell and Susan 1979.

Westerhoff (1973) conducted a 15-week pilot-plant study to determine the effect of recycling alum recovered from waste alum residuals by an acidic process. The pH of the sludge was reduced to 2.0 for conversion of aluminum hydroxide to aluminum sulfate. The main purpose of the study was to evaluate potential contaminant buildup in the recycled alum. Measurements were made on total microscopic count, coliform, hardness, alkalinity, cyanide, fluoride, phenol, dissolved solids, nitrates, sulfates, chlorides, and several metals such as copper, lead, and zinc. Throughout the study, final water analyses for the pilot plant using recycled alum and for the full-scale plant using fresh alum were essentially the same, indicating that impurities were not built up by recycling alum.

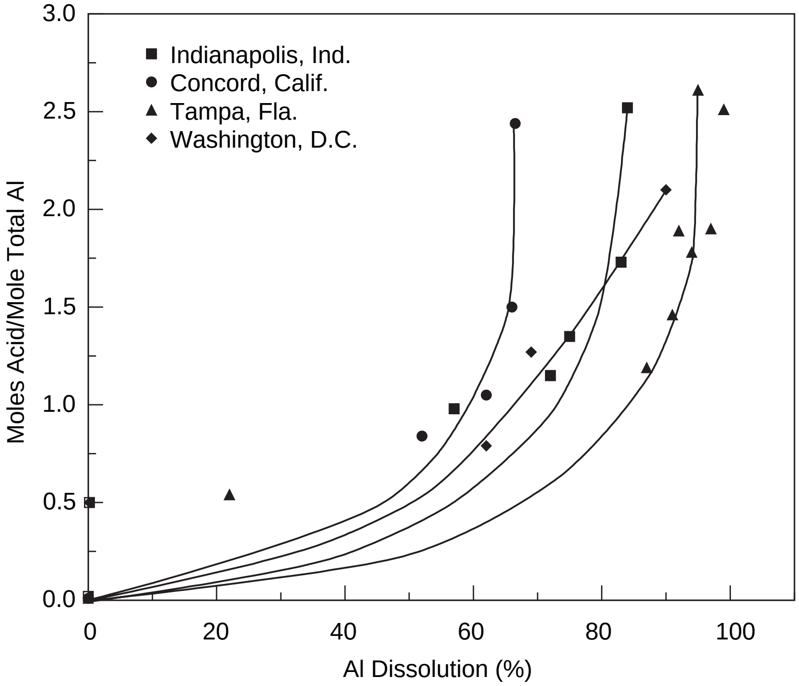
Westerhoff and Daly (1974) conducted a complete study of various alum residuals dewatering facilities. They tested pressure filtration with and without alum recovery, centrifugation, rotary vacuum filtration, horizontal vacuum filtration with and without alum recovery, coagulation, filter press, and freeze–thaw. The studies showed alum recovery followed by horizontal vacuum filtration to be a workable process warranting economic evaluation. The recovery of alum varied from 50 to 90 percent. Coagulation basin residuals were thickened from an initial 4 to 6 percent solids to a final 21 percent solids by acid treatment. After filtration, the solids content was 37 percent. However, because of the low alum dosage used for raw water turbidity removal, the most economical method of alum residuals treatment was determined to be pressure filtration without alum recovery.

Cornwell and Susan (1979) reported laboratory experiments on sulfuric acid dissolution of aluminum and the characteristics of the remaining solids. They conducted studies on five different residuals (characteristics of the sludges are shown in Table 5.20). The sludge obtained from Washington, D.C., was high in nondissolvable solids associated with the raw water suspended solids content, with a relatively low aluminum hydroxide content as shown by the dissolvable inorganic solids concentration. The residuals from Indianapolis, Ind., and Concord, Calif., are both from raw waters with low color and turbidities in the 40-ntu range. Concord uses a higher ratio of alum to turbidity, as reflected by the percentages shown. Tampa, Fla., is a highly colored, low-turbidity raw water, and Moline, Ill., uses alum and a high dose of lime in its treatment process. Figure 5.12 shows aluminum dissolution as a function of pH. Except for two of the residuals, maximum dissolution was obtained at about pH 2. Figure 5.13 shows the acid demands to achieve the aluminum dissolution. As predicted by theory, 1.5 mol of sulfuric acid are required per mole of aluminum



Source: Cornwell and Susan 1979.

Figure 5.12 Aluminum dissolution from alum sludges as a function of pH



Source: Cornwell and Susan 1979.

Figure 5.13 Stoichiometric acid demand for aluminum dissolution from alum sludges

dissolved. The percentages of aluminum content of the residuals were determined by a total digestion of the residuals, and therefore reflect aluminum associated with the precipitated aluminum hydroxide as well as aluminum complexed with the naturally occurring clay particles. The aluminum associated with the clay is generally not dissolved in the pH range used for alum recovery.

Lindsey and Tongkasame (1975) developed a method to purify the reclaimed alum that results from acidifying alum residuals by using UF. UF is a pressure-driven membrane process. The theory of operation is that by applying 20 to 60 psi (138 to 415 kPa) to the reclaimed alum, the water molecules and small aluminum molecules would pass through the membrane whereas larger color molecules would not pass through. In this way, color would be removed from the recovered alum solution. Little results were presented on the exact aluminum passage and color (TOC) rejection; however, the implication was that the results were favorable. Concern did exist, however, as to whether the flux rate could be maintained at an acceptable level to be economical.

Pigeon, Linstedt, and Bennett (1978) evaluated iron recovery from residuals produced when iron was the coagulant. They investigated a combination of acid and sulfide to solubilize the iron. The purpose of the sulfide was to reduce ferric iron to the more soluble ferrous form:

$$2Fe(OH)_3 + 3Na_2S = 2FeS + S + 6NaOH \tag{5.27}$$

The results did show that sodium bisulfate addition would enhance the recovery of iron and that iron recovery could be achieved at a higher pH than using acid alone. However, caution would be advised for the control of H_2S and the effectiveness of the recovered ferrous iron as a coagulant.

Sen Gupta et al. (1997) studied a selective alum recovery process using a special class of composite ion-exchange materials (CIMs). In the CIMs, ion exchanger beads of sizes <100 µm are physically enmeshed in a thin sheet of highly porous polytetrafluoroethylene. Because of the physical configuration of the CIMs, the ion exchanger beads can be inserted into and withdrawn from the residuals. The process studied essentially involves two steps: Step 1—the selective sorption of aluminum from residuals onto CIMs; Step 2—the regeneration of the CIMs using sulfuric acid. Aluminum present in the residual is essentially recovered as alum (or aluminum sulfate) after the second step. The functional groups of the polymeric ion exchanger beads can be chosen in a way to improve aluminum selectivity over other undesirable constituents, namely, natural organic matter (NOM), heavy metals, and others. Two commercially available CIMs with different functional groups (chelating and strong acid) were used during this study. The specific goals of the project were, first, to study the effectiveness of the process pertaining to aluminum recovery, aluminum selectivity, and the efficiency of regeneration; second, to characterize the physicochemical properties of the CIMs and their susceptibility to fouling in a residual medium; third, to evaluate the effectiveness of recovered alum as a coagulant; and fourth, to develop a general design algorithm for the two-step process.

Both the chelating IX material and the strong acid IX material were capable of recovering aluminum. However, the aluminum sorption capacity of the chelating material diminished sharply as the pH of the residuals dropped with an increased number of cycles. On the contrary, the aluminum uptake by the strong acid material remained practically unchanged even at a pH as low as 1.0. The strong acid material, therefore, did not require any pH adjustment during the entire course of the two-step process. Compared to the chelating material, the strong acid material offered greater selectivity in recovering aluminum. The total capacity of the strong acid material (0.12 meq/cm^2) was greater than that of the chelating material (0.045 meq/cm^2).

Both the strong acid and chelating material were found to be quite robust. The aluminum uptake capacities of these materials dropped during the first 10 cycles, but no further significant reduction in capacity and/or progressive fouling were observed even after 200 cycles of operation. The aluminum-loaded strong acid IX material was found quite amenable to efficient regeneration with 10 percent.

Experimental results confirmed that the two-step process could recover more than 80 percent of the aluminum. For an aluminum content of 2,000 mg/L as aluminum, approximately 20 in.^{2} (129 cm^2) of the strong acid material per liter of residual was able to achieve such a high recovery.

Cornwell et al. (1993) reported on the results of a full-scale alum recovery process at Durham, N.C. The Williams Water Treatment Plant has a design capacity of 22 mgd (83 ML/d) and produces 780 tons/year (7×10^5 kg/L), about 2 tons/d (1,800 kg/d), of alum sludge. Peak sludge production can be about twice the annual average. The turbidity averages 30 ntu, but monthly averages can be almost 60 ntu. Color is also in the 30 cu range and the raw water TOC is 4 to 5 mg/L. Raw water iron and manganese average 4 and 1 mg/L, respectively.

Sludge from the sedimentation basins is withdrawn on a regular basis into one of two decant tanks. Polymer is added to the decant tanks and the sludge is thickened to about a 4 percent solids concentration through a batch settling and decanting process. Thickened sludge is pumped to one of two acidification tanks. Sulfuric acid is added to the thickened sludge to lower the pH and dissolve the aluminum. The remaining solids are allowed to settle in the acidification tank, and the decant (recovered alum) is pumped to a holding tank. The thickened, remaining acidified solids are pumped to centrifuges for dewatering. Dewatered solids are mixed with lime and disposed of with the city's wastewater sludge or directly landfilled.

As a result of acidifying the sludge and recovering the aluminum, there is a reduction in solids that needs to be dewatered and disposed of. The sludge reduction can represent a significant savings in dewatering capital costs and operating requirements, and ultimate disposal costs. After the alum was decanted from the acidification tank, the remaining sludge volume was recorded with a liquid level sensor. The remaining sludge was then mixed and a sample taken for suspended solids analyses. Using this procedure, the weight of solids remaining after acidification could be determined.

The amount of sludge reduction that can be achieved is a function of the amount of aluminum hydroxide solids in the sludge, which is therefore related to the raw water suspended solids (turbidity) and the alum dose. The higher the alum-dose-to-raw-water-turbidity ratio, the higher the solids reduction. The percent reduction achievable can be calculated as

$$\text{percent reduction} = \frac{(100)(0.44)(\text{alum dose})}{(0.44)(\text{alum dose}) + \text{turbidity}} \tag{5.28}$$

Table 5.21 shows that the actual average solids reduction achieved was 35 percent, essentially the same as estimated from the calculation in Equation 5.28. Also, Table 5.21 shows that the average solids concentration of the remaining solids was 8.6 percent with a range of 5.4 to 11 percent. The volume of sludge remaining after decanting the alum was about one third of the original volume.

In the sludge treatment process, there are two locations where aluminum is recovered. The first location where recovered alum is obtained is from the acidification tanks. After acidification,

Table 5.21
Sludge reduction obtained by alum recovery at Durham, N.C., WTP

Batch no.	Raw sludge processed (tons)	Acidified sludge remaining (gal)	Solids acidified sludge (%)	Solids remaining (tons)	Solids reduction (%)
2	11.04	27,560	5.4	6.21	
3	11.72	25,970	6.4	6.93	
4	10.47	28,620	6.1	7.28	
Subtotal	33.23			20.42	39
5	12.60	18,020	8.3	6.24	
6	12.67	22,790	6.7	6.37	
7	15.70	29,150	9.4	11.43	
8	14.12	23,320	11.0	10.70	
Subtotal	55.09			34.74	37
9	14.12	22,790	9.0	8.55	
10	14.12	29,680	10.4	12.87	
11	14.64	25,440	9.3	9.87	
Subtotal	42.88			31.29	27
12	15.11	25,440	10.4	11.02	
13	15.87	22,790	9.7	9.22	
14	13.44	22,000	9.8	8.99	
15	13.63	19,600	8.8	7.19	
Subtotal	58.05			36.42	37
Average			8.6		35
Calculated average					33

Source: Cornwell et al. 1993.

the solids are allowed to settle and the decant is sent to the recovered alum storage tank. The second recovered alum stream can be the centrate from the centrifuge.

The recovered alum concentration averaged about 3.2 percent (as alum) or about 15 times more dilute than commercial alum. About two thirds of alum that can be recovered is recovered in the decant. The remaining recovered alum is the centrate from the dewatering operation.

The amount of alum recovered was compared to the amount of alum used in coagulation. The amount of alum used in coagulation was determined from the operating records from the daily alum dose and water treated. Alum recovery for the 2 months studied at steady state was 85 and 114 percent, or nearly an average of 100 percent. Variations occurred because not all the sludge produced in a given month was processed that same month.

One of the direct benefits in reducing the sludge quantity is that the dewatering device can be smaller. In this case, about 35 percent less sludge had to be dewatered because of the alum

Table 5.22
Centrifuge performance on acidified alum sludge at Durham, N.C., WTP

Batch no.	Solids cake (%)	Solids centrate (%)	Solids recovery (%)
1	33.7	0.40	95.6
2	33.9	0.36	97.2
3	36.9	0.36	97.7
4	36.7	0.30	97.5
5	42.9	0.50	96.3
6	38.1	0.10	99.2
7	36.5	0.10	99.3
8	35.4	0.10	99.3
9	25.8	0.10	99.4
10	33.5	0.10	99.2

Source: Cornwell et al. 1993.

recovery. For the City of Durham's Williams WTP, that meant that one less centrifuge had to be purchased.

An additional benefit of recovering the aluminum is that the remaining solids dewater to a higher solids concentration. Table 5.22 shows the centrifuge results from each of the batches tested. When dewatering straight acidified sludge, cake solids concentrations in the range of 35 percent were obtained. Solids recovery was consistently >95 percent and was >99 percent for the last five batches. Polymer doses ranged from 5 to 8 lb/ton (2.5 to 4 g/kg) and averaged about 7 lb/ton (3.5 g/kg).

REFERENCES

AWWA (American Water Works Association). 1981. Lime Softening Sludge Treatment and Disposal. Sludge Treatment and Disposal Committee Report. *Jour. AWWA*, 73(11):600.

AWWA (American Water Works Association). 1987. Research Needs for Alum Sludge Discharge. Sludge Treatment and Disposal Committee Report. *Jour. AWWA*, 79(6):99.

AWWA (American Water Works Association). 1991. State Regulatory Approaches for Water Plant Waste. Water Treatment Plant Waste Disposal Committee. Paper presented at the AWWA Conference and Exposition Preconference Seminar, Philadelphia, Pa., June.

AwwaRF (Awwa Research Foundation). 1969. Disposal of Wastes From Water Treatment Plants—Part 2. Committee Report. *Jour. AWWA*, 61(11):619.

Baltussen, J. 1985. Research Into the Prevention and Combating of Hydrogen Sulfide Gas at the Point Where the Erp Pumping Station Pressurized Delivery Pipe Joins the Under Transport Sewer. GTD Oost-Braant Internal Report, 85-ZTA-12. Boxtel, The Netherlands.

Bugbee, G.J., and C.R. Frink. 1985. *Alum Sludge as a Soil Amendment: Effects on Soil Properties and Plant Growth*. Bulletin 827. New Haven, Conn.: Connecticut Agricultural Experiment Station.

Ceratech. 1998. Brick Making [Online]. Available: http://www.ccex.org/tables/projects.htm. [cited August 13, 1998]

Cornwell, D.A., T. Bailey, A.T. Rolan, and K. Himmelstern. 1993. The Benefits and Performance of a Full-Scale Alum Recovery Facility. In *Proc. of the Joint Residuals Conference*. Denver, Colo.: AWWA.

Cornwell, D.A., and H. Koppers, eds. 1990. *Slib, Schlamm, Sludge*. Denver, Colo.: AwwaRF, AWWA, and KIWA.

Cornwell, D.A., R.N. Mutter, and C. Vandermeyden. 2000. *Commercial Application and Marketing of Water Plant Residuals*. Final Report. Denver, Colo.: AwwaRF and AWWA.

Cornwell, D.A., and J. Susan. 1979. Characteristics of Acid Treated Alum Sludges. *Jour. AWWA*, 71(10):604.

Cornwell, D.A., C. Vandermeyden, G. Dillow, and M. Wang. 1992. *Landfilling of Water Treatment Plant Coagulant Sludges*. Denver, Colo.: AwwaRF and AWWA.

Crites, R., and G. Tchobanoglous. 1998. *Small and Decentralized Wastewater Management Systems*. New York: McGraw-Hill.

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.

Elliott, H.A., and B.A. Dempsey. 1991. Agronomic Effects of Land Application of Water Treatment Sludges. *Jour. AWWA*, 83(4):126–131.

Elliott, H.A., B.A. Dempsey, D.W. Hamilton, and J.R. DeWolfe. 1990. *Land Application of Water Treatment Sludges: Impacts and Management*. Denver, Colo.: AwwaRF and AWWA.

Everhart, W.H., and R.A. Freeman. 1973. *Effects of Chemical Variations in Aquatic Environments*. II. Toxic Effects of Aqueous Aluminum to Rainbow Trout. EPA-R3-73-0116. Cincinnati, Ohio: USEPA.

FWPCA (Federal Water Pollution Control Administration). 1965. Water Quality Act, Public Law 89-234. 79 Stat. 903.

FWPCA (Federal Water Pollution Control Administration). 1977. Federal Water Pollution Control Act, as amended by the Clean Water Act of 1977, Public Law 92-500, Oct. 18, 1972. 86 Stat. 816, U.S.C. 1215 *et seq.*

Freeman, R.A. 1973. Recovery of Rainbow Trout From Aluminum Poisoning. *Transactions of the American Fisheries Society*, 102:152.

Freeman, R.A., and W.H. Everhart. 1971. Toxicity of Aluminum Hydroxide Complexes in Neutral and Basic Media to Rainbow Trout. *Transactions of the American Fisheries Society*, 100:644.

Fulton, G.P. 1974. Recover Alum to Reduce Waste-Disposal Costs. *Jour. AWWA*, 66(5):312.

Geertsema, W.S., W.R. Knocke, J.T. Novak, and D. Dove. 1994. Long Term Effects of Sludge Application to Land. *Jour. AWWA*, 86(11):68–74.

George, D.B., S.G. Berk, V.D. Adams, E.L. Meyor, R.O. Roberts, C.A. Halloway, R.C. Cott, L.K. Holt, R.S. Ting, and A.W. Welch. 1991. *Alum Sludge in the Aquatic Environment*. Denver, Colo.: AwwaRF and AWWA.

Grubbs, J. 1985. EPA Perspective on Direct Discharge of Alum Sludges. Paper presented at the AWWA Annual Conference on Recent Advances in Sludge Treatment and Disposal, Washington, D.C., June.

Hall, R.J., C.T. Driscoll, G.E. Likens, J.M. Pratt. 1985. Physical Chemical and Biological Consequences of Episodic Aluminum Addition to a Stream Ecosystem. *Limnology and Oceanography*, 30:212.

Hsieh, H.N., and D. Raghu. 1997. *Criteria Development for Water Treatment Plant Residual Monofills*. Denver, Colo.: AwwaRF and AWWA.

Isaac, P.C.G., and I. Vahidi. 1969. The Recovery of Alum Sludge. In *Proceedings, Society for Water Treatment and Examination*, 10:91.

Knocke, W.R., J.T. Novak, J. Lucas, J. Kupar, R. Reneau, and T. Dillaha. 1991. Groundwater and Crop Growth Issues Associated With Water Treatment Residuals. In *Proc. of the AWWA and WPCF Joint Residuals Management Conference.* Washington, D.C.: Water Pollution Control Federation

Lamb, D.S., and G. Bailey. 1981. Acute and Chronic Effects of Alum to Midge Larva. Bulletin. *Environmental Contamination and Toxicology*, 27:59.

Lin, S.D., and D.C. Green. 1990. Two-Year Study of Alum Sludge Application to Corn and Soybean Farmland. Report of Investigation 113. Champaign, Ill.: Illinois State Water Survey.

Lindsey, E.E., and C. Tongkasame. 1975. Recovery and Reuse of Alum From Water Filtration Plant Sludge by Ultrafiltration. *AICHE Symposium Series—Water,* 185.

Maury, J. 1998. Cherokee-Sanford Brick Company, Sanford, N.C. Personal communication.

Migneault, W.H. 1989. Sludge Marketing in Santa Clara County. In *Proc. of the WEF Joint Residuals Management Conference.* Denver, Colo.: AWWA.

Mutter, R.N. 1994. An Assessment of Cropland Application of Alum Sludge. Master's thesis. Virginia Polytechnic Institute and State University, Blacksburg.

Ohio Department of Health. 1969. *Supplement to Report on Waste Sludge and Filter Wastewater Disposal From Water Softening Plants*. Columbus, Ohio: Ohio Environmental Protection Agency.

Owen, P. 1998. Hyder Water Utility, Wales, U.K. Personal communication.

Palin, A.T. 1954. The Treatment and Disposal of Alum Sludge. In *Proceedings, Society for Water Treatment and Examination*, 3:131.

Parkin, G.F., and W.E. Owen. 1986. Fundamentals of Anaerobic Digestion of Wastewater Sludges. *Jour. of Environmental Engineering*, 112(5):867–920.

Parrish, B. 1998. Lawrenceville Brick Company, Lawrenceville, Va. Personal communication.

Pigeon, P.E., K.D. Linstedt, and E.R. Bennett. 1978. Recovery and Reuse of Iron Coagulants in Water Treatment. *Jour. AWWA*, 70(7):397.

Reeves, T. 1980. The Mahoning Valley Sanitary District Plans to Spread Slurry on Farmland. *Farm and Dairy.*

Roberts, J.M., and C.P. Roddy. 1960. Recovery and Reuse of Alum Sludge at Tampa. *Jour. AWWA*, 52(7):857.

Roberts, M.H., and R.D. Diaz. 1985. Assessing the Effects of Alum Sludge Discharges Into Coastal Streams. In *Proc. of the AWWA Annual Conference on Recent Advances in Sludge Treatment and Disposal*. Denver, Colo.: AWWA.

Rolan, A.T. 1976. Evaluation of Water Plant Sludge Disposal Methods. Proc. of the Annual Meeting. *Jour. North Carolina Section AWWA*, 12(1):56.

Rolan, A.T., and J.C. Brown. 1974. Effects of Water Treatment Plant Sludge on the Treatment of Municipal Wastewater. *Jour. North Carolina Section AWWA*, 49(1):29.

Rolan, A.T., and L. Matthews. 1975. A Study of the Effect of Water Treatment Plant Sludges on the Northside Wastewater Plant. Internal Report. Durham, N.C.: City of Durham.

Russell, G.A. 1980. Agricultural Application of Lime Softening Residue. Paper presented at the Missouri AWWA Section Meeting. April, 1980; and Illinois AWWA Section Meeting, May, 1980.

Salotto, B.V., J.B. Farrell, and R.B. Dean. 1973. The Effect of Water Utility Sludge on the Activated Sludge Process. *Jour. AWWA*, 65(6):431.

Schofield, C.L., and J.R. Trojnar. 1980. Aluminum Toxicity to Brook Trout in Acidified Waters. In *Polluted Rain*. Edited by M.W. Miller and P.E. Morrow. New York: Plenum.

Sen Gupta, A.K., P. Li, L. Cumbal, and S. Gokhale. 1997. *Selective Alum Recovery From Water Treatment Residuals*. Denver, Colo.: AwwaRF and AWWA.

Shannon, L.D., C.H. Yu, R. Wieklinski, J. Jarvis, and D. DeKraker. 1997. *Beneficial Reuse of Lime Softening Residuals for Flue Gas Desulfurization*. Denver, Colo.: AwwaRF and AWWA.

Sheen, R.T., and H.B. Lammers. 1944. Recovery of Calcium Carbonate or Lime From Water Softening Sludges. *Jour. AWWA*, 36(11):1, 145.

Sheff, B.B., R.V. Rogers, and D.J. Murrell. 1997. Reuse of Residual Lime for Land Reclamation. In *Proc. of the Joint Water Residuals and Biosolids Conference*. Washington, D.C.: WEF.

Spangler, M.G., and R.L. Handy. 1982. *Soil Engineering.* New York: Harper & Row.

Srinath, E.G. 1978. *Unit Operations for Treatment of Hazardous Wastes*. Park Ridge, N.J.: Noyes Data Corporation.

USDA (U.S. Department of Agriculture). 1992. Census of Agriculture. [Online] Available: <http://www.nass.usda.gov/Census_of_Agriculture/1992/index.asp>. [cited April 6, 2006]

USEPA (U.S. Environmental Protection Agency). 1978. *Sludge Treatment and Disposal—Sludge Disposal*. Vol. 2. EPA-625/4-78-012. Cincinnati, Ohio: USEPA.

USEPA (U.S. Environmental Protection Agency). 1976. *Quality Criteria for Water.* EPA-055-001-01049-4. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1981. 40 CFR Part 403. 1981. General Pretreatment Regulations for Existing and New Sources of Pollution. Jan. 28, 1981. 46 *Fed. Reg*. 9439. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1983a. 40 CFR Part 261. Identification and Listing of Hazardous Waste. Apr. 1, 1983. 48 *Fed. Reg*. 14293. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1983b. *Process Design Manual for Land Application of Municipal Sludge*. EPA-625/1-83-016. Municipal Environmental Research Lab. Cincinnati, Ohio: USEPA.

USEPA (U.S. Environmental Protection Agency). 1991. 40 CFR Parts 257 and 258. Solid Waste Disposal Facility Criteria, Final Rule. 56 *Fed. Reg.* 50978. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1998. *Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, SW-846*. 3rd ed., Update 2B. Washington, D.C.: USEPA.

USGS (U.S. Geological Survey). 1997. Cement-Statistical Compendium [Online]. Available: <http://minerals.er.usgs.gov/minerals/pubs/commodity/cement/stat/>. [cited August 13, 1998]

Webster, J.A. 1966. Operation and Experimental Experience at Daer Water Treatment Works With Special Reference to the Use of Activated Silica and the Recovery of Alum From Sludge. *Jour. of the Institute of Water Engineering*, 20(2):167.

WEF and ASCE (Water Environment Federation and American Society of Civil Engineers). 1998. *Design of Municipal Wastewater Treatment Plants*. Alexandria, Va.: WEF.

Wertz, C.F. 1960. Miami Lime Recovery Plant. *Jour. AWWA*, 52(3):326.

Westerhoff, G.P. 1973. Alum Recycling: An Idea Whose Time Has Come? *Water and Wastes Engineering*, 10(12):28.

Westerhoff, G.P., and M.P. Daly. 1974. Water Treatment Plant Wastes Disposal—Parts 1, 2, 3. *Jour. AWWA*, 66:5–7.

Wooley, I. 1996. Turf Farm Application of WTP Residuals. In *Proc. of the AWWA Annual Conference.* Denver, Colo.: AWWA.

CHAPTER 6
MEMBRANE RESIDUALS

INTRODUCTION

As membrane systems are increasingly used for water utility applications, the management of their residuals has become a growing challenge. Membrane technology uses a driving force (e.g., electrical, pressure–vacuum, etc.) to separate contaminants from the water. Pressure-driven membranes include MF, UF, NF, and RO. Electrically-driven membranes include ED and its variant EDR. Whereas MF and UF membranes are designed for particle removal and use low pressure, NF, RO, and ED/EDR are designed for desalination (and softening). In the next sections, membrane residuals will fall into two categories: low-pressure membranes (MF and UF) and desalination membranes (NF, RO, and ED/EDR).

LOW-PRESSURE MEMBRANES

Process Description

Low-pressure membranes are principally applied for particle and pathogen removal. From a practical treatment perspective, UF differs from MF in its ability for virus removal due to smaller pore size (usually between 0.01 to 0.04 µm). The MF and UF configuration generally consists of modules of hollow fibers that can either be housed in vessels and operated under pressure, or assembled into racks that are submerged in a tank and operated under vacuum (AWWA 2003).

Low-pressure membranes usually operate in dead-end mode (direct filtration), in which all the feedwater is converted into permeate. However, in cases of feedwaters with high suspended solids, cross-flow filtration is employed, in which a portion of the feedwater flows across the membrane surface so that a portion is converted to permeate and the remainder becomes the concentrate that is either wasted or recycled and blended with feedwater.

Over time, membrane systems may experience a loss in productivity from the accumulation of foulants on the membrane surface and within pores. These foulants can be inorganic (clay or silt colloids, precipitated metals, etc.), organic (NOM, coagulant aid polymers, etc.), or biological (biofilms, etc.). To control fouling, backwashing and chemical cleaning (or clean-in-place, CIP) are used to restore productivity.

Backwashing typically occurs every hour or so and is accomplished from the permeate side to the feed side, in the opposite direction of filtration, and using air, air and permeate water, or permeate water alone. Chemically enhanced backwash (CEBW) involves the addition of chemicals to increase the backwashing effectiveness by oxidizing or dissolving some of the solids in the fouling layer and by inactivating biological growth.

However, with time, the extent of irreversible fouling (the fouling portion that is not backwashable) requires that the system be removed from service for a half day or full day to perform chemical cleaning (or CIP). Tank-batched solutions of appropriate oxidant, caustic soda, acid, and detergent are recirculated individually or in combination to dissolve membrane foulants. Sometimes these solutions are heated to help loosen the foulants. Citric acid, caustic soda, and chlorine

Table 6.1
Typical characteristics of low-pressure membrane spent backwash

Parameter	Characteristics
Frequency of application	Every 30 to 60 minutes
Volume of waste produced	2% to 15% of plant feed flow rate for recoveries of 85% to 98%
Composition of spent backwash	• Algae, precipitated solids, possible chemical residues if using CEBW • For recoveries of 85% to 98%, feedwater TSS will be concentrated 7 to 50 times. • pH may be <6 or >9. • Chlorine (Cl_2) residuals may be up to 1,000 mg/L. • If coagulant is used, TOC could be five times the feed concentration; otherwise TOC can be up to twice that of the feed concentration.

Source: Adapted from AWWA 2003.

are the most common agents used for cleaning low-pressure membranes. CIP is usually performed on a monthly interval or more often.

When membranes are used in combination with other treatment processes (integrated membrane systems), the nature of the residuals will differ compared to a stand-alone membrane system. The backwash waste will not only contain the contaminants naturally occurring in the source water but also those added during upstream treatment (e.g., coagulation). Therefore, residuals management alternatives (i.e., disposal, reuse, etc.) may differ depending on whether the residuals are from an integrated or stand-alone membrane system (AWWA 2003).

Generally, MF/UF systems produce residuals on an intermittent basis. However, in large membrane plants (>10 mgd [>38 ML/d]), the backwash sequence operation is rotational such that intermittent backwash flow rates from individual trains essentially become continuous on a plant basis.

Backwashing Quantities and Characteristics

Specific characteristics of MF and UF backwash and concentrate residuals depend on the quality of the water being treated and the recovery of the membrane. Recovery is the ratio of water produced (i.e., feed flow minus water used for backwash) to feed flow and can range from 85 to 98 percent, thereby concentrating solids 7 to 50 times, respectively. Backwash flow rates typically represent >95 percent of the total volume of residuals produced (the remaining portion comes from chemical cleaning procedures). Table 6.1 presents typical spent backwash characteristics. If coagulants or other pretreatment chemicals are applied, the characteristics of the MF/UF residuals become more similar to the residuals from a conventional WTP, while those from spent CEBW will more resemble those from CIP (AWWA 2003).

Table 6.2
Typical characteristics of MF/UF spent chemical cleaning solutions

Parameter	Characteristics
Frequency of application	Every month
Volume of waste produced	Monthly CIP wastes normally <0.05% of plant feed flow rate
Chemicals commonly used	• Sodium hypochlorite (NaOCl) (500–1,000 mg/L as Cl_2) • Citric of hydrochloric acid (pH 1 to 2) • Caustic soda (pH 12 to 13)
Composition of spent CIP	• pH from 2 to 14 • TSS up to 500 mg/L (neutralization may precipitate additional solids) • Low concentrations of surfactants • Cl_2 residuals may be up to 1,000 mg/L • TOC 10 to 30 times the feedwater concentration • BOD_5 up to 5,000 to 10,000 mg/L if citric acid is used

Source: Adapted from AWWA 2003.

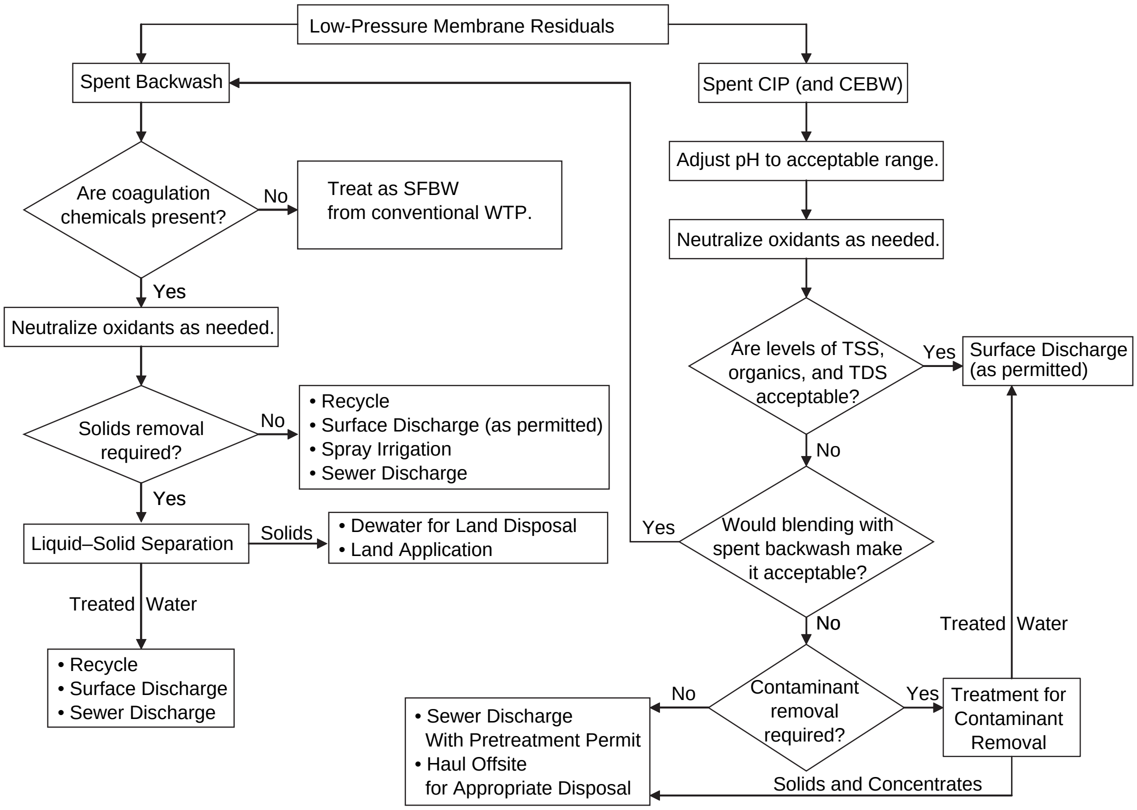
Chemical Cleaning Quantities and Characteristics

Cleaning solution residuals reflect the chemicals used in the cleaning process, so the resulting chemical cleaning waste includes some remaining active chemical ingredient, as well as dissolved organic materials, suspended solids, and salts from chemical reactions between the chemicals and foulants (AWWA 2003). Table 6.2 describes typical CIP waste characteristics.

Chlorine residuals in CEBW and CIP wastes may range from 1 to 1,000 mg/L as Cl_2, and pH may be acidic (pH <6) or basic (pH >9) depending on the chemicals used. When surfactants are employed, they may cause foaming when the spent cleaning solution is discharged. Mixing the acid and caustic waste solutions will minimize the need for neutralization chemicals, but it may also precipitate additional solids such as calcium carbonate or some iron compounds, which will increase the suspended solids concentration of the waste solution. If citric acid is used in the CIP, the BOD of the spent cleaning solution is likely to be high as well.

Cleaning solution volumes will be 2 to 3 times the volume normally present in the membrane modules and piping (one volume for the actual cleaning solution, and one or two additional volumes to rinse the solution), or for immersed systems, equal to the tank volume (AWWA 2003). Rinse waters have low levels of contamination and may be returned and blended with the feedwater, combined with backwash wastes for recycling or disposal, or used to make up the next batch of cleaning solution. Chemical cleaning is performed far less frequently than backwashing and is thereby contributing less than 5 percent in the generated residuals volume.

Membrane modules may be shipped to the plant preserved in specialty solutions, such as glycerin, which are typically discharged to the sewer. In the absence of a sewer connection, these wastes must be collected and hauled to an appropriate disposal site (AWWA 2003).



Source: Adapted from AWWA 2003.

Figure 6.1 Decision tree for MF/UF residuals management

Residuals Management

Membrane residual treatment and disposal alternatives depend on site-specific factors such as local climate, land availability, size of the MF/UF installation, capabilities of the operating staff, and feasibility and local requirements for different disposal options. Smaller installations will tend to use simple methods capable of handling both types of residuals (backwash and cleaning wastes), such as discharge to the sanitary sewer, settling lagoons or percolation/evaporation ponds, and EQ basins to store, mix, and neutralize chemical solutions (AWWA 2003). It is believed that, in the future, large installations will increasingly include recycling of backwash and reuse of chemical cleaning solutions, which will significantly reduce wastewater volumes and the disposal cost (MacPhee, Cornwell, and Brown 2002). A decision tree for considering low-pressure membrane residuals disposal alternatives is provided in Figure 6.1. According to federal regulations, low-pressure membrane backwash residuals are considered industrial wastes, similar to other water treatment wastes. The comparative economics for each disposal alternative will be influenced by the cost and availability of land, the local value of water, the availability of a sewer system to accept the process wastewaters, and the demand (amount and seasonal variation) for irrigation water (AWWA 2003).

Surface Water Discharge

The NPDES regulates discharge into receiving surface waters according to water quality standards that protect the designated usage of the receiving waters. Surface water discharge is the most common form of backwash waste disposal regardless of plant size (Kenna and Zander 2000). For most installations, a simple settling basin or lagoon provides enough treatment to allow subsequent surface water discharge.

Underground Injection

The Underground Injection Control (UIC) Program administers groundwater discharge permits (e.g., deep-well injection) and provides safeguards to maintain current and future underground sources of drinking water. An NPDES permit may also be necessary if a secondary disposal method is required or if the receiving groundwater is hydraulically linked to a surface water.

Sewer Discharge

Indirect discharge to a POTW may be controlled under the Industrial Pretreatment Program (IPP) if the POTW practices surface water discharge and has its own NPDES permit (AWWA 2003). Under the IPP, a POTW is required to enact local sewer ordinances and to enforce standards laid out in the IPP to protect the POTW's NPDES permit from any violation caused by indirect discharge. Other federal and local regulations may apply depending on the method of waste disposal used by the receiving WWTP. Most plants do not provide any treatment of the spent backwash prior to discharge to a sewer, but discharges may need to be equalized to meet available sewer capacity (AWWA 2003).

Land Disposal

Percolation ponds, leaching fields, and other land disposal methods that carry a risk of liquid discharge reaching any groundwaters are affected by the same regulations governing underground discharge (i.e., NPDES and UIC programs). When high concentrations of solids are present, land application of membrane residuals is regulated by the Solid Waste Disposal Act. If disposal is by means of a landfill, additional RCRA requirements will apply.

Solids residuals must also undergo a TCLP test. Residuals that fail the TCLP test are deemed hazardous wastes and are subject to additional regulations, but in nearly all cases, low-pressure membrane process residuals do not fall into this category. Spray irrigation is often used for land application because it is a low-cost alternative, which provides benefits for the water utility and the organization needing irrigation water. To be successful, spray irrigation requires year-round demand, low concern for aesthetics, and little effect on groundwater at the application site. Spray irrigation may not be feasible for large MF or UF plants because of the large residuals volumes.

Evaporation Ponds

Although there is no federal regulation for evaporation ponds, NPDES or UIC program permits may be required if there is a chance of intermittent discharge to surface or groundwater. For this type of discharge, optimal conditions include low flows and an arid climate. Evaporation ponds should provide adequate space for accumulating several years' worth of solids to allow infrequent cleaning of the pond (AWWA 2003).

Backwash Recycle

The Filter Backwash Recycling Rule (FBRR), which regulates the recycling of backwash and other recycled flows in conventional treatment plants, does not address membrane residual recycling unless membrane wastes are commingled with other conventional process wastes prior to recycling (AWWA 2003). At large installations, recycling of backwash wastes is generally accompanied by some type of settling process in order to concentrate solids waste for disposal (Kenna and Zander 2000). Additional treatment may be required at installations where backwash residuals contain a higher concentration of contaminants such as *Cryptosporidium* (MacPhee, Cornwell, and Brown 2002). Recycling clarified backwash water along with disposing of the concentrated backwash solids and neutralized CEBW wastes may reduce the volume of disposal wastes by more than 90 percent and would make hauling the concentrated residuals offsite more economically feasible. The following recommendations were made for treatment of spent MF/UF backwash prior to recycle (LeGouellec, Cornwell, and MacPhee 2004):

- MF backwash streams can vary widely in solids content (<10 to >125 ntu).
- Addition of a metal salt coagulant is necessary if the raw water is not coagulated ahead of the membrane in the main treatment train.
- Ferric chloride generally outperformed aluminum-based coagulants; pH adjustment was not needed.
- Sedimentation and DAF are both effective means of clarification for spent backwash.
- A sedimentation hydraulic overflow rate of 0.25 to 0.50 gpm/ft^2 (0.6 to 1.2 m/hr) is appropriate for settling if the right dose of a primary coagulant is used. Addition of a polymer with the metal salt coagulant may enhance performance.
- Bench-scale testing is an excellent tool for screening coagulant, polymer, pH adjustment, and overflow rate requirements for treatment of MF/UF backwash streams intended for recycle.
- In general, a sedimentation treatment system for MF/UF backwash would include EQ to provide a relatively constant backwash quality and quantity for sedimentation and to allow chemical dosing to be set based on average conditions.
- Under suitable treatment conditions, an average of at least 1-log *Cryptosporidium* removal can be achieved through sedimentation of spent backwash.
- Removal levels for *Bacillus subtilis* and particle counts are similar to those for *Cryptosporidium*, suggesting that these parameters could be used as surrogates for gauging treatment performance for pathogen removal.
- To limit potential recycle impacts, backwash turbidity levels and particle counts after treatment should be close to those of the raw (influent) water.

CIP Disposal

CIP residuals disposal methods for low-pressure membrane plants depend mainly on the frequency of cleaning and the type and strength of chemicals used (AWWA 2003). For example, large plants that perform frequent chemical cleanings may not be able to use surface water discharge for the spent CIP, unless the CIP wastes can be blended with spent backwash for chemical neutralization. Plants usually redirect their CIP wastes to the sewer after pH adjustment and/or dechlorination, if needed, or blend the CIP residuals with the backwash residuals prior to joint disposal (AWWA 2003).

DESALINATION MEMBRANES

ED and RO were first used for potable water treatment in the 1960s, and NF followed in the mid-1980s. In the early 2000s, there were more than 230 desalting facilities (187 RO, 29 NF and 18 ED) with a capacity >25,000 gpd (95,000 L/d) operating in the United States (Mickley, N.d.). The use of desalination membrane processes will continue to increase as utilities and other users recognize that such technologies can extend water resources by treating seawater, brackish water, and reclaimed water for drinking water use. With the continuing installation of desalting plants, residuals management has become a growing issue especially since the principal residual from these systems (i.e., concentrate) typically contains high levels of TDS, including heavy metals and radionuclides, which makes its disposal more challenging than for residuals from low-pressure (MF and UF) membrane systems.

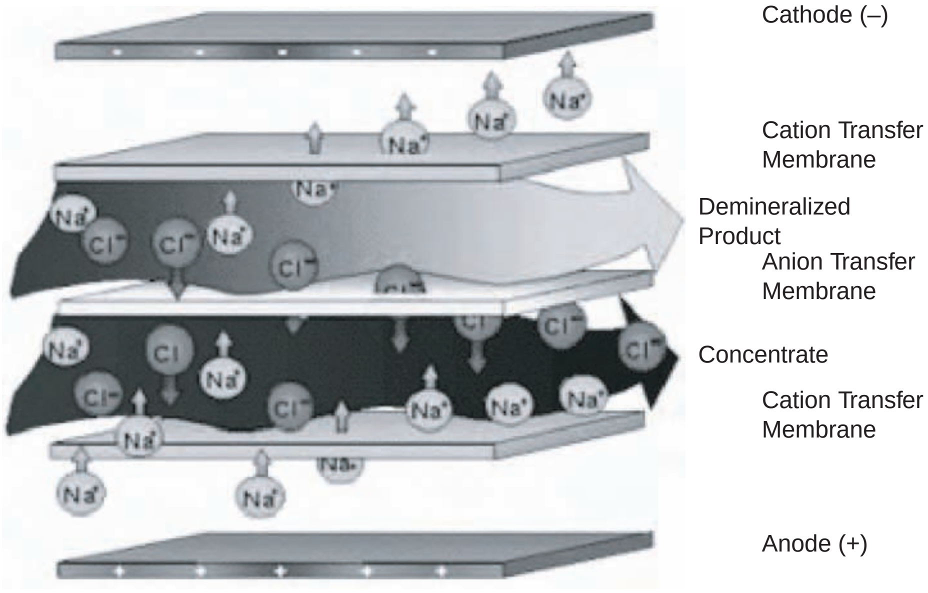
Process Description

The separation of dissolved substances from water can be made under a pressure differential (RO and NF) or under an electrical potential (ED/EDR). Operation of NF and RO systems, as well as ED/EDR, on feeds containing >1 mg/L suspended solids, 1 ntu turbidity, or a silt density index of 5.0 will cause significant performance degradation and a high frequency of chemical cleaning.

Electrically Driven Processes

ED and EDR use a difference in electrical potential to induce dissolved ions to pass through IX membranes. ED/EDR membranes are essentially impervious to water flow. In contrast to pressure-driven processes, purification is achieved by the movement of contaminants, not water, across the membrane. The ED process, shown schematically in Figure 6.2, places alternating pairs of cation (+) and anion (–) transfer membranes between positively and negatively charged electrodes. When a voltage is applied across the electrodes, a DC causes cations to move toward the cathode through the cation membrane but are stopped by the anion membrane. Conversely, anions move in the direction of the anode and are transported through the anion membrane and are stopped by the cation membrane. Water is fed to all channels, and the flow from channels from which contaminants are transferred becomes the demineralized product stream (diluent). The flow from channels into which salts are transferred becomes the concentrate stream.

With ED, the direction of electrical current flow stays the same, whereas with EDR, the current flow changes several times an hour by reversing the electrical polarity applied to the electrodes to flush scale-forming ions from the membrane surfaces. This periodic flushing helps to



Source: Adapted from Ionics, Inc., 2003a.

Figure 6.2 Electrodialysis schematic

control the formation and buildup of scale and may allow the EDR process to operate at higher recovery than ED, and/or reduce or eliminate the need for addition of scale-inhibiting chemicals.

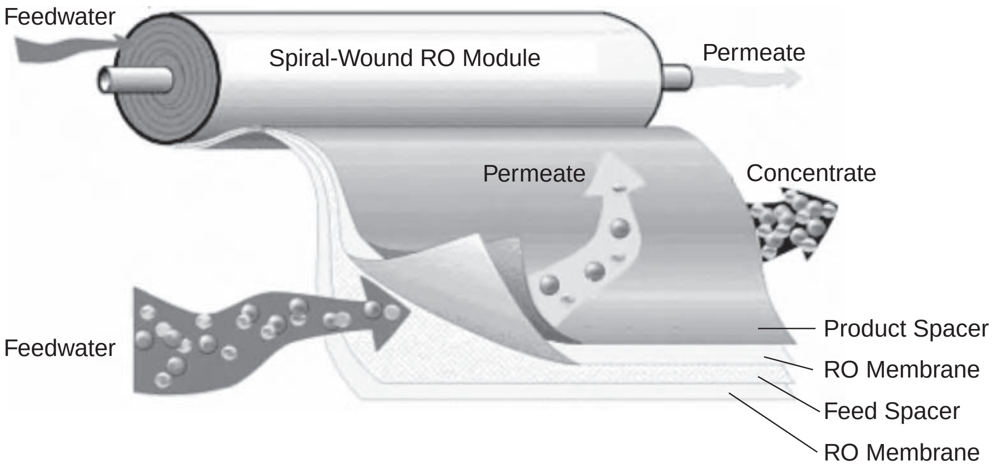
The ED/EDR pump pushes the feedwater through a series of membrane "stacks" that contain several hundred anion/cation membrane pairs and associated feed channels. Feed pressures for ED/EDR are typically between 50 and 70 psi (350 and 480 kPa) and are governed by the head loss associated with water flowing through the membrane stacks. Another pump recirculates a portion of the concentrate within the concentrate channels of each stack to maintain appropriate fluid velocity. Power is supplied to the electrodes via a rectifier that converts alternating current to DC. For EDR, a series of electrically actuated valves allow switching the product and concentrate streams within the stacks upon electrode polarity reversal.

Since product water does not result from flow passing through a membrane, the ED/EDR does not provide a barrier to non- or weakly charged solutes such as pathogens, particulates, silica, and dissolved organics. ED/EDR is only designed for demineralization of brackish water supplies.

Pressure-Driven Processes

Feed pressures range widely, reflecting both the permeability of different types of desalting membranes and the salinity (and associated osmotic pressure) in the feedwater (e.g., seawater osmotic pressure is about 400 psi [2,760 kPa]). NF typically operates at lower feed pressures than RO because of greater water permeability (i.e., "looser" than RO) and lower osmotic pressures associated with the applicable type of waters.

Basic components of an NF or RO system include a high-pressure pump to provide the driving force, pressure vessels with multiple modules, and backpressure control to achieve the desired permeate water flow and recoveries. NF and RO use flat-sheet membrane material



Source: Adapted from Ionics, Inc., 2003b.

Figure 6.3 Spiral-wound RO module

arranged in spiral-wound modules that are 4 in. or 8 in. (10 or 20 cm) in diameter and 40 or 60 in. (100 or 150 cm) in length (Figure 6.3). Typically six to eight membrane modules are housed in series in a pressure vessel. Multiple pressure vessels are then staged, both in parallel and in series, to achieve the desired permeate production capacity and maximize the conversion (recovery) of feedwater to permeate. In this arrangement, each module converts about 15 percent of the feedwater to permeate. Typical overall recoveries for NF plants are 75 to 85 percent for brackish water plants depending on the types and concentrations of potential scaling species (sparingly soluble salts and silica) in the feedwater. Recoveries for seawater plants range from 40 to 60 percent because of the material resistance limit imposed by the high osmotic pressure.

Fouling

Over time, membrane systems will experience a loss in productivity from foulants accumulating on the membrane surface or within the feed channel between the layers of membrane material. Although these foulants can be identified as belonging to one of the following categories, fouling usually consists of a combination of factors, with the relative contribution of each foulant category a function of feedwater characteristics, membrane type, and system operating conditions (AWWA 2004):

- Inorganic: clays and silts, in colloidal or suspended form; metal precipitates (e.g., aluminum, iron, and manganese oxides); precipitates from sparingly soluble salts (e.g., barium sulfate, calcium carbonate, calcium sulfate, etc.); and silica
- Organic: NOM (humic and fulvic acids, etc.)
- Biological: microorganisms that accumulate or actively colonize the membrane surface, as well as the exopolymeric substances that such organisms produce for the purpose of attachment or protection (i.e., biofilm)

Residuals Quantities and Characteristics

The final characteristics of the concentrate from the desalination membrane systems will be dependent upon what constituents are removed or added by the upstream treatment process. The residuals of NF, RO, and ED systems result from the separation process itself and the CIP process. Thus, the residuals produced by desalination processes are two-fold (AWWA 2004):

- Concentrate that contains dissolved and particulate contaminants at levels that are a direct function of their feedwater concentration and recovery of the system. The volume of continuously produced concentrate is much greater than that of intermittently generated cleaning waste.
- Spent cleaning solutions, generated on an intermittent basis (typically semiannually), that contain high concentrations of the cleaning chemicals and removed contaminants.

Because the characteristics of the concentrate from NF, brackish water RO, and ED/EDR systems can be considered similar, residuals management will be about the same. The exception is that ED/EDR systems produce a specialized waste stream of limited flow called “electrode waste” that contains significant levels of hydrogen and chlorine gases. These gases are typically stripped from the electrode waste stream using a degasifier. Although free chlorine may be present in the poststripper waste stream, it will be diluted when the stripped electrode waste is mixed with the concentrate flow (AWWA 2004).

The typical addition of acid and/or antiscalants as part of the pretreatment in desalination membrane systems helps to control scaling from sparingly soluble salts. Chemical cleaning is achieved by using appropriate acidic, caustic, and selected detergent solutions, either individually or in combination, to loosen and dissolve membrane foulants. These solutions are tank-batched and circulated directly through the membrane train. Heating the chemical solution can be used to help loosen accumulated materials. Chemical cleaning is typically performed every 6 to 12 months depending on feedwater quality and operating characteristics. Some systems with low-fouling feedwaters can operate indefinitely without the need for chemical cleaning, particularly if designed at conservative flux rates (AWWA 2004).

Residuals from the chemical cleaning of ED/EDR systems are different from those from NF and RO systems:

- NF and RO systems are typically cleaned with acid (mineral or citric) and alkaline (typically containing detergents/surfactants and sometimes chelants) solutions.
- ED/EDR systems are typically cleaned with concentrated sodium chloride solutions and sometimes with chlorine to remove biological foulants (NF and RO systems do not use chlorine because it damages the membranes in these systems).

Concentrate

The concentration of contaminants in the concentrate is dependent upon the rejection characteristics of the membrane and is directly proportional to recovery. In contrast to the spent backwash from low-pressure membrane systems, the concentrate from desalination membrane systems contains low particulate concentrations, typically <10 mg/L total suspended solids. This is because feedwater turbidity is required to be less than 1.0 ntu to avoid plugging the flow channels. For ED/EDR, ions will be concentrated in the waste stream to the same levels as described for NF and RO, but the concentrate will also contain some free chlorine from the generation of chlorine gas at the electrode.

Scale-inhibiting agents added to the membrane feedwater to control precipitation will be concentrated in the desalination membrane process concentrate to levels typically <30 mg/L. For salinity, the brine concentration can be estimated based on the feedwater quality and a concentration factor that is related to the water recovery, as shown in Equation 6.1. These types of equations assume a membrane process with 100 percent salt rejection and therefore provide a conservative (high) estimate of residuals concentrations:

$$C_b = \frac{C_f}{1 - Y} \tag{6.1}$$

where C_b = brine concentration
C_f = feed concentration
Y = water recovery

For example, recoveries of 50 and 90 percent would correspond to concentration factors of 2 and 10, respectively.

The quantity of concentrate is a function of the amount of feedwater that is purified (or converted to permeate), as defined by the product water recovery. NF salt rejection is typically less than for RO; therefore, for a given feed and recovery, NF concentrate will be less saline than RO concentrate. Additionally, NF rejects monovalent ions (e.g., sodium and chloride) to a lesser degree than multivalent ions (e.g., calcium and sulfate), thereby enriching the concentrate in multivalent ions compared to the feedwater.

Chemical Cleaning (CIP) Waste

Each membrane supplier has developed its own specific solutions or formulations based on laboratory and field experience with their membrane product, and many companies catering to the membrane industry have developed proprietary cleaners in an attempt to optimize cleaning efficiency for specific fouling situations (AWWA 2004). Table 6.3 presents a listing of standard cleaning solutions to remove various categories of foulants.

The volume of cleaning solution used and the amount of generated waste are function of

- Source water characteristics (type and level of foulants present)
- Membrane process and module/system characteristics
- Size/capacity of membrane train
- System operating conditions (flux and recovery)
- Cleaning system design (piping diameter and lengths)
- Frequency of cleaning

Chemical cleaning typically requires several steps. Usually, a low-pH solution is used first, followed by a high-pH solution. The trains are also cleaned in steps. The typical approach is to

Table 6.3
Typical chemical cleaning solutions for desalting membranes

Foulant type	Cleaning solution
Inorganic salts/precipitates	• 0.2% HCl • 0.5% phosphoric acid (H_3PO_4) • 2% citric acid
Metal oxides	• 2% citric acid • 1% sodium sulfate (Na_2SO_4)
Inorganic colloids (silt)	• 0.1% sodium hydroxide (NaOH) and 0.05% sodium dodecyl benzene sulfate at pH 12
Silica (and metal silicates)	• 0.1% NaOH, 0.05% sodium dodecyl benzene sulfate at pH 12 • Ammonium bifluoride
Biofilms and organics	• Hypochlorite, hydrogen peroxide, 0.1% NaOH, 0.05% sodium dodecyl benzene sulfate at pH 12 • 1% sodium tripolyphosphate, 1% trisodium phosphate, 1% sodium ethylenediamine tetraacetic acid

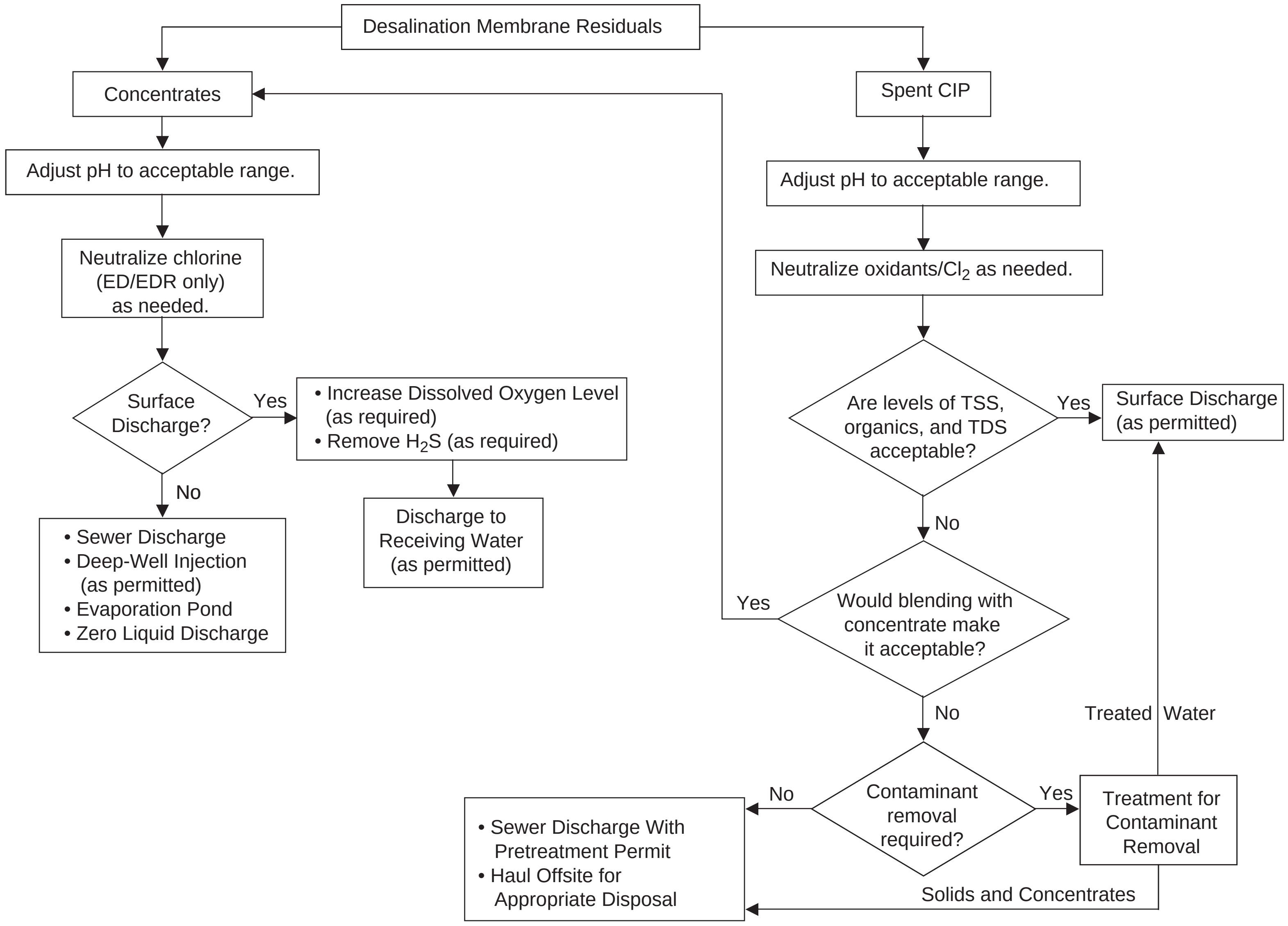
Source: Adapted from AWWA 2004.

first clean the modules in one half of the vessels in the first stage, followed by the other half of the first stage, and finally all modules in the second stage. Spent cleaning solutions will contain the foulants and the chemicals used to remove them (e.g., detergents, surfactants, acid, caustic soda). Typically, the volume of the spent cleaning solution is a very small percentage of the flow treated (significantly <0.1 percent). The total volume of spent cleaning solution also includes the rinses between each cleaning step. The spent cleaning solution may need to be treated before disposal, most often by simple pH adjustment (AWWA 2004).

Residuals Management

Decisions regarding residuals management should include capital and operating costs but also the assessment of the long-term viability of any selected approach. Regulatory changes, in particular, have the potential to adversely impact operation of a desalting plant. Disposal of desalination membrane residuals is generally regulated by the same guidelines governing industrial residuals, which results in a longer and costlier permitting process than if the nature of membrane concentrate was taken into consideration at the beginning of the permitting process. Distinct from many industrial wastes, membrane concentrate is not characterized by process-added chemicals but by the nature of the raw water fed to the process (AWWA 2004).

Unlike residuals from MF/UF membranes, management and disposal of desalination membrane concentrate can be particularly challenging because volume and high concentration of dissolved solids can limit the number of options that can be permitted or cost-effective, particularly in geographical regions where surface water discharge is unavailable. A decision tree for considering residuals disposal alternatives for desalination membrane processes is presented in Figure 6.4.



Source: Adapted from AWWA 2003.

Figure 6.4 Decision tree for desalination membrane residuals management

Surface water discharge is the most common method of NF, RO, and EDR concentrate management, followed by sanitary sewer discharge and deep-well injection (Mickley 2001). The two methods typically used for spent chemical cleaning solutions are sewer disposal and blending with concentrate. Prior to blending, it may be necessary to neutralize the acidity or alkalinity of the cleaning solution to prevent unwanted reactions or to ensure that the blended residual is compatible with regulations governing the concentrate discharge. For certain ED/EDR cleaning wastes, residual chlorine may need to be neutralized with a reducing agent.

For small facilities, spent cleaning solution treatment sometimes is accomplished in the cleaning tank itself at the end of each cleaning step. However, for larger facilities, treatment is usually accomplished in separate facilities because of the relatively low cost of these facilities and the significant benefit of reducing membrane system downtime for the overall cleaning process. Commonly, one or two spent cleaning tanks are used with pumps that recirculate the spent cleaning solution from and back to the tank(s) or transfer the treated solution to the disposal point. For pH adjustment, either acid (e.g., sulfuric acid) or base (e.g., sodium hydroxide) is added to the recirculating fluid until the desired pH neutralization point is reached. This can be monitored and controlled manually or automatically (AWWA 2004).

Surface Water Discharge

The discharge of NF, RO, and ED residuals to a surface water body (i.e., lake, river, lagoon, ocean, etc.) is the most common management practice, primarily because this method has the lowest cost (unless pipeline conveyance distances are long) and the residuals volume improves stream flow. However, the residuals must be compatible with the environment of the receiving water body. Discharge to surface waters requires an NPDES permit, which includes an assessment of salinity impacts, as well as those of specific constituents, on the receiving stream. Discharge of a concentrate that would result in a >10 percent increase of the receiving stream salinity is rarely allowed. Some facilities address this by diluting the concentrate (e.g., with WWTP effluent, cooling water, etc.).

In some states, whole effluent toxicity tests (bioassays) are part of the NPDES permit. The concentrate produced by some groundwaters treated by RO have been found to fail toxicity tests on sensitive test organisms because of major ion toxicity, which is due to excessively high (or low) concentrations of common ions (Mickley 2000). Most cases in Florida have been associated with high calcium levels. Some of these situations have also been complicated by toxicity due to high levels of fluoride. Because major ion toxicity is different from that due to contaminants such as heavy metals or pesticides, the state of Florida has made allowances in cases where only major ion toxicity is present in membrane concentrates.

Because the principal effect of membrane processes is an increase in the concentration of naturally occurring constituents which typically have no toxicity, desalting residuals are well suited for many surface water discharge applications. However, for membrane facilities treating brackish groundwater, the resulting salinity of the residuals may not be compatible with fresh surface waters, and, therefore, such plants are limited to areas with available brackish receiving streams (e.g., estuary, tidal river, etc.).

Concentrates from desalination of most groundwaters have low levels of dissolved oxygen, which must be increased prior to discharge to avoid negative impacts on receiving stream biota. Occasionally, the groundwater may also contain hydrogen sulfide, which must also be suitably removed from the concentrate prior to discharge. As mentioned earlier, ED/EDR concentrates can contain free chlorine. The free chlorine must be neutralized prior to surface water discharge, which requires that the reducing agent be compatible with the receiving stream (AWWA 2004).

Sewer Discharge

Disposal to the sewer (indirect discharge) requires permission from the receiving POTW, which may impose pretreatment standards because the discharge permit of the WWTP could be affected. Additionally, the capacity of the sewer line and WWTP must be addressed. If the concentrate has high salinity and its flow contribution is significant, impacts of salinity on the biological efficiency of the wastewater plant should also be considered. These capacity and quality criteria may limit the amount of desalting residuals discharged to the sewer. Sanitary sewer discharge of small volumes usually represents a low-cost disposal method with limited permitting requirements.

Land Application

Disposal of concentrate by land application (percolation pond, rapid infiltration basin, irrigation, etc.) must be compatible with state regulations protecting groundwater and vegetation (e.g., in the case of irrigation). Factors associated with land application include the availability and cost of land, percolation rates, irrigation needs, water quality tolerance of target vegetation to salinity, and the ability to meet groundwater quality standards. An assessment of the compatibility with target vegetation should be conducted, including assessment of the sodium adsorption ratio, trace metals uptake, and other vegetative and percolation factors (AWWA 2004).

Regulations governing groundwater quality and protection of drinking water aquifers should be investigated as early as possible to confirm the acceptability of this alternative. Usually, dilution of the concentrate is required to meet groundwater standards. Where salinity levels are excessive, special salt-tolerant species (halophytes) could be considered for irrigation. In general, land applications are used only for smaller-volume concentrates as this disposal option is frequently limited by availability of land, dilution water, and/or climate in locations where land application is not possible year-round (AWWA 2004).

Deep-Well Injection

Deep-well injection and other subsurface injection alternatives, such as boreholes, are regulated by USEPA’s UIC Program, which is typically administered by the state. Regulatory considerations include the transmissivity and salinity of the receiving aquifer and the presence of a structurally isolating and confining layer between the receiving aquifer and any overlying underground source of drinking water, which is defined as any water-bearing formation containing <10,000 mg/L TDS (AWWA 2004).

Deep wells are not feasible in areas subject to earthquakes or where faults are present that can provide a direct hydraulic connection between the receiving aquifer and an overlying potable aquifer. One or more small-bore monitoring wells in the local area are also typically required to confirm that vertical movement of fluid has not occurred. The cost for deep-well injection is higher than surface water disposal, sewer disposal, and land application in cases where these alternative methods do not require long transmission pipelines. Disposal to deep wells is usually restricted to larger-volume concentrates because economies of scale make this disposal option more affordable. A backup means of disposal must be available for use during periodic maintenance testing of the well. Geologic characteristics make this option inappropriate in many areas of the United States, and nearly all of the plants using subsurface injection are located in Florida (Mickley 2001).

Evaporation Pond

Solar evaporation is a viable alternative in relatively warm, dry climates with high evaporation rates, flat terrain, and low land costs. Construction of evaporation ponds is subject to state requirements for pond construction, which typically requires an impervious lining and monitoring wells. With modest economy of scale, evaporation ponds are usually used only for small-volume concentrates. Although evaporation ponds are typically designed to accommodate concentrate for the projected life of the desalination facility, precipitation of salts is expected and must be incorporated into the depth requirements of the pond (AWWA 2004). In addition, the ultimate fate of the concentrated salts and the future regulatory implications should be considered for any evaporation pond project.

Other Alternatives

Other concentrate management and disposal alternatives such as blending with WWTP effluent or with power plant cooling water are subsets of the alternatives previously presented. Blending provides a dilution effect that can support the implementation of a desalting facility.

Zero-liquid discharge systems such as thermal evaporators, brine concentrators, vapor compressors, crystallizers, and spray dryers are available to reduce residuals to a solid product for landfill disposal. However, the cost for these thermal systems is typically much higher than the cost for the desalination membrane facility, both from a capital and operating (energy) perspective, making this disposal option unfeasible except for very small concentrate flows (AWWA 2004). Use of high-recovery RO systems placed in front of the thermal evaporators can reduce cost for waters of limited hardness where chemical and solids disposal costs are not excessive. The selective and sequential removal of salts followed by their use (sale) may offer promise to reduce zero-liquid discharge costs (Mickley, N.d.).

Research is ongoing to evaluate the technical and cost feasibility of using desalination membrane concentrates as feed to sodium hypochlorite generation and for solar energy ponds that recover energy by heat generation (AWWA 2004). These and other research activities may ultimately provide additional alternatives for management of desalination residuals.

REFERENCES

AWWA (American Water Works Association). 2003. Residuals Management for Low-Pressure Membranes. Committee Report. *Jour. AWWA*, 95(6):68–82.

AWWA. 2004. Current Perspectives on Residuals Management for Desalting Membranes. Committee Report. *Jour. AWWA*, 96(12):73.

Ionics, Inc. 2003a. Electrodialysis schematic [Online]. Available: <http://www.ionics.com/toolbox/edr.htm>. [cited July 7, 2004]

Ionics, Inc. 2003b. Spiral-wound reverse osmosis module [Online]. Available: <http://www.ionics.com/toolbox/ro.htm>. [cited July 7, 2004]

Kenna, E., and A. Zander. 2000. *Current Management of Membrane Plant Concentrate*. Denver, Colo.: AwwaRF and AWWA.

LeGouellec, Y., D. Cornwell, and M. MacPhee. 2004. Treating Microfiltration Backwash. *Jour. AWWA*, 96(1):72–83.

MacPhee, M., D. Cornwell, and R. Brown. 2002. *Trace Contaminants in Water Treatment Chemicals*. Denver, Colo.: AwwaRF and AWWA.

Mickley, M. 2000. *Major Ion Toxicity in Membrane Concentrate*. Denver, Colo.: AwwaRF and AWWA.

Mickley, M. 2001. *Membrane Concentrate Disposal: Practices and Regulation*. Report No. 69, U.S. Department of Interior, Bureau of Reclamation, Technical Service Center, Water Treatment Engineering and Research Group [Online]. Available: <http://www.usbr.gov/pmts/water/media/pdfs/report069.pdf>. [cited July 7, 2004]

Mickley, M. N.d. *Membrane Concentrate Disposal: Practices and Regulation*. 2nd ed. Report No. 69. U.S. Department of Interior, Bureau of Reclamation, Technical Service Center, Water Treatment Engineering and Research Group. Forthcoming.

CHAPTER 7
ION EXCHANGE AND INORGANIC ADSORPTION PROCESS RESIDUALS

Ion exchange and adsorption processes can be used to remove cations, anions, and ionic complexes from water. These processes function by passing the raw water through a vessel containing a solid medium. In ion exchange, the ion removed exchanges onto the medium for another ion of lesser attraction to the medium. The ion released from the medium ends up in the finished water. The process will continue until the medium is exhausted (i.e., no more contaminant is removed). At that point, the medium is regenerated by passing a volume of water containing a high concentration of an exchangeable ion past the medium. The medium releases the contaminant ion and accepts the exchangeable ion. Therefore, the spent regenerant will contain the contaminant ion and excess exchangeable ion. An IX medium can be a strong-acid cation (SAC) exchanger, weak-acid cation (WAC) exchanger, or anion exchange medium. Adsorption processes work in a similar fashion although occasionally an absorption medium is not regenerated and the medium is discarded when exhausted (see chapters 8 and 9).

The primary cations removed by IX or adsorption include

- Hardness using usually SAC but occasionally WAC
- Barium using SAC
- Radium using SAC
- Radon using activated alumina (AA), specialty resins, and GAC

The anions that are removed by IX or adsorption include

- Fluoride using AA
- Nitrate using anion exchange
- Arsenic using AA, IX, or iron-impregnated resins
- Perchlorate using anion exchange
- Selenium using AA or anion exchange
- Chromium using anion exchange

By far, the most common IX process used is softening for the removal of calcium and magnesium. The following section addresses softening residuals. The next subsections include discussions on some of the other contaminants removed by IX or adsorption. Chapters 8 and 9 address arsenic and radon, respectively.

SOFTENING

In water softening by IX, hard water is passed through a column containing the IX material. The hardness in the water exchanges with an ion from the IX material. Generally, the ion exchanged with the hardness is sodium:

$$\mathrm{Ca\,(HCO_3)_2 + 2NaR = CaR_2 + 2NaHCO_3} \tag{7.1}$$

Table 7.1
IX softening resin capacity

Salt used (lb/ft^3)	Exchange capacity (kg/ft^3)	Regenerate use (lb NaCl/kg removed)
6	1.4–1.6	4.0
10	1.8–2.0	5.3
15	1.9–2.2	7.5

NOTES: $lb/ft^3 \times 16.017 = kg/m^3$; $ft^3 \times 0.02832 = m^3$.

where R represents the solid IX material. By the reaction in Equation 7.1, calcium (or magnesium) has been 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 essentially 100 percent removal of the hardness from the water until the exchange capacity of the IX material is reached. When the IX resin becomes saturated, "breakthrough" is said to have occurred because the hardness is no longer removed. At this point, the IX material is regenerated. During regeneration, the hardness is removed from the material by passing water containing a large amount of Na^+ through the column. The mass action of having so much Na^+ in the water will cause the hardness on the IX material to enter the water and exchange with the sodium:

$$CaR_2 + 2NaCl = 2NaR + CaCl_2 \tag{7.2}$$

The IX material can now be used to remove more hardness.

Ion exchange has been used for many years as a softening process. Most large plants that utilized IX for softening in the past were located near coastal areas so that brine wastes were discharged to the ocean. Many of these plants have been abandoned because of corrosion and high maintenance costs. However, IX is still being practiced by small treatment systems. Its chief advantage over lime softening is the ease of operation. In areas where disposal of lime sludges is a problem, IX may also be favorable if a suitable brine waste discharge point can be found. Wastes are produced from an IX process from the three phases of bed cleaning: backwash, regeneration, and rinse. IX columns will normally be backwashed at a 5-to-6-gpm/ft^2 (3-to-4-L/m^2/sec) rate for about 10 minutes, therefore generating 50 to 60 gal (190 to 230 L) of backwash waste for each square foot of contactor area. The regenerate, or brine waste volume, depends upon the type of exchange media used, the amount of cation capacity that is available, and the efficiency of regeneration. Natural greensand filters have an exchange capacity of about 0.2 kg/ft^3 (7 kg/m^3). (The units used for IX are in terms of kilograms of hardness expressed as $CaCO_3$. Hardness refers to any divalent or higher cation.) The capacity of synthetic IX resins depends upon the regenerant driving force used, as shown in Table 7.1.

The theoretical salt (NaCl) demand for regeneration is 2.6 lb NaCl/kg (1.2 kg/kg) hardness removed. Most plants will operate between 8 and 10 lb/ft^3 (130 and 160 kg/m^3) regeneration, whereas small "homeowner" systems will operate at 6 lb/ft^3 (96 kg/m^3). The strength of the initial regenerant solution will typically be between 8 and 18 percent NaCl. This results in the waste production, considering only the regeneration step of bed cleaning, as shown in Table 7.2.

Table 7.2
Volume of spent regenerate produced in IX softening

Regenerant use (lb NaCl/kg removed)	Volume of regenerant used for indicated solution strength, gal/kg removed			
	6%	10%	12%	16%
4.0	8.0	4.8	4.0	3.0
5.3	10.6	6.4	5.3	4.0
7.5	15.0	9.0	7.5	5.6

NOTES: lb/ft^3 × 16.017 = kg/m^3; ft^3 × 0.02832 = m^3.

Therefore, for every kilogram of hardness removed, approximately 3 to 15 gal (11 to 57 L) of brine are produced. This brine will contain the excess salt from the brine and the cations removed from the resin.

The rinse process is conducted in a downward flow mode for about 30 minutes and uses 20 to 40 gal/ft^3 (2,600 to 5,300 L/m^3) of resin volume. The following example will illustrate the calculation of IX waste production.

A 1-mgd (3.8-ML/d) plant with an initial hardness of 224 mg/L as $CaCO_3$ reduces the hardness to 85 mg/L using a synthetic IX resin. The regeneration rate is 6 lb NaCl/ft^3 using a 10 percent salt solution.

Hardness removed = 224 – 85 = 139 mg/L
Total removed = 5.2×10^2 kg/d

The exchange capacity of the media is about 1.5 kg/ft^3 (53 kg/m^3) (Table 7.1); therefore, every day 350 ft^3 (520 kg/d/1.5 kg/ft^3 [10 m^3]) of resin is exhausted. If the contactor operates at 6 gpm/ft^2 (14.6 m/hr), then the surface area is about 72 ft^2 (7 m^2) (because the resin removes essentially all the hardness, 62 percent of the flow would pass through the contactor and 38 percent would be bypassed in order to result in a final water containing 85 mg/L hardness). The amount of backwash water produced at 50 gal/ft^2 (400 L/m^2) would be about 3,600 gal (13,500 L). The concentrated brine waste produced would be 4.8 gal/kg (18 L/kg) (Table 7.2) or 2,500 gal (9,500 L). The rinse water rate wastage would be about 10,500 gal (39,700 L). Total residual flow, therefore, is 16,600 gpd (63 m^3/d) for the 1-mgd (3.8-ML/d) plant.

The volume of brine solution produced can also be determined in terms of bed volumes (BVs). Bed volume, as its name implies, is the equivalent volume occupied by the media. Therefore, 1 ft^3 of resin would have a BV of about 7.5 gal (28 L). The number of BVs that a column can operate prior to exhaustion can be found by dividing the exchange capacity by the hardness of the raw water. A resin regenerated with 6 lb/ft^3 (96 kg/m^3) of salt has an exchange capacity of about 1.5 kg/ft^3 (53 kg/m^3) or 50,000 mg/L. In our example, the raw water hardness was 224 mg/L, so the BVs to exhaustion would be 220. Because the resin removes essentially all the hardness, and given that the final desired hardness was 85 mg/L, 38 percent of the water would be bypassed around the column and blended with the treated water to reach the final desired hardness (85/224 = 38 percent). Therefore, the IX system would treat an equivalent of 350 BV (220/0.62).

Table 7.3
Residuals quantities produced by softening IX plants

Plant	Gallons wastewater per 1,000 gal water processed*	Raw water total hardness (mg/L as $CaCO_3$)	Gallons regenerant per ft^3 resin	Gallons rinse per ft^3 resin	Concentration of brine (%)	Gallons processed per ft^3 resin-cycle	Dosage (lb $NaCl/ft^3$ resin)
Crystal Lake Plant No. 6	21.9	233	7.3	19.4	11	1,220	6.6
Crystal Lake Plant No. 8	17.2	244	5.1	19.0	15	1,400	6.5
Eldon	71.9	75	3.9	61.7	17	750	5.6
Grinnel	49.5	388	14.5	35.0	6	1,000	7.2
Holstein	53.5	885	5.7	19.7	14	475	6.6
Estherville	82.8	915	4.4	24.7	15	350	5.5

Source: Adapted from Snoeyink 1984.

NOTES: $ft^3 \times 0.02832 = m^3$; $lb/ft^3 \times 16.017 = kg/m^3$; gal × 3.785 = L.

*Includes backwash water, regenerant waste, and rinse water.

The BVs of regenerate used is a function of the volume of regenerate used and the hardness removed. At a regenerate use of 4.0 lb NaCl/kg (1.8 kg/kg) removed and a 10 percent solution, the regenerate use is 4.8 gal/kg (18 L/kg) hardness removed (or 0.65 ft^3/kg [0.02 m^3/kg]) and the exchange capacity is about 1.5 kg/ft^3 (53 kg/m^3) for a total regenerate or brine volume of 1.0 BV (0.65 ft^3/kg × 1.5 kg/ft^3 [0.02 m^3/kg × 53 × 53 kg/m^3]).

The ratio of brine to treated water is about 0.3 percent (1.0 × 100/350), or approximately 3,000 gal (11,400 L) of brine for 1 mgd (3.8 ML/d) of finished water.

Table 7.3 shows data collected by Snoeyink (1984) on quantities of residuals produced by softening IX plants at six sites. The amount of rinse water used at each plant was in the range of 20 to 40 gal/ft^3 (2,600 to 5,300 L/m^3) of resin. For Crystal Lake Plant No. 6, which was closest to the conditions in the previous example, the regenerate use was about 1 BV, similar to that calculated in the example.

The TDS concentration of the residual can be estimated as the excess salt plus the cations removed. All of the chloride should be present as well as the excess sodium not used in exchange. The theoretical NaCl demand is 2.6 lb/kg (1.2 kg/kg), and NaCl is 40 percent sodium; therefore, the theoretical sodium demand is 1.04 lb/kg (0.47 kg/kg). In this example, 4 lb/kg (1.8 kg/kg) of NaCl was supplied, or 1.6 lb/kg (0.7 kg/kg) of sodium. The total computation is therefore

Excess sodium = (1.6 – 1.04) 520 = 291 lb
Chloride = 0.6(4) (520) = 1,248 lb
Hardness, as Ca^{+2} = 0.4 (520) 1,000/454 = $\underline{458\ \text{lb}}$
1,997 lb

The TDS of the regenerant waste solution itself would therefore be 96,700 mg/L, assuming 2,500 gal (9,500 L) of brine. If blended with the rinse water, the strength would be 18,600 mg/L.

Fluoride Removal by Activated Alumina

As indicated earlier, there are several ions other than hardness that can be successfully removed by IX or adsorption processes. Of these, probably the use of AA for fluoride removal is the most widely practiced. AA will also remove arsenic and selenium. AA is mixtures of amorphous and gamma aluminum oxide prepared by low-temperature dehydration of aluminum hydroxide. The fluoride adsorption onto AA can be represented as

$$\mathrm{R{\cdot}OH + H^+ + F^- \rightarrow R{\cdot}F + HOH} \tag{7.3}$$

AA for anion adsorption works best at a pH <8.2. The optimal pH is 5.5 to 6.0. Therefore, the neutral AA is treated with acid and protonated so that the reaction with fluoride and a protonated alumina is written as

$$\mathrm{HCl + HF \rightarrow R{\cdot}HF + HCl\ R{\cdot}} \tag{7.4}$$

To regenerate the alumina once it is exhausted with fluoride, a dilute solution of 0.25 to 0.5 N sodium hydroxide is used. Because alumina is both a cation and anion exchanger, Na^+ is exchanged for H^+, which immediately combines with hydroxide to form water (Clifford 1999). The reaction is written as

$$\mathrm{R{\cdot}HF + 2NaOH \rightarrow R{\cdot}NaOH + HF + HOH} \tag{7.5}$$

In order to restore the fluoride removal capacity, this alumina that has been regenerated with sodium hydroxide is acidified by contacting it with excess acid. The acid used is generally 0.5 N sulfuric acid or hydrochloric acid and the reaction is written as

$$\mathrm{R{\cdot}NaOH + 2\ HCl \rightarrow R{\cdot}HCl + NaCl + HOH} \tag{7.6}$$

As an alternative to rinsing the basic alumina with acid, the source water pH can be reduced to 5.5 to 6.0. This will avoid the production of an acid rinse residual.

Table 7.4 shows a typical fluoride capacity for AA as a function of the fluoride concentration in order to reach a final fluoride level of 1.4 mg/L.

Prior to regeneration, the exhausted column is backwashed to remove particles and to restratify the media. Backwashing is accomplished at a flow rate of 8 to 9 gpm/ft^2 (19.5 to 22.0 m/hr) for 5 to 10 minutes. This backwash will result in the range of 8 BV of spent backwash water being produced.

Table 7.4
Fluoride capacity of AA

Raw water fluoride (mg/L)	Raw water TDS (mg/L)	Run length (BV)	Fluoride capacity (g/m^3)
2.0	810	2,300	3,700
3.0	1,350	1,200	3,000
5.0	1,210	1,150	4,600

Source: Adapted from Clifford 1999.

A slow rinse follows both the sodium hydroxide regeneration and the acid rinse. The rinse takes place at about 0.5 gpm/ft^3 (1.2 m/hr) and is equivalent to about 2 BV for each rinse. The sodium hydroxide regeneration requires about 0.5 gpm/ft^3 flow rate for ~75 minutes, or about 5 BV. A 1 percent NaOH concentration is typically used. The acid rinse is also done at a flow rate of 0.5 gpm/ft^3 with the use of about 1.5 BV at a 2 percent sulfuric acid concentration. A final rinse of about 30 to 50 BV is used prior to placing the AA back in service.

In another example, a 1-mgd plant (694 gpm [44 L/sec]) treats 3-mg/L F at a loading rate of 6 gpm/ft^2 (14.6 m/hr) and uses a 5-minute empty bed contact time (EBCT). The total F treated per day is 11,250 g, and at an exchange capacity of 3,000 g/m^3 (Table 7.4), a total of 3.15 m^3 (134 ft^3) of media are exhausted per day. The number of BVs treated to exhaustion would be

$$\text{treated BV} = \frac{1{,}000{,}000 \text{ gpd}}{(134 \text{ ft}^3\text{/day})(7.5 \text{ gal/ft}^3)} = 995 \tag{7.7}$$

A 6-gpm/ft^2 loading rate would result in an approximate 115-ft^2 (11-m^2) vessel (694/6).

The initial rinse would, therefore, be 1,035 gpm (66 L/sec) (9 gpm/ft^2 × 115 ft^2) for 10 minutes, or 10,350 gal (3.9×10^4 L) total of backwash would be produced each time the vessel is regenerated. If the vessel has a 5-minute EBCT, that is equivalent to 3,470 gal (1.3×10^3 L) (460 ft^3). Therefore, the vessel would last about 3.4 days (460/134) before the need for regeneration. The spent backwash BVs produced is

$$\text{spent backwash BV} = \frac{10{,}350 \text{ gal}}{(460 \text{ ft}^3)(7.5 \text{ gal/ft}^3)} = 3 \tag{7.8}$$

The sodium hydroxide regeneration would produce

$$\text{spent NaOH regenerate} = (0.5 \text{ gpm/ft}^3)\ (460 \text{ ft}^3)\ (75 \text{ min}) = 17{,}250 \text{ gal} = 5 \text{ BV} \tag{7.9}$$

Table 7.5
Residual characteristics of AA used to remove fluoride at Palo Verde, Ariz.*

	F^-	Al	As	$SO_4^=$	Na	TDS	Mg	Turbidity (ntu)	Ca	Volume percent of flow treated
Backwash	1.8	230	0.210	220	225	880	2.8	17	53	0.6
NaOH upflow regeneration	3.2	13	0.017	184	750	1,344	4.3	6.3	30	0.12
Upflow rinse	250	150	0.140	640	740	2,629	1.8	20	6.9	0.9
NaOH downflow regeneration	23.5	11	0.030	156	410	1,264	0.2	1.0	7.5	0.12
Acid and final rinse	8.4	150	0.475	580	1,240	2,879	2.9	18	6.8	0.7
Residual composite 3,010 gal	91.5	156.4	0.246	469.2	743.5	2,130	2.4	17	19.7	2.5

Source: Adapted from Snoeyink 1984.

*The terms have been modified from the original source and the residual volume percentages were calculated. All units are in milligrams per liter unless noted.

The acid wash and displacement rinses are done for about 2 BV or 6,900 gal (26,000 L) each. The total residuals production per regeneration would be summarized as

Spent backwash = 10,350 gal = 3 BV
Spent NaOH = 17,250 gal = 5 BV
Displacement rinse = 6,900 gal = 2 BV
Acid rinse = 6,900 gal = 2 BV
Displacement rinse = 6,900 gal = 2 BV
Final rinse = 138,000 gal = 40 BV
Total 186,300 gal 54 BV

The percentage of residuals produced relative to the water treated is calculated by either the BV or the volume, 54/995 or (186,300 gal/regeneration)/(1,000,000 gpd × 3.4 days/regeneration), which is 5.4 percent.

Snoeyink (1984) presented characteristics of residuals produced at two AA plants, as shown in Tables 7.5 and 7.6. In both cases, the sodium hydroxide regeneration was done in both an upflow and downflow mode. The acid and final rinse was combined by rinsing using raw water with the pH lowered to about 3.2 using sulfuric acid. Also shown is the percentage each residual represents to the total volume of raw water treated.

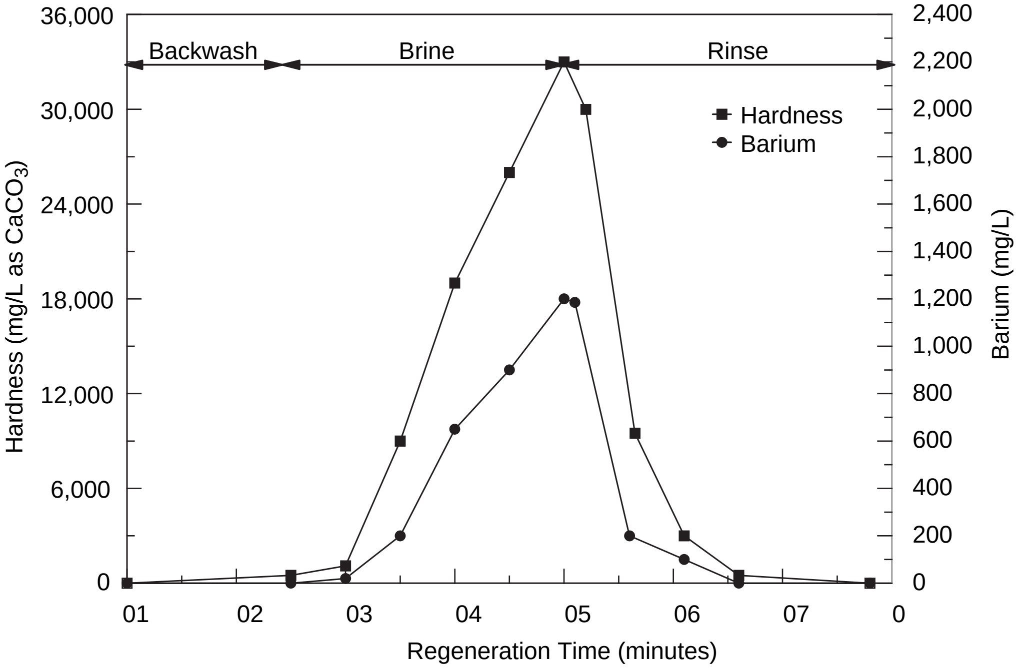
Table 7.6
Residual characteristics of AA used to remove fluoride at a plant in Gila Bend, Ariz.*

	F^-	Al	As	$SO^=_4$	Na	TDS	Cl^-	Mg	Turbidity (ntu)	Silica	Ca	Cr	Fe	Volume percent of flow treated
Backwash	5.4	2.1	<0.001	219	438	1,278	590	0.4	300.0	9.5	33.0	0.013	14.5	0.4
NaOH upflow regeneration	5.1	0.42	<0.001	206	444	1,310	590	0.4	140.0	7.7	32.0	0.016	NA†	0.17
Rinse A	6.1	0.22	0.076	739	741	2,338	700	0.3	45.0	NA	30.5	0.026	NA	0.08
Rinse B	192	193.3	0.49	721.5	1,319	3,704	605	<0.1	105.5	5.2	2.0	0.031	NA	0.9
NaOH downflow regeneration	33.4	8.0	0.050	187	528	1,453	580	NA	1.0	2.3	4.7	0.021	NA	0.1
Acid and final rinse	5.6	20.1	0.054	162.6	454	1,380	570	<0.1	0.38	3.3	1.4	0.010	NA	3.0
Residual composite	41.9	50.7	<0.13	285	624	1,832	581	NA	49	NA	5.6	0.015	NA	4.65
Raw water	5.0	0.08	0.014	174	432	1,260	570	0.4	0.5	10.6	35.0	0.017	0.11	NA

Source: Adapted from Snoeyink 1984.

*The terms have been modified from the original source and the residual volume percentages were calculated. All units are in milligrams per liter unless noted.

†NA = not analyzed.



Source: Snoeyink 1984.

Figure 7.1 Barium and hardness concentrations in spent brine well no. 6, Crystal Lake, Ill.

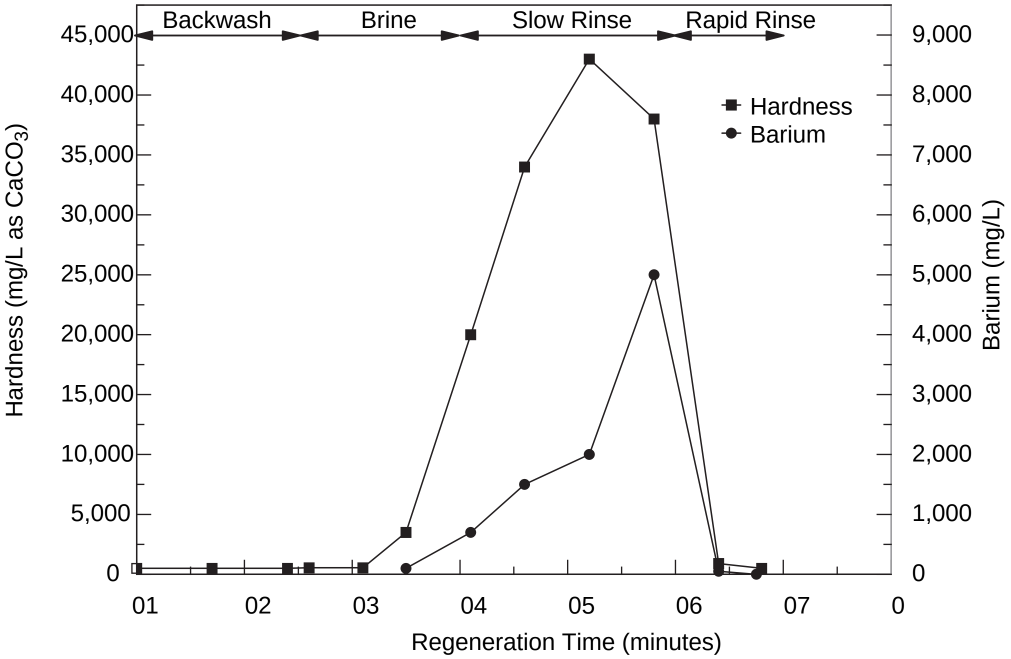
Barium Removal

Barium is removed identically to hardness, and the regeneration is similar. However, the barium is more difficult to remove from the resin than hardness, and more excess NaCl is required. Two IX softening plants located in Crystal Lake, Ill., treat groundwater containing 9.5 mg/L and 16.1 mg/L Ba^{2+} at wells no. 6 and no. 8, respectively (Figures 7.1 and 7.2). At well no. 8, 153 percent of the barium removed by the IX column during the service cycle was found in the spent regenerant (more than 100 percent barium removal from the column is possible if the previous regeneration was incomplete). The distribution of this barium was 1.8 percent in the backwash water, 3 percent in the brine, and 148 percent in the rinse water. The 10,000 gal (3.79×10^4 L) of residual produced from one regeneration cycle contained 30 kg of barium (800 mg/L) (Snoeyink 1984).

At well no. 6, 84 percent of the barium placed on the column was removed during regeneration. Almost 2 percent was removed during backwash, 45 percent during the brine application, and 37 percent during the rinse cycle. Six and one-half kilograms of barium were contained in the 9,500 gal (3.6×10^4 L) of residual regenerated from one regeneration cycle (180 mg/L).

Sorg and Logsdon (1980) found that the regeneration of the IX resin in their study removed 85 percent of the barium placed on the column. Eighteen kilograms of barium was contained in the 9,250 gal (3.6×10^4 L) of residual generated. The average residual barium concentration in this plant was 534 mg/L, with a peak concentration of 6,000 mg/L Ba. The residual volume was 2.6 percent of the produced water.

Barium concentrations in the water and residual at Crystal Lake are given in Table 7.7. Figures 7.1 and 7.2 show hardness and barium concentrations during regeneration for wells no. 6



Source: Snoeyink 1984.

Figure 7.2 Barium and hardness concentrations in spent brine well no. 8, Crystal Lake, Ill.

Table 7.7
Crystal Lake's water and residual barium concentrations

	Ba in raw water (mg/L)	Peak Ba during regeneration (mg/L)	Average concentration in brine and rinse water (mg/L)	Average concentration in backwash, brine, and rinse water (mg/L)
Well no. 6	9.5	1,197	328	153
Well no. 8	16.1	5,161	1,297	789

Source: Snoeyink 1984.

and no. 8. By comparing these figures, it can be seen that the peak hardness and barium concentrations coincide at well no. 6, but the peak hardness value occurs before the peak barium concentrations at well no. 8. This phenomenon may be caused by the greater amount of the barium on the column before regeneration at well no. 8 (78.4 g Ba/ft^3 [2.19 g/m^3]) compared with that at well no. 6 (40.4 g Ba/ft^3 [1.13 g/m^3]). More barium accumulates on the resin during the service cycle at well no. 8 because of a higher raw water barium concentration, and less regenerant is applied to the column per unit volume of water produced during the service cycle.

Table 7.8
Brine characteristics at full-scale perchlorate and nitrate treatment facilities

Utility*	Treatment type	Capacity (gpm)	Contaminants (concentration)	Residual type	Quantity	Residual handling	Permits
A	Regenerable IX	9,000	Nitrate (31–84 mg/L) Perchlorate (~14 µg/L)	Liquid brine	~250,000 gpd of brine	Brine discharge	NA†
B	Regenerable IX	2,500	Nitrate (15 mg/L) Perchlorate (35 µg/L)	Liquid brine	25 gpm of brine	Brine discharge until 2006; Perchlorate brine treatment by 2006	Perchlorate discharge
D	Regenerable IX	1,350	Nitrate (45 mg/L)	Liquid brine	1,700 gpd of brine	Brine trucking to a WWTP by the equipment provider	
G	Regenerable IX	1,000	Nitrate (50 mg/L)	Liquid brine	6,000 gpd	Brine discharge to a WWTP (pending)	
H	Regenerable IX	400–2,100	Nitrate (55–70 mg/L)	Liquid brine	140,000 gpd	Brine discharge to a WWTP	
I	Regenerable IX	3,000	Nitrate (34–90 mg/L)	Liquid brine	6,000 gpd	Regional WWTP	
J	Regenerable IX	7,900	Perchlorate (NA)	Liquid brine	80 gpm of perchlorate brine	Brine discharge until 2006; brine treatment by 2006	

Source: Min et al. 2004.
NOTES: gpm × 0.06309 = L/sec; gal × 3.785 = L.
*All utilities are located in California.
†NA = information not available.

Removal of Nitrate and Perchlorate Using Anion Exchange

Nitrate and perchlorate can each be removed by anion exchange resins. Sodium chloride is used as the regenerate. Min et al. (2004) summarized the amount of residual produced at several operating nitrate and perchlorate plants (Table 7.8). Also shown are approximate values for nitrate and perchlorate in the brine residual as well as the disposal method being used.

Min et al. (2004) also reported quality analysis on a regenerate brine collected from La Puente Valley County Water District in California. A continuous IX process was used to treat a raw water containing about 25 mg/L nitrate and 0.10 mg/L perchlorate. A 7 percent salt was used for regeneration producing a brine of about 1 percent of production.

Table 7.9 provides the anion analysis of the composite solutions contained in Drums A and B, two samples that were analyzed. The nitrate concentration of each composite is similar. However, because of very high sulfate concentration of Drum no. 3, sulfate concentration in composite Drum A is more than three times higher than that of composite Drum B.

Table 7.9
Anion analysis of La Puente IX brine composites (in mg/L)

Drum	NO_3^-	NO_2^-	SO_4^{2-}	PO_4^{3-}	Cl^-	ClO_4^-
A	1,347	ND*	2,849	ND	30,443	4.121
B	1,593	ND	900	ND	24,895	4.588

Source: Min et al. 2004.
*ND = not detected.

Table 7.10
Solids and other analyses of La Puente IX brine composites (in mg/L)

Drum	TS	TSS	TDS	TOC	TKN	pH	Conductivity (mS/cm)
A	89,725	7.5	68,625	2.6	>0.20	7.17	91.6 (6.78% TDS)
B	53,500	5.0	51,980	2.2	0.21	1.61	80.5 (5.84% TDS)

Source: Min et al. 2004.

Table 7.11
ICP analysis of La Puente brine composites (in mg/L)

Drum	As	Ca	Co	Cu	Fe	K	Mg	Mn	Mo	Na	Ni	P
A	0.16	23	0.032	0.10	0.58	3.5	2.5	0.015	0.58	26,000	<0.040	1.6
B	0.062	21	0.034	0.64	0.56	4.3	2.4	0.011	0.27	20,000	<0.040	4.5

Source: Min et al. 2004.

Table 7.10 provides solids and other analyses of the composite solutions. Solids analysis (TS, TSS, and TDS) by standard gravimetric methods was difficult to conduct because the high bicarbonate content of the brine made complete water evaporation difficult to achieve. However, there is a general agreement between the measured TDS and the TDS based on conductivity for sodium chloride solutions. The alkalinity of Drum A was measured at approximately 8,000 mg/L.

Table 7.11 provides the results of ICP metal analyses. Generally, the results agree with the expected values in the IX brine. Sodium concentrations were consistent with chloride concentrations.

Min et al. (2004) also summarized the regulations affecting nitrate and perchlorate brines and the treatment options, which are discussed in the following paragraphs.

There are no established regulations for discharging nitrate wastes, although there may be some local restrictions based on the regional basin management goal. USEPA is progressing with ecoregional nutrient criteria for surface water quality. These criteria are not necessarily applicable for regulating nitrate discharge, but they may provide insight for potential future requirements, based on ecoregion location for discharging of nitrate waste. Where ocean discharge is available, a different set of regulations may apply. In California, ocean discharge is regulated by the California Ocean Plan (California Environmental Protection Agency 2001). Nitrate is not listed specifically by the Ocean Plan, but a narrative requirement states that “Nutrient materials shall not cause objectionable aquatic growths or degrade indigenous biota.”

In California, the Orange County Sanitation District (OCSD) responsible for ocean discharge does not currently limit the nitrate into the brine line known as the Santa Ana Regional Interceptor (SARI) line. However, OCSD is expected to add constituents and adjust local limits as it transitions toward implementation of the groundwater replenishment system (GWRS). These additions and limits are based on the California Department of Health Services (DHS) Title 22 requirements since the treated GWRS water will be percolated into the Orange County groundwater basin. Additionally, the Santa Ana Watershed Project Authority is beginning an evaluation of the SARI System Enhancements Program that, if implemented, would require treatment of nitrate prior to discharge into the SARI line, which would then discharge directly into the OCSD outfall, bypassing OCSD’s Plant No. 2. In addition, if Plant No. 2 becomes a reclamation plant, limits could be imposed for nitrate by DHS. Similar local and state regulations may be imposed elsewhere when such approaches are implemented.

Currently, there is limited information available on perchlorate discharge regulation, in part because there are no established state or federal drinking water regulations for perchlorate. In California, where several treatment plants produce perchlorate-laden IX brine, different agencies are involved with regard to regulating disposal of the perchlorate waste. As part of the Superfund cleanup activities, several utilities in the San Gabriel area in Los Angeles County are required to treat perchlorate brine by 2006 under USEPA’s Record of Decision requirement. This regulation and time line is limited to utilities involved in the Superfund program.

In 2003, the Los Angeles County Regional Water Quality Control Board adopted a 4-µg/L perchlorate discharge limit to the NPDES permit. This limit was set to the action level for perchlorate at that time. Shortly after that, the Los Angeles County Sanitation District (LACSD) implemented the same 4-µg/L maximum discharge limit to their system where ocean discharge is not available. For areas where the ocean discharge is available, LACSD does not limit the amount of perchlorate that is discharged into the line as long as the contaminant does not originate from a Superfund site.

On the other hand, OCSD currently regulates perchlorate discharge given that it is an emerging contaminant and environmental effects are unknown. There is no set limit on the amount of perchlorate that can be discharged to OCSD. Permits are granted for discharge on a case-by-case basis. Before a permit can be granted for the discharge of perchlorate into the system, a letter pertaining to the discharge characteristics must be submitted to OCSD for their review of the project. Important parameters to be considered by OCSD include duration of project, minimum and peak flows, minimum and peak perchlorate levels in waste, and characterization of source waters. These criteria are reviewed to set the discharge limit.

In Kansas, the Kansas Department of Health and Environment allows a manufacturing facility (Kerr McGee) to discharge up to 100 µg/L of perchlorate to a WWTP. Utah currently does not impose any additional restrictions.

The principal options for treating or disposing of nitrate and perchlorate brines are biological degradation followed by discharge, thermal degradation followed by discharge, or direct discharge to a WWTP or body of water if allowed. Significant research has been conducted on both degradation processes, but actual application is limited.

Disposal of Brines

Economics dictate that a readily available location be used for disposal of the brine. Therefore, most large plants that utilize IX softening are located in coastal communities so that ocean brine disposal is practiced. Ion exchange has been used in small water supply systems in other parts of the country, and residuals have most often been discharged to municipal wastewater systems or to receiving streams. Two additional waste streams are also produced in conjunction with IX. Prior to the use of the regenerant, the column is usually backwashed in an upflow mode to remove any suspended material. After regeneration, the column is rinsed, which produces a waste stream that is high in dissolved solids.

One of the major ways to dispose of the residuals is discharge to a wastewater plant. The effects of changing concentrations of salts in the wastewater due to slug discharge of high TDS brine is of considerable significance. A concentration change of 100 to 200 mg/L of NaCl is rather low and would have no impact on the biological process, whereas a concentration change of 35,000 mg/L NaCl (almost an undiluted discharge of IX brine) would cause considerable stress to the biological organisms. A mid-range change of 10,000 mg/L NaCl can actually stimulate growth. Figure 7.3 shows response curves of a completely mixed, once-through biological reactor to shock loadings of sodium chloride. Figure 7.3(a) is a dose response to 30,000 mg/L NaCl. As the salt was added, the biomass concentration rose slightly and then decreased significantly. Correspondingly, the effluent COD increased almost immediately after the salt addition. Within 2 days, the substrate removal efficiency was regained. The volatile solids concentration was abnormally low but had partially recovered at day 6.

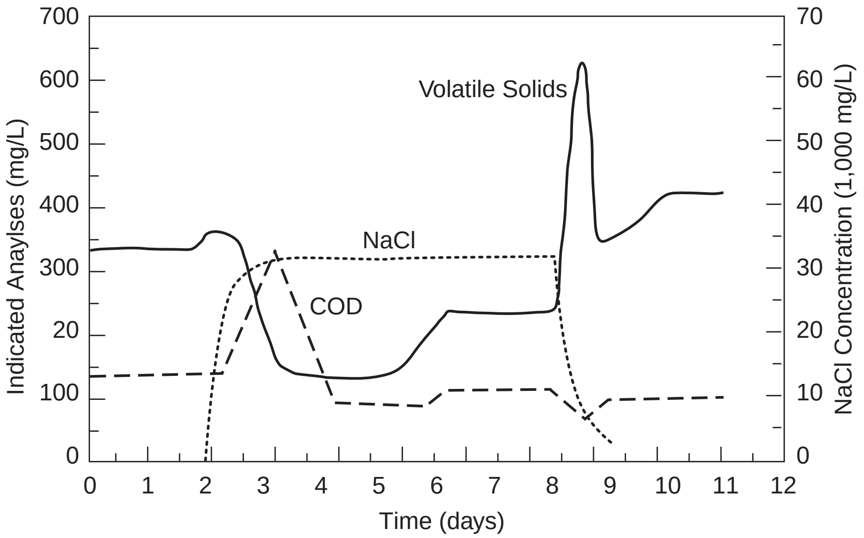
Upon removal of the salt feed to the system, the volatile solids immediately increased and leveled off at a slightly higher concentration than prior to the shock loading. Figure 7.3(b) shows the effects of a shock loading of 10,000 mg/L NaCl. A significant increase in cell yield (about 75 percent) was sustained until the salt feed was stopped. The effluent COD also decreases during the period of 10,000 mg/L NaCl addition.

From this research, it would appear that the effects of brine discharges from WTPs should have an insignificant impact on the wastewater treatment process as long as some precautions for EQ and dilution are followed. Toxicity of specific compounds within the waste streams should be carefully monitored and diluted sufficiently to eliminate toxic effects.

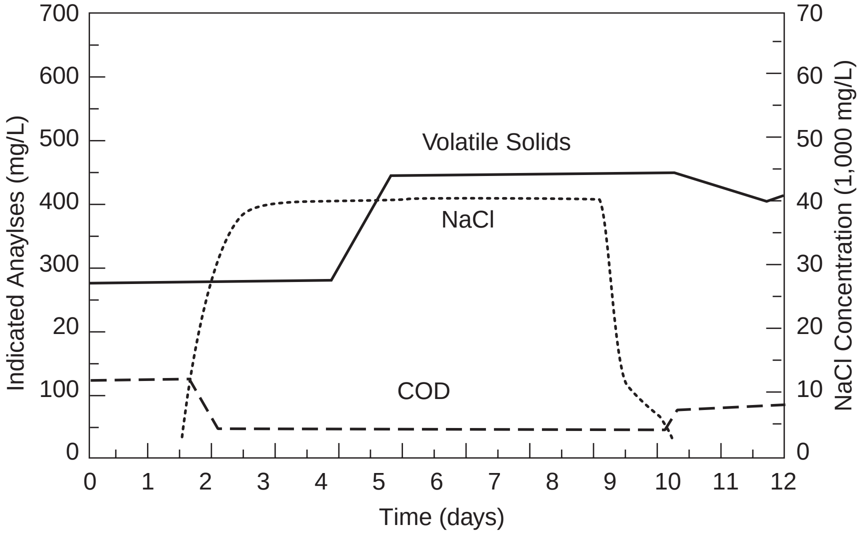
In addition to effects on the aerobic biological process, salts can affect anaerobic digestion. Toxicity is normally associated with the cation rather then the anion of the salt. Table 7.12 shows the stimulatory and inhibitory concentrations for some cations of interest.

Limited data are available on the effects of barium on the activated sludge process. Experimental data indicate that the soluble barium concentrations would have to exceed 50 mg/L for toxicity to freshwater aquatic life to occur. In most wastewaters, sulfate or carbonate would be present to precipitate at least some of the barium as an insoluble, nontoxic compound. To precipitate 1 mg/L Ba requires about 0.7 mg/L SO_4^{-2} or 0.23 mg/L of oxidized sulfide. Calculations should be conducted to ensure that the dilution or reduction of barium is lowered to an acceptable level.

Selenium can be found in small concentrations in IX residuals from some groundwater treatment systems. Selenium is toxic to bacteria at 90 mg/L and 180 mg/L for protozoa. It is unlikely that these concentrations would be reached in a residual discharged to the sewer system.



(a) NaCl dose = 30,000 mg/L



(b) NaCl dose = 10,000 mg/L

Source: Adapted from Gaudy and Gaudy 1980.

Figure 7.3 Response of biomass to NaCl shock loading

Table 7.12
Stimulatory and inhibitory concentrations of alkali and alkaline cations in anaerobic biological treatment (in mg/L)

Cation	Stimulatory	Moderately inhibitory	Strongly inhibitory
Sodium	100–200	3,500–5,500	8,000
Calcium	100–200	2,500–4,500	8,000
Magnesium	75–150	1,000–1,500	3,000

Source: Adapted from McCarty 1964.

Chapter 5 also contains information on toxicity levels of several ions on aerobic and anaerobic wastewater treatment processes.

Discharge of the brine to brackish coastal water and to seawater is also a widely practiced method for disposal. Because the brines are primarily a high TDS residual consisting mostly of salt, the permitting process has been fairly easy. Membrane concentrates have been more difficult to permit in some states because of the requirements to pass acute toxicity tests. These tests could also be applied to IX brines, which would make permitting of their disposal more difficult. Mickley (2000) has extensively described the requirements and problems related to toxicity testing on membrane concentrates. The following summarizes issues raised by Mickley.

The discharge permits for membrane concentrates are increasingly requiring the use of the whole effluent toxicity (WET) tests (not to be confused with the CALWET for hazardous waste compliance).

WET tests involve exposing selected test organisms to a known concentration of effluent sample for a specified period of time and determining the effect of the exposure on the organisms. For industrial samples, the known concentration is called the effluent dilution percentage, where 100 percent corresponds to actual effluent, 50 percent is a dilution of the effluent by an equal part of balanced water at the same salinity, and so forth. Acute toxicity tests use an endpoint (effect) based on lethality, and chronic toxicity tests use endpoints of some chronic manifestation such as growth (average size) or reproducibility (number of offspring) characteristics. Most studies involve determination of the acute effect brines or concentrates on the marine organism *Mysidopsis bahia*, more commonly known as the mysid shrimp.

The WET tests follow the USEPA 96-hour acute toxicity guidelines (Weber 1993) for mysid shrimp. A screening WET test may simply involve exposure of the mysid to a single concentration, typically 100 percent, of the concentrate. The result of the test is expressed in terms of percent survival. A definitive WET test is a series of tests conducted at different dilution levels of the concentrate. The definitive WET test enables measurement of the degree of toxicity as expressed by the calculation of lethal concentration (LC) values, or LC_{50}, the concentration at which 50 percent of the organisms are killed. For the concentrate, the LC_{50} value is the dilution level (100 percent, 50 percent, 25 percent, etc.) of the concentrate that results in 50 percent lethality (50 percent survival). For example, 100 percent concentrate may result in 0 percent survival, whereas 50 percent dilution of the concentrate may result in 40 percent lethality (60 percent survival). In this case, the LC_{50} value would be between 50 and 100 percent, and closer to 50 percent. When a test is performed with a known toxicant, the LC_{50} may be expressed as the concentration of the toxicant that results in 50 percent lethality. In this case, LC_{50} values are expressed as concentrations, such as in milligrams per liter.

Marine organisms can adapt to and live in synthetic seawater or in a range of dilutions where most of the major ions are in the same relative proportions as found in natural seawater. There are, of course, upper and lower limits of salinity that are organism dependent. This type of water that maintains the same relative proportions of ions as in seawater is referred to as a “balanced” water, regardless of salinity. Although seawater itself can vary as to salinity, it varies much less in composition, and a common definition of seawater has proven useful. Table 7.13 lists the composition of natural seawater, according to its definition by the USEPA (Weber 1993).

Major ion toxicity (also referred to in the literature as major ion toxicity, major seawater ion toxicity, ion imbalance, salt toxicity, and ion toxicity), as demonstrated by failed WET tests, appears to occur where the tested water, regardless of salinity level, has a sufficient excess amount (or a sufficient deficit amount in some cases) of one or more commonly occurring (on a statistical

Table 7.13
Composition of natural seawater

Ion specie	Seawater concentration (mg/L)	Seawater weight fraction
TDS	30,893	Not applicable
Sr^{2+}	6.5	0.00021
F^-	0.66	0.00004
HCO_3^-	123.5	0.00400
Ca^{2+}	359.9	0.01160
SO_4^{2-}	2,381	0.07710
Mg^{2+}	1,135.7	0.03680
K^+	348.8	0.01130
Cl^-	17,002	0.55030
Na^+	9,463	0.30630

Source: Weber 1993.

basis) ions. There are readily available "balanced" salts that have the common salts in seawater proportions that can be used to make a balanced water at any salinity. A balanced water, then, is a water, regardless of salinity level, that has ionic species in seawater proportions.

The 10 most common seawater ions in terms of weight fraction are Cl^-, Na^+, SO_4^{2-}, Mg^{2+}, Ca^{2+}, K^+, HCO_3^-, Br^-, $B_4O_7^-$, Sr^{2+}, and F^-, with weight fractions as shown in the third column of Table 7.13.

When a water fails a WET test, it may trigger a regulatory process where the utility is required to determine the cause of the failure. USEPA guidelines for toxic identification evaluations (TIEs) do not yet recognize the role or existence of major ion toxicity. Thus, if a concentrate had toxicity only due to major ions, and a laboratory were to perform the manipulations defined in the TIEs, the manipulations would not provide any direct clues to the cause of toxicity because the manipulations are looking for different types of toxicity. Indeed, this very situation has occurred in past years, leaving utilities and their laboratories with an undefined source of toxicity.

Should major ion toxicity be considered differently than other types of toxicity, such as caused by heavy metals or pesticides? By definition, through the failure of the WET test, major ions can occur in toxic levels. Clearly, the discharge of effluent into a receiving water that changes the makeup of that receiving water is of some level of concern—regardless of the more exact nature of the effluent. Of most concern, however, are effluents containing toxins that (1) have short-term effects directly affecting life forms in the receiving water or (2) have long-term effects (such as bioaccumulation in the tissues of the aquatic life forms present in the receiving water). It appears that major ion toxicity does not generally fit into either of these situations. Excess levels of major ions, such as those typically found in concentrates, are quickly diluted in the receiving water leaving little, if any, observable effect on the receiving water life forms. Of course the relative amount of effluent to receiving water must be taken into account. And the major ions do not bioaccumulate in the same sense that potentially dangerous chemicals do.

While major ion toxicity regulatory policy must be addressed, there are considerations that should be taken into account in assessing the potential environmental impacts. First, several of the major ions, and arguably all of them, are essential for survival and general health. Second, the modest dilution of the effluent by receiving water or other waters can reduce or eliminate major ion toxicity. Third, although organisms can accumulate many of these ions (i.e., Ca^{2+} in skeletal structures), most species have also evolved into elaborate mechanisms for transporting and storing these common ions. Thus, they generally do not bioaccumulate in the same, potentially damaging, manner as other chemicals.

In general, there are no economically feasible technical options for specifically removing excess ions from concentrate within the existing economics of the municipal drinking water industry. In site-specific instances, there may be means of providing enhanced dilution of the concentrate prior to or at final discharge through blending or the use of end-of-the-pipe diffusers (Mickley 2000).

REFERENCES

California Environmental Protection Agency. 2001. State Water Resources Control Board [Online]. California Ocean Plan [Online]. Resolution 2000-108. Available: <http://www.swrcb.ca.gov/plnspols/oplans/op2001.pdf>. [cited November 16, 2000]

Clifford, D.A. 1999. Ion Exchange and Inorganic Adsorption. In *Water Quality and Treatment: A Handbook of Community Water Supplies*. 5th ed. New York: McGraw-Hill.

Gaudy, A.F., and E.T. Gaudy. 1980. *Microbiology for Environmental Scientists and Engineers*. New York: McGraw-Hill.

McCarty, P.C. 1964. Anaerobic Waste Treatment Fundamentals, Part Three. *Public Works*, 95(11):91.

Mickley, M. 2000. *Major Ion Toxicity in Membrane Concentrate*. Denver, Colo.: AwwaRF and AWWA.

Min, J., L. Boulos, J. Brown, D. Cornwell, T. LeGouellec, E. Coppola, S. Baxley, J. Rine, J. Hering, and N. Vural. 2004. *Innovative Treatment Alternatives to Minimize Residuals Containing Nitrate, Perchlorate, and Arsenic*. Denver, Colo.: AwwaRF and AWWA.

Snoeyink, V.L., C.K. Jongeward, A.G. Myers, and S.K. Richter. 1984. Characteristics and Handling of Wastes From Groundwater Treatment Systems. Paper presented at the AWWA Annual Conference, Dallas, Texas, June 10.

Sorg, T.J., and G.S. Logsdon. 1980. Treatment Technology to Meet the Interim Primary Drinking Water Regulations for Inorganics: Part 5. *Jour. AWWA*, 72(7):411.

Weber, C.I. 1993. Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms. EPA 600/4-90/027F. Washington, D.C.: USEPA.

CHAPTER 8
ARSENIC RESIDUALS

OVERVIEW

USEPA defined a standard for arsenic in drinking water, an MCL of 10 µg/L. For utilities that need to utilize an arsenic removal treatment technology, it is critical to evaluate the types of residuals that would be generated, their expected arsenic concentrations, and the residuals pretreatment strategies that would be required prior to final disposal. A utility needs to understand the residuals impacts from removing arsenic in the raw water.

This chapter includes materials from *Regulations on the Disposal of Arsenic Residuals From Drinking Water Treatment Plants* (USEPA 2000), *Arsenic Removal From Drinking Water by Coagulation/Filtration and Lime Softening Plants* (Fields, Chen, and Wang 2000), *Treatment of Arsenic Residuals From Drinking Water Removal Processes* (MacPhee, Charles, and Cornwell 2001), *Disposal of Waste Resulting From Arsenic Removal Processes* (Cornwell et al. 2003), and *Innovative Treatment Alternatives to Minimize Residuals Containing Nitrate, Perchlorate, and Arsenic* (Min et al. 2004).

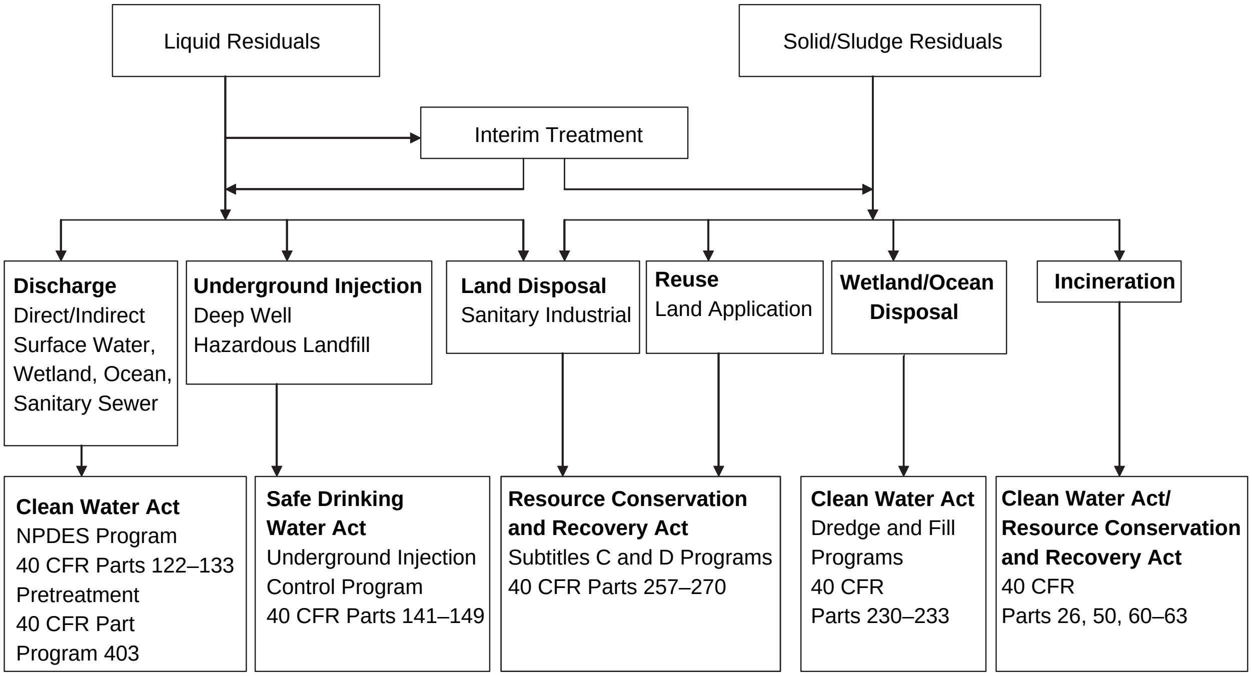
REGULATIONS

Arsenic Discharge and Disposal Regulations

Arsenic management methods are classified into two groups depending upon whether they address liquid residuals or solid/sludge residuals. In the case of liquid residuals, disposal methods may include discharge to a water body, indirect discharge via sanitary sewer system, underground injection, management in lagoons, and possible land application. Solid/sludge residuals may be disposed through landfill, land application, storage in lagoons, and reactivation, recovery, or disposal of spent resin. Federal and state regulations apply to handling, transportation, and disposal of arsenic-laden residuals. Most states have the administrative authority to implement these federal regulations, and are then required to establish and administer regulations meeting the requirements of these acts. Waste disposal regulations are primarily the responsibility of the states, but the state regulations must meet or exceed the federal regulations.

Federal Regulatory Review

Different federal regulations apply to various residual disposal methods. The main parameters controlling federal regulations applicability on arsenic removal residuals are the physical form of residuals (liquid or solid), the method used to manage residuals, and whether or not residuals constitute a hazardous or nonhazardous waste as defined under the RCRA. Figure 8.1 depicts how these three variables can determine which federal regulations apply to the management of arsenic residuals. No federal regulations specifically address WTP residuals disposal; however, existing federal regulations affect specific disposal methods of these residuals, as federal regulations aim to protect the receiving water, land, or underlying groundwater where residuals disposal occurs. Residuals from WTPs are disposed of according to the physical form of the residual liquid or solid.



Source: Adapted from USEPA 2000.

Figure 8.1 Arsenic residuals disposal—federal regulations

Liquid residuals can be disposed of by the following methods:

- Direct discharge (NPDES permit limits)
- Indirect discharge (POTW pretreatment limits)
- Underground injection (UIC Program)
- Beneficial use (RCRA)

Solids residuals are typically disposed of by

- Landfilling (RCRA)
- Beneficial use (RCRA)

Lagoons are also used to manage liquid or solid residuals; however, lagoons are only for temporary handling and are not considered a final disposal option. Each of these disposal methods is regulated by different requirements and regulations. The following sections provide a general overview of these regulations.

Liquid Residuals

Direct Discharge to Surface Waters. Discharge of pollutants to waters of the United States is regulated by the CWA, 33 USC s/s 1251 *et seq*. (1977), and a 1977 amendment to the 1972 Federal Water Pollution Control Act. The act was amended in 1978, 1980, 1981, and 1987, and aims to ensure that water bodies maintain the appropriate quality for their intended uses, including navigation, swimming, fishing, agriculture, and drinking water supply (Cornwell et al. 2003).

The CWA makes it unlawful for any person to discharge any pollutant into navigable waters or wetlands unless an NPDES permit is obtained under the act. The permit specifies the permissible concentration or level of contaminant in a facility's effluent. Although this is a federal regulation, its implementation and enforcement is usually carried out by the states under USEPA authorization. This federal regulation requires acceptance or assumption of primacy by the individual states; otherwise, the program is administered by the regional USEPA office. In states where authorization or primacy has not been given by USEPA, the USEPA regional offices grant the NPDES permits. The CWA provides for USEPA's delegation of many permitting, administrative, and enforcement activities of the law to state governments. In states with the authority to implement CWA programs, USEPA still retains oversight responsibility (Cornwell et al. 2003).

State regulations and guidelines controlling the discharge of residuals may vary throughout the United States. In all states allowing direct discharge, there are limitations on suspended solids concentrations that can be released. Some states do not allow direct discharge of residuals even if residuals pretreatment is performed to remove contaminants.

The CWA gave USEPA the authority to set effluent standards on an industry basis (technically-based) and continued the requirements to set water quality standards for all contaminants in surface waters. Under the CWA, drinking water plants that directly discharge residuals must have an NPDES permit and may only discharge pollutants in conformance with the terms of that permit. Generally, each permit must include technically-based water quality effluent limits if such limits have been developed for each industry. To date, USEPA has not developed categorical standards for WTPs. Therefore, such permit limits are generally based on best professional judgment (BPJ). Under the CWA, arsenic is specifically identified as a toxic pollutant; therefore, a BPJ-based effluent limit must be included as a condition for an NPDES permit for any residuals containing arsenic.

Federal guidelines for controlling drinking water plant effluents were drafted by USEPA in 1994 but never promulgated. The draft guidance specified BPJ control technologies for different types of plants, depending on treatment used. The discharge limits, however, only specified limits on pH and TSS. No limits on the toxic pollutants, such as arsenic, were suggested (Cornwell et al. 2003).

USEPA has established numeric water quality criteria under the authority of the CWA for several parameters, including arsenic (40 CFR 131.36; USEPA 1989). For acute levels of arsenic in fresh waters, a recommended limit of 0.36 mg/L was set, and for chronic levels, a limit of 0.19 mg/L was set. For salt waters, the acute level was set at 0.069 mg/L, and the chronic level was set at 0.036 mg/L. These criteria are used by the regulatory agency (USEPA or state) to determine water quality standards. Generally, states must develop numeric surface water quality standards for the priority pollutants where a discharge or the presence of the pollutant could reasonably be expected to interfere with the designated uses of a receiving water body (Cornwell et al. 2003).

Indirect Discharge to a Sanitary Sewer. In some cases, a drinking water treatment plant (DWTP) may be able to dispose of its residuals to a sanitary sewer. In this case, an NPDES permit would not be required of the drinking water plant. However, if the sewer system is part of a POTW, the drinking water facility must meet the technically based local limits (TBLLs) of the POTW. TBLLs are individually determined for each POTW and take into account background levels of contamination in the municipal wastewater. This requirement comes from the provisions of the CWA pretreatment requirements. USEPA requires that all wastewater plants comply with pretreatment requirements, and this rule clearly specifies that in this case, water plant residuals should be classified as industrial waste. Pretreatment requirements for the residual may be specified, including limits on the amount of arsenic, as arsenic could impact the operation of the POTW. In general, a discharge to a POTW is only acceptable if the arsenic concentration falls within the established TBLL of the Industrial Pretreatment Program. In some cases, the arsenic limit may be based on the limits imposed on the POTW in their land application of wastewater biosolids under 40 CFR 503 (Cornwell et al. 2003).

For wastewater biosolids, 40 CFR 503 specifies allowable limits for arsenic concentrations as a function of disposal methods. POTWs utilizing land application are subject to the land disposal limit, land application ceiling limit, and land application clean sludge limit, which are 73 mg/kg, 75 mg/kg, and 41 mg/kg, respectively. If the arsenic concentration in the biosolids exceeds the clean sludge criterion, land application is limited to 41 kg/ha (36.6 lb/acre). As a result, most TBLLs are based on the clean sludge criteria. For example, the typical POTW removal efficiency for arsenic is approximately 45 percent. Assuming biosolids production is around 1,200 lb/mil gal of water treated, the maximum allowable headworks loading will be around 0.109 lb of arsenic per million gallons of wastewater treated. This equates to a total influent arsenic concentration of ~13 µg/L. As a result, if a water system has a background arsenic concentration near 13 µg/L, it may not be possible to discharge to the sanitary sewer if the TBLL is based on the POTW's biosolids disposal permit (Amy et al. 2000).

Underground Injection. Liquid residuals may be disposed of by means of underground injection. This disposal method would be regulated through the UIC Program of the SDWA (Cornwell et al. 2003). Federal UIC regulations prohibit the subsurface discharge of fluid through a well or hole whose depth is greater than its width without a permit. Underground disposal of liquid residuals from a drinking water facility would be through Class V wells unless the material was designated as hazardous. For hazardous material injection, the designation of Class I well is used. These two types of wells are characterized as follows:

- 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 ¼ mi (400 m) of the well bore.
- Class V includes the injection wells that are not considered in Class I, are not oil and gas wells, and are not involved in mining. Class V injection wells are typically shallow disposal systems for placing nonhazardous fluids underground, and have been regulated under the SDWA since 1983 to protect underground sources of drinking water.

In general, underground injection is prohibited where it would cause any underground source of drinking water to exceed any SDWA-mandated drinking water standards or otherwise impact public health.

Solid Residuals

Landfilling of Nonhazardous Material (RCRA Subtitle D). Under the RCRA Subtitle D regulations (40 CFR Parts 257 and 258 [USEPA 1991a]), criteria have been established for the design and operation of nonhazardous, solid waste landfills (Cornwell et al. 2003). Landfills that receive only drinking water treatment residuals (monofills) are subject to the requirements of

40 CFR Part 257. These same criteria are used to evaluate any landfill that accepts nonhazardous, nonhousehold solid waste and are strictly performance based. Landfills that accept municipal solid waste are subject to the criteria of Part 258, which in addition to the performance-based criteria of Part 257, contains design and operation criteria. RCRA contains requirements that restrict the location, operation, and design of these landfills, and also sets requirements for groundwater monitoring and closure requirements. These two types of landfills—monofills and commercial nonhazardous landfills—are the most commonly used methods of disposal of water plant residuals in the country.

Landfill requirements under both Parts 257 and 258 specify that residuals be in a solid form with no free-flowing liquids. USEPA prohibits the disposal of noncontainerized or bulk liquid wastes in MSWLFs. RCRA requires that the landfill operator or owner determine whether the wastes are "liquid" according to the Paint Filter Liquids Test Method 9095 (in USEPA 1998). In this test, the wastes are placed on a no. 60-mesh filter paper. If any liquid passes through the filter within 5 minutes, the waste is considered a liquid. This requirement would impact the disposal of, for example, conventional treatment plant SFBW.

Sanitary landfills also require that a material pass the TCLP test. The TCLP test (discussed in chapters 2 and 5) includes analyses for several compounds, including arsenic. It is used to determine whether the waste should be characterized as toxic. Toxicity is one of the four characteristics examined to determine whether the waste is hazardous. This toxicity test is the primary area of concern for arsenic-containing wastes. The current TCLP limit for arsenic is based on the former drinking water MCL, and is set at 100 times that level, or 5 mg/L. Although the SDWA-mandated MCL for arsenic has recently been lowered to 10 µg/L, the TCLP levels currently remain at 5 mg/L (Cornwell et al. 2003).

Subtitle D requires management practices at these landfills to protect the environment. One of these mandates is that the landfill leachate may not contaminate an underground water source beyond the solid waste facility boundary. Any release of these compounds to the groundwater that exceeds the permitted level would be considered contamination.

Landfilling of Hazardous Material (RCRA Subtitle C). Under RCRA Subtitle C, the landfilling of materials that are characterized as hazardous is regulated. As previously discussed, four criteria are used to determine if a material is hazardous: toxicity, ignitability, corrosivity, and reactivity. Water treatment residuals that exceed the TCLP limit for arsenic would be deemed hazardous material because of toxicity.

Hazardous waste landfills are generally regulated by the federal government under the authority of the RCRA, but some states also have this authority. Hazardous waste landfills are required to be permitted according to 40 CFR 270 (USEPA 1983), which specifies landfill construction and operation criteria, and are designed to isolate hazardous contaminants from the environment (Cornwell et al. 2003).

Bulk liquid waste disposal is not commonly permitted in landfills. If any water treatment residual contains free liquids as measured by the Paint Filter Liquids Test (SW-845, Method 9095; in USEPA 1998), it cannot be landfilled without being stabilized or treated by another method to remove free liquids. In addition to these requirements, the RCRA requires extensive documentation and manifestation of the material. Transporting of the material once it is deemed to be hazardous is also regulated.

Onsite Lagoons. Lagoons are not considered to be an ultimate disposal method for drinking water treatment residuals, because they are not a permanent disposal and will eventually require cleaning and final disposal of the solids (Cornwell et al. 2003). However, many utilities choose to manage some liquid residuals by storing them in lagoons onsite, allowing for decantation and some evaporation. If the material stored in these lagoons is nonhazardous, then the management of the lagoons would be regulated by the SDWA. The requirements of that act specify that states must establish programs to protect wellhead areas (i.e., areas around a well or a public water supply wellfield) from contaminants that may pose adverse effects to human health.

If the water treatment residual has failed the TCLP test because of arsenic concentrations, then onsite lagoons would be regulated by the RCRA. Under the RCRA, USEPA has established criteria that prohibit practices that contaminate surface or groundwater (40 CFR 257; USEPA 1991a) and has also established comprehensive design and operation standards applicable to surface impoundments.

Beneficial Use. Many drinking water treatment residuals are disposed of through land application. This practice is governed by the RCRA, either under Subtitle C (hazardous material) or Subtitle D (nonhazardous material). In this set of regulations, land application would be considered “reuse.”

If a waste failed the TCLP test for arsenic and was characterized as hazardous, the residual could be land applied under an RCRA 40 CFR 266 exemption (USEPA 1991b). Under this exemption, the material could be land applied, but only after it had undergone a chemical reaction such that it was rendered nonhazardous.

For nonhazardous materials, there are very few federal regulations governing potential land application. The RCRA criteria of Subtitle D, Section 257, establishes some general provisions governing the management of these operations. In general, the requirements address location in a floodplain, protection of endangered species, protection of surface water, land application to food-chain crops, minimization of disease vectors, protection of air quality, and limits on explosive gases. It should be emphasized that drinking water residuals are specifically exempted from the requirements set forth in 40 CFR 503(b) of the CWA (USEPA 1993a). That regulation sets limits on the concentrations of toxic materials, including arsenic, allowed in land applied biosolids, but these limits do not apply to water treatment residuals.

State Regulatory Review

In addition to federal regulations, arsenic-laden residuals are also regulated by state regulations. Many state regulatory programs are generally consistent with the federal minimum requirements. However, differences between state and federal regulatory programs include the surface water quality standards applicable to control the amount of arsenic in direct discharge of liquid effluent, the local limits that specify how much arsenic may be discharged to a sanitary sewer system, the regulation of solid waste landfills, the protection of groundwater resources, and the regulation of land application activities (Cornwell et al. 2003).

Overall, the approach to residuals disposal regulations was similar for Colorado, Indiana, Kansas, Louisiana, Minnesota, Montana, New Hampshire, Texas, Utah, and Wisconsin. For example, all states require an NPDES permit for direct stream discharge and typically limit arsenic to 0.05 mg/L or lower. Indirect discharge (sanitary sewer) arsenic limits are based on state or USEPA industrial pretreatment programs, with local or regional limits typically falling in the 0.05-to-1-mg/L range. Landfilling requires that residuals contain no free liquids and be nonhazardous based on the TCLP test (arsenic <5.0 mg/L). In California, arsenic must be less than 0.5 mg/L, as determined in the CALWET, and total arsenic must be less than

500 mg/kg. Some states have adopted the federal 503 biosolids metals limits for residuals reuse and require residuals to be nonhazardous based on the TCLP test (Cornwell et al. 2003).

Liquid Residual

In the management of liquid residuals through direct discharge to surface waters, the most relevant state standards are those water quality standards for selected states through recommended national numeric water quality criteria for arsenic through NPDES permit.

In California, regional basin plans also impose numeric surface water quality standards. Most surface water quality standards imposed under statewide plans are replaced by the California Toxics Rule. In case of indirect discharge to a sanitary sewer system, the key regulatory standard is the local arsenic discharge limit to the system. In Arizona, the local arsenic discharge limit to a POTW is 0.1 mg/L. In New Mexico, the local arsenic discharge limits range from 0.05 mg/L in Albuquerque to 2.74 mg/L in Santa Fe.

Solid/Sludge Residual

In the case of solid/sludge residual, the typical disposal methods are solid waste landfills, hazardous waste landfills, and land application (reuse). Solid waste landfills must comply with arsenic groundwater monitoring requirements and other design and operating provisions. These requirements are implemented at the state level and, as noted, have been adopted by at least 40 states. Depending on the leaching test level, solid waste is disposed of at various class-level landfills (i.e., Class I, II, or III). Most of the federal regulation for solid and sludge arsenic treatment residuals occurs under RCRA Subtitle D (nonhazardous waste).

When arsenic treatment residuals constitute hazardous waste, residuals must be managed according to RCRA Subtitle C requirements (hazardous waste). For hazardous waste landfills, Arizona, California, Maine, Nebraska, New Mexico, Nevada, and Pennsylvania are some of the states that adopt the federal arsenic standard (i.e., 5.0 mg/L) for the purpose of the toxicity characteristics.

If the extract from a representative sample of the waste (based on use of the TCLP test) exceeds this threshold, the waste is regulated as a hazardous waste. California also developed an additional leaching test known as the CALWET. Based on the CALWET, California defines as hazardous waste those wastes that have an arsenic soluble limit threshold concentration of 5.0 mg/L, and those wastes that have a total arsenic soluble limit threshold concentration of 5.0 mg/L, and those wastes that have a total arsenic threshold limit concentration of 500 mg/kg. The land application of DWTP sludge appears to be regulated primarily on a case-by-case basis under the respective state regulations.

PROCESSES

Waste From Arsenic Treatment Processes

Several drinking water treatment technologies are used for removal of arsenic from water supplies. Conventional coagulation with alum or iron salts followed by filtration removes arsenic (Chen and Snoeyink 1994, Scott et al. 1995, and Sorg 1993). Lime softening and iron removal are conventional treatment processes that can potentially remove arsenic from source waters (McNeill

Table 8.1
Arsenic-containing waste resulting from drinking water treatment

Treatment technology	Form of residual	Type of residual
Ion exchange (regenerable)	Liquid	Spent brine Rinse/backwash waste
	Solid	Spent resin
Membrane (RO and NF)	Liquid	Brine (concentrate)
Media adsorption (single use)	Liquid	Backwash
	Solid	Spent media
Media adsorption (regenerable)	Liquid	Backwash Spent regeneration solution
	Solid	Spent media
Coagulation–microfiltration	Liquid	Backwash Settled sludge
	Solid	Separated from backwash
Conventional treatment	Liquid	Backwash
	Solid	Settled sludge

Source: Adapted from USEPA 2000; Min et al. 2004.

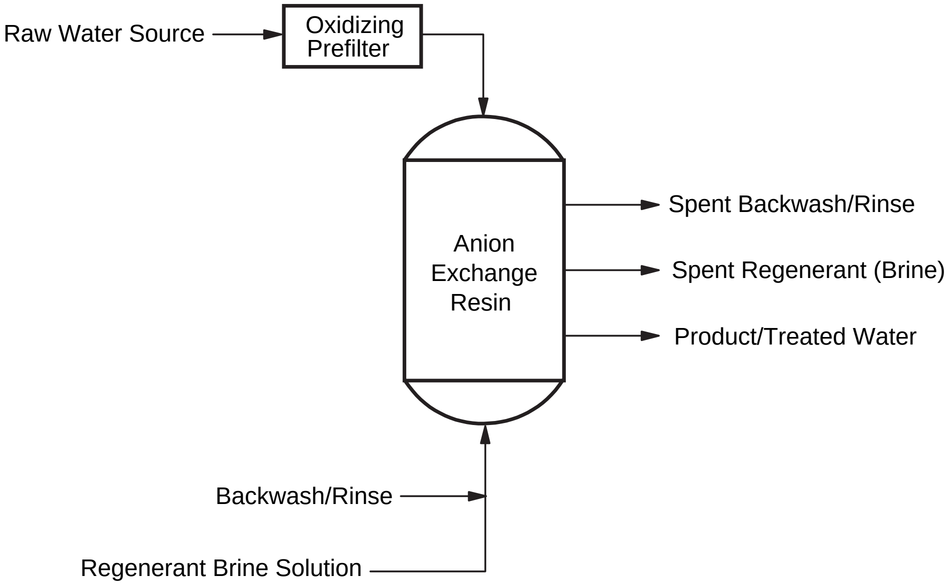
and Edwards 1997a, b). IX and AA adsorption are able to remove arsenic and result in sludge-free operations. Iron-based adsorption media, such as granular ferric hydroxide, demonstrated high arsenic-removal capacities in laboratory and pilot tests (Driehaus, Jekel, and Heldebrandt 1998; Min et al. 2004). Other technologies for arsenic removal include manganese greensand, RO, EDR, NF, and adsorption on activated carbon (USEPA 2000).

Drinking water treatment processes for arsenic removal generate residuals containing arsenic and/or other contaminants removed from raw water. The treatment processes most frequently used for arsenic removal are presented in Table 8.1.

Ion Exchange (Regenerable)

IX is a physical and chemical process where ions holding electrostatically on the surface of a solid phase—a synthetic anion exchange resin—are exchanged for ions of similar charge in a solution (drinking water) (Wang, Chen, and Fields 2000). IX is used in drinking water treatment for softening (i.e., removal of calcium, magnesium, and other cations in exchange for sodium), and nitrate, arsenate, chromate, and selenate removal from municipal water (Clifford 1999). Anion exchange resins come in two classes: strong-base anion (SBA) and weak-base anion (WBA). SBA resins are used for arsenic removal because they tend to be more effective over a larger pH range than WBA resins (USEPA 2000).

To remove arsenic (As) from drinking water, water is passed through one or more chloride-form, strong-base anion exchange resin beds. Arsenate ions are preferentially removed according to the order of preference for exchange. IX does not remove As(III) because As(III) occurs



Source: MacPhee, Charles, and Cornwell 2001.

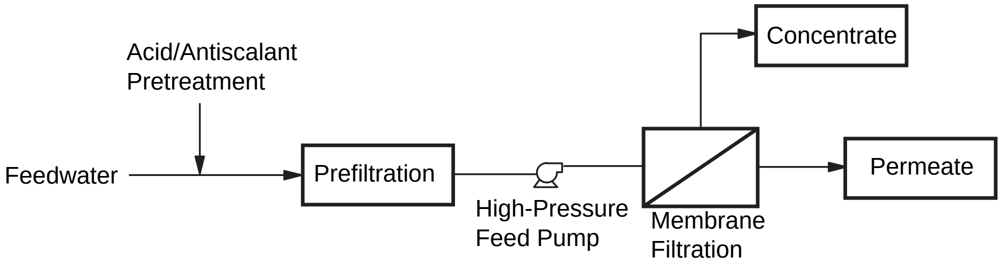
Figure 8.2 Schematic of IX process with regeneration for arsenic removal

predominantly as an uncharged ion (H_3AsO_3) in water with a pH value of <9.0 (Clifford 1999). If As(III) is present, it is necessary to oxidize As(III) to As(V) before removal by IX, as only negatively charged As(V) species are removable by IX (Clifford and Lin 1986). When all available sites on the resin have been exhausted, the bed is regenerated with a brine solution (chloride exchange). The efficiency of the IX process for arsenic removal is strongly affected by competing ions, such as TDS and sulfate (Clifford 1999). Other factors affecting the use of the IX process include EBCT and spent regenerant disposal.

Regeneration steps include resin backwashing, brine regeneration, and a final rinse to remove the brine water. All three regeneration waste streams are usually blended together for final disposal (Figure 8.2).

Membrane Filtration

Membrane processes such as NF and RO can remove contaminants in the ionic and molecular size ranges from drinking water. RO is generally used for salt removal from brackish water or seawater, whereas NF can be used for softening freshwaters and for DBP precursor removal. Both processes can remove trace inorganic contaminants such as dissolved arsenic (USEPA 2000). A simplified schematic of membrane filtration is presented in Figure 8.3 (Waypa, Elimelech, and Hering 1997). The removal efficiency for RO is typically 95 percent for arsenic (SAIC and HDR 1994). As membrane pore size decreases, so does the recovery rate of treated water. These membrane processes generate two streams: the permeate (product water) and the concentrate (waste stream). The concentrate is generally high in TDS (DPRA 1993). Concentrate discharge is subject to disposal requirements under the CWA and RCRA.



Source: MacPhee, Charles, and Cornwell 2001.

Figure 8.3 Schematic of membrane process for arsenic removal

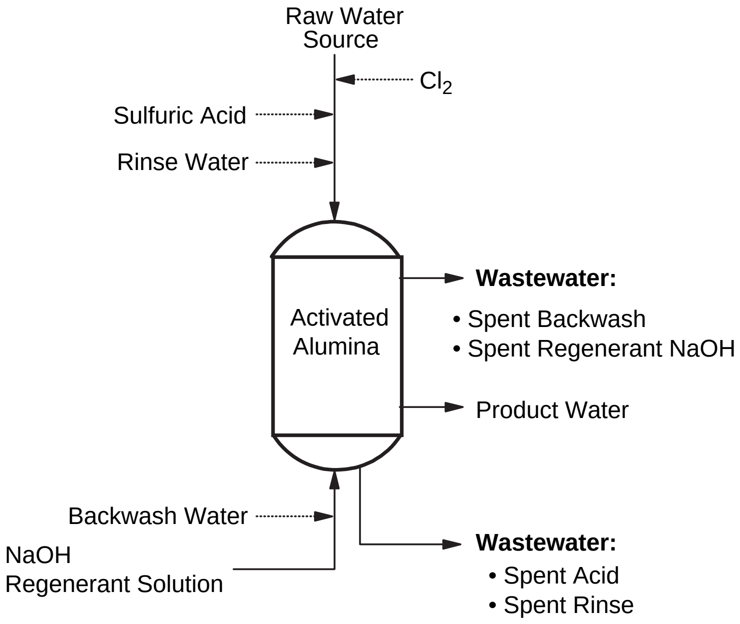
Activated Alumina

AA adsorption is a physicochemical process by which ions in solution (i.e., drinking water) are removed by the available adsorption sites on an oxide surface (Figure 8.4). The AA is a filter medium made by treating aluminum ore so that it becomes porous and highly adsorptive. AA will remove a variety of contaminants, including arsenic. Alumina adsorption is specific for arsenate As(V). In order to achieve effective removal of arsenic from raw water by means of columns, arsenite must be oxidized to arsenate through an oxidative pretreatment ahead of AA columns operated at optimum pH of 6.0. The capacity of alumina is significantly reduced in the presence of sulfate ions but almost not affected by chloride, suggesting that HCl rather than H_2SO_4 would be preferred for pH adjustment. Alumina particle size and EBCT impact arsenic removal. Finer particles of alumina (28 × 48 mesh, 0.6 to 3 mm) have higher arsenic capacity, lower arsenic leakage, and longer run length than larger alumina particles (14 × 28 mesh, 1.18 to 0.6 mm) (Clifford and Lin 1991, Simms and Azizian 1997). To minimize bed size and alumina inventory, an operation is run between a 3- and 6-minute EBCT range (Simms and Azizian 1997).

AA systems can operate using either media replacement after exhaustion or onsite regeneration. Arsenic is difficult to remove from alumina and only 50 to 70 percent of the adsorbed arsenic is eluded during regeneration, as the arsenic capacity of the alumina decreases by 10 to 15 percent on each subsequent run (Clifford and Lin 1986, 1991). During regeneration and acidification of spent alumina, enough aluminum dissolves to make precipitation of $Al(OH)_3(s)$ a feasible treatment step for the removal of arsenic from spent regenerant wastewater. A process schematic for a full-scale AA water treatment system is shown in Figure 8.4.

Liquid and/or solid residual may be produced from an AA process system depending on the type of operation (USEPA 2000). If the system is regenerated, a liquid waste is produced from backwash, caustic regeneration, neutralization, and rinse steps. Sludge may also be generated from regeneration and neutralization streams because some alumina dissolves during the regeneration step and may be precipitated as aluminum hydroxide (AWWA 1990, USEPA 1993b). If an aluminum-based sludge is produced from lowering the pH of the liquid residual, this sludge will contain a high amount of arsenic because of its arsenic adsorption characteristics. This sludge and the remaining liquid fraction of the solution will require disposal. Because both residuals contain arsenic, their disposal may be subject to CWA and RCRA disposal requirements.

Because the AA media will filter out particulate material in the source water, the media bed will occasionally require backwashing. This backwash water will likely contain some arsenic attached to either the particulate material or the very fine AA material that is removed during



Source: MacPhee, Charles, and Cornwell 2001.

Figure 8.4 Schematic of AA adsorption process with regeneration for arsenic removal

backwashing. Consequently, the disposal of the backwash water may also be subject to the disposal requirements under the CWA and RCRA.

When operated on a throwaway basis, the exhausted AA medium will be the principal residual produced. This medium has the potential of being classified as a hazardous waste because of its high arsenic content. A TCLP test is necessary, therefore, to determine its classification and ultimate disposal restrictions. For throwaway systems, the need for backwash to mitigate headloss accumulation from trapped particles generally coincides with media exhaustion, and, as such, backwash streams are rare when the throwaway option is used.

Iron-Based Sorbents

USFilter's (Warrendale, Pa.) Granular Ferric Hydroxide (GFH) media is an adsorption process for arsenic and other heavy metals removal from raw water supply. Raw water is passed through the media to remove contaminants. The process utilizes a ferric-based, nonregenerative media to absorb arsenic onto the media. The adsorption life of the media is determined by raw water pH, arsenic concentration levels, and operating cycles per day. GFH is a nonregenerative medium and is operated as a fixed bed adsorber. Other iron sorbents include WRT's (Arvada, Colo.) Z-33 and Severn Trent's (Birmingham, England) Bayoxide E-33.

Typical installation is in pressure vessels to allow a single pumping stage for the water treatment system. The vessels are arranged in parallel or series arrangement depending on water parameters and required removal concentrations with an EBCT of 5 minutes. If a consistent 90 percent reduction is needed across the system, the series design is used, whereas if the percentage is less than 90, then the parallel design is typically applied. Pressure vessels include an overdrain system for distribution of influent water and collection of backwash waste, and an underdrain system for collection of treated water and distribution of backwash water. GFH media is placed on a gravel support bed of 12-in. (30-cm) depth. The standard hydraulic loading rate is 5 to 8 gpm/ft^2 with a total EBCT of 5 minutes. Backwash supply water is provided by in-service vessels or return from the system. The vessels are backwashed once every 2 to 6 weeks to prevent bed compaction and remove trapped particulate. Once the medium is exhausted, it is removed from the vessels and a new medium is installed.

Iron-based sorbents have greater arsenic adsorption capacity than AA; therefore, backwashing to remove head loss–causing particles is often necessary before the sorbent is exhausted. Hence, two types of residuals are regenerated: spent media and spent backwash (USEPA 2000). The spent media can generally be landfilled as nonhazardous waste according to TCLP or CALWET results, while the spent backwash would require treatment prior to final disposal. The spent backwash usually contains high levels of particles, some of which may be carried over from the sorbent if the backwashing operation is not optimized. Backwashing does not release soluble arsenic, but because raw water is often used as the backwash water, the level of dissolved arsenic in the spent backwash corresponds to the dissolved arsenic level in the raw water (Min et al. 2004).

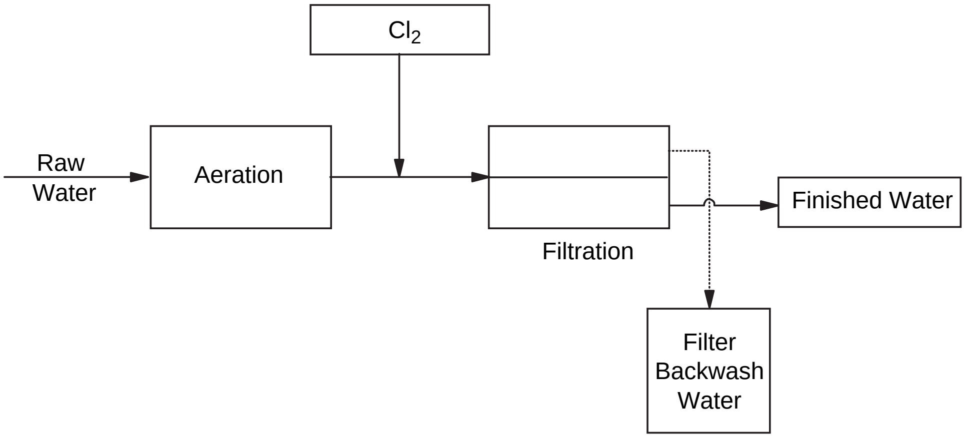
Iron-Manganese Removal Systems

Because arsenic, particularly arsenate, is readily adsorbed onto iron hydroxide, iron and manganese removal processes are effective for arsenic removal (Lauf and Waer 1993). A schematic of an air oxidation-filtration iron and manganese removal water treatment process is presented in Figure 8.5. The oxidation step converts soluble iron (ferrous) into the insoluble form (ferric) that is then removed by the filtration process, usually a granular media. Because air oxidation is not normally effective for oxidizing As(III) to As(V), chlorine may be required on source waters that contain As(III). When the filtration medium reaches its filtering capacity, the medium is backwashed, producing a liquid residual (SFBW) for disposal.

The use of potassium permanganate, in conjunction with a manganese greensand filter, is also a widely used technology for removing iron and manganese from water. A simplified schematic of a greensand filtration process is presented in Figure 8.6. Potassium permanganate can be fed continuously ahead of the filter to oxidize As(III) to As(V) and iron and manganese, which are then adsorbed on greensand. The potassium permanganate also regenerates manganese greensand. Alternatively, the bed of greensand may be activated intermittently with permanganate to form an active coating of manganese dioxide. Because arsenic removal is due to adsorption onto the iron, the capacity for arsenic removal is dependent on the concentration of iron in the source water. Greensand filters require periodic backwashing to remove excess solids. Backwashing is accomplished by reversing the flow of water through the filter bed to flush out particulates. The backwash waste contains elevated concentrations of Fe and Mn as well as other contaminants (MacPhee, Charles, and Cornwell 2001).

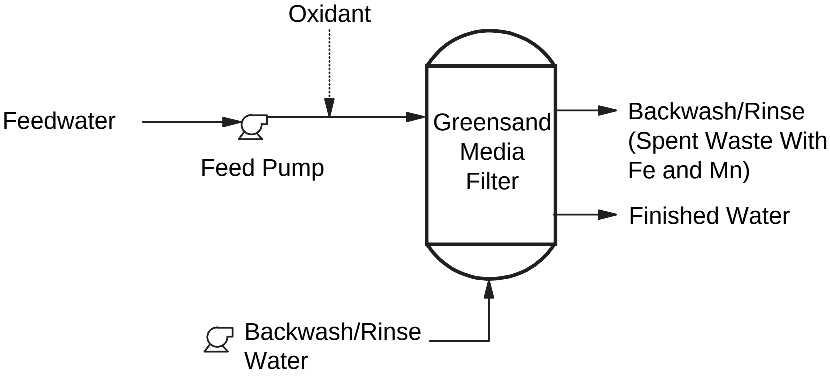
Coagulation–Microfiltration, Conventional Treatment, Lime Softening

Coagulation using alum and Fe(III) salts can be used to remove dissolved arsenic. Removal mechanisms for dissolved inorganics consist of two primary mechanisms: adsorption and occlusion (Benefield and Morgan 1990). During the adsorption process, the dissolved contaminant attaches to the surface of a particle or precipitate. Occlusion occurs when the dissolved contaminant is adsorbed to a particle and then entrapped as the particle continues to



Source: USEPA 2000.

Figure 8.5 Schematic of oxidation-filtration iron and manganese removal process for arsenic removal



Source: MacPhee, Charles, and Cornwell 2001.

Figure 8.6 Schematic of iron and manganese (greensand) filtration process for arsenic removal

agglomerate. Several factors affect the coagulation process, including coagulant dosage, pH, turbidity, NOM, anions and cations in solution, zeta potential, and temperature. Arsenic removal is directly correlated with coagulant dosage (Sorg and Logsdon 1978, Sorg 1993), and turbidity removal is a prerequisite for arsenic removal (Chen and Snoeyink 1994).

Lime softening can be effective for removal of heavy metals, radionuclides, dissolved organics, and viruses (Benefield and Morgan 1990) through adsorption and occlusion with calcium carbonate and magnesium hydroxide. Lime softening is very effective at arsenic removal.

Coagulation–microfiltration (CMF) is the same approach as conventional treatment, except that the sedimentation and filtration steps take place in a single stage: the MF membrane system. CMF, conventional treatment using granular media filtration, and lime-softening systems each generate a liquid residual for disposal. The CMF process would generate a backwash stream,

Table 8.2
Summary of example residuals quantity from arsenic processes

Treatment technology	Volume of residuals produced (gal/MG)	Arsenic concentration in residuals volume (mg/L)	Quantity of solids produced (lb/MG)	Arsenic concentration in solids (mg/kg dry weight)
Conventional coagulation	4,300	9.25	180	1,850
Softening	9,600	4.2	2,000	165
IX	4,000	10	23.4	14,250
AA	4,200	9.52	23.4 (calculated)	14,250 (calculated)
Iron oxide–coated sand	21,000	1.9	23.4 (calculated)	14,250 (calculated)
NF–RO	664,000	0.098	NA*	NA
CMF	52,600	0.76	112.6	2,957

Source: Amy et al. 2000; adapted from MacPhee, Charles, and Cornwell 2001.
NOTES: lb/mil gal × 0.12 = kg/ML.
*NA = not applicable.

whereas conventional softening and lime softening would generate SFBW and settled solids blowdown from clarification.

RESIDUALS QUANTITIES

Table 8.2 provides a summary of example arsenic concentrations in water treatment residuals reported by Amy et al. (2000). The residuals volumes and arsenic concentrations shown in the table for various types of residuals were calculated assuming a raw water arsenic content and arsenic removal for each treatment technology. Arsenic concentrations in residuals volumes generated in processes listed in Table 8.2 ranged from 0.098 mg/L for membrane technologies to approximately 10 mg/L for AA and IX. On a dry-weight basis, theoretical arsenic concentrations ranged from 165 to >14,000 mg/kg.

A liquid and solid residual may be generated from an anion exchange system (USEPA 2000). The liquid residual consists of the backwash water, regenerant solution, and rinse water. These waters constitute 1.5 to 10 percent of the treated water volume depending on the feedwater quality and type of IX unit used (DPRA 1993).

CHARACTERISTICS

Several studies address the levels of arsenic in residuals found at various treatment facilities. Table 8.3 summarizes total arsenic concentrations and TCLP results obtained by MacPhee, Charles, and Cornwell (2001) for arsenic residuals collected at six different utilities. Arsenic levels in the TCLP extract of all but one residuals sample were well below the existing limit of

Table 8.3
Results of TCLP tests from six utilities

Sludge source	Treatment method	Total arsenic (mg/kg dry solid)	TCLP concentration (mg/L)
Utility 1	Lime softening Coagulation	6.9 2.4	0.0039 0.0009
Utility 2	Lime softening	14.8	0.002
Utility 3	Lime softening	24.6	0.028
Utility 4	Alum coagulation	NA[*]	0.0093
Utility 5	Fe/Mn removal	369	0.0444
Utility 6	Iron coagulation	338	1.56

Source: Adapted from MacPhee, Charles, and Cornwell 2001.
*NA = not analyzed.

Table 8.4
Residuals sample characterization

	Untreated residuals characteristics dissolved									
Sample ID	pH	Alkalinity (mg/L as $CaCO_3$)	Total hardness	TDS (mg/L)	Total As (mg/L)	Dissolved As (mg/L)	Total Fe (mg/L)	Total Mn (mg/L)	Conductivity (µS/cm)	Sulfate (mg/L)
AA regenerant	7.1	268	13	10,240	2.63	0.12	0.83	0.09	22,640	16,338
SFBW (A)	7.6	430	365	460	1.41	<0.002	78.5	7.52	900	4.82
SFBW–ACF[*] (B)	8.1	197	400	341	1.74	0.03	45.9	3.75	680	97.3
RO (A)	7.9	2,800	460	14,300	<0.002	<0.002	0.65	0.23	28,500	544
RO (B)	7.3	600	840	11,750	<0.002	<0.002	0.86	1.11	23,800	NA[†]
NF (A)	7.1	325	1,560	1,765	0.013	0.007	2.16	0.14	3,515	1,075
NF (B)	6.6	210	1,750	1,533	0.005	0.009	0.46	0.08	3,080	1,190
IX (B)	9.7	7,000	86	6,240	24.8	24.7	<0.01	<0.005	8,100	910
IX (A)	9.0	950	90	4,100	10.5	10.3	0.49	NA	12,440	NA

Source: Adapted from USEPA 2000.
*ACF = adsorption clarifier flush.
†NA = not analyzed.

5.0 mg/L. It was also found that the arsenic level in the extract was increased by tenfold when the CALWET procedure using citric acid was performed on an iron coagulation sludge.

Table 8.4 provides a summary of the key water quality results obtained in work by USEPA (2000) for residuals samples collected from various treatment techniques. Results showed that about 99 percent of the arsenic in the IX regenerant samples was in the dissolved form. Almost none of the arsenic in the AA regenerant stream and the SFBW samples was dissolved. The concentrate samples collected from the RO and NF plants had very low arsenic concentration.

Table 8.5
Concentration of arsenic in residuals

	Arsenic concentration (mg/L)		
Sample ID	Source water	Residuals stream	Concentration factor
IX (A)	0.039	10.5	270
IX (B)	0.105	24.8	236
SFBW (A)	0.023	1.41	61
SFBW–ACF	0.149	1.74	12
AA regenerant	0.060	2.63	44

Source: Adapted from USEPA 2000.

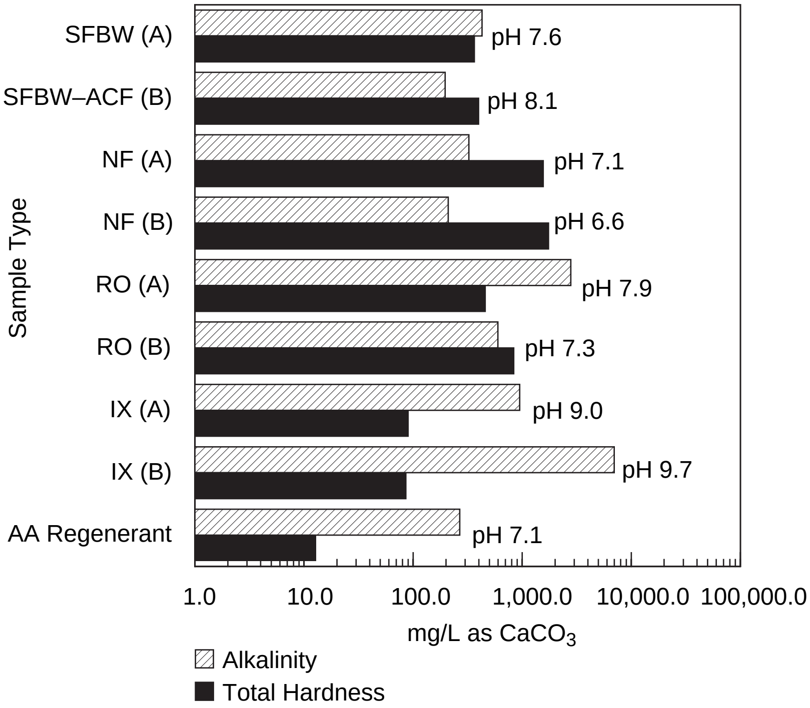
Arsenic levels in the residuals streams were compared to corresponding source water arsenic levels to determine the “concentration factor,” or the degree to which arsenic levels were concentrated in the residuals by the various treatment processes, as shown in Table 8.5.

Arsenic concentrations reached as high as 10 and 25 mg/L in the IX regenerant streams. The SFBW, and SFBW and ACF blend had total arsenic levels of about 1.5 mg/L. Total arsenic in the AA regenerant stream was 2.6 mg/L. Concentration factors for the SFBW and SFBW–ACF samples were 12 and 61, respectively. Arsenic concentration for the AA regenerant stream was comparable, with a concentration factor of 44. The highest concentration of arsenic occurred in the IX waste streams. Arsenic levels were 270 and 236 times greater than the corresponding source water arsenic concentrations for the composite waste streams (brine, backwash, and rinse waters) tested. Arsenic concentration was greater for brine streams than the blends. Clifford, Ghurye, and Tripp (1998) reported also that arsenic was concentrated by a factor of 144 in the brine.

Alkalinity, pH, and total hardness of liquid-fraction residuals samples varied significantly (MacPhee, Charles, and Cornwell 2001). Results are shown in Figure 8.7. The highest alkalinities (as $CaCO_3$) were measured for IX and RO. Alkalinities of the AA regenerant, the SFBW stream and SFBW–ACF blend, and the NF concentrates were comparable, ranging from 200 to 400 mg/L (as $CaCO_3$). The pH ranged from 6.5 to 8.0 for most residuals samples. Along with higher alkalinity, IX regenerant samples exhibited a much higher pH (in the 9.0-to-9.7 range).

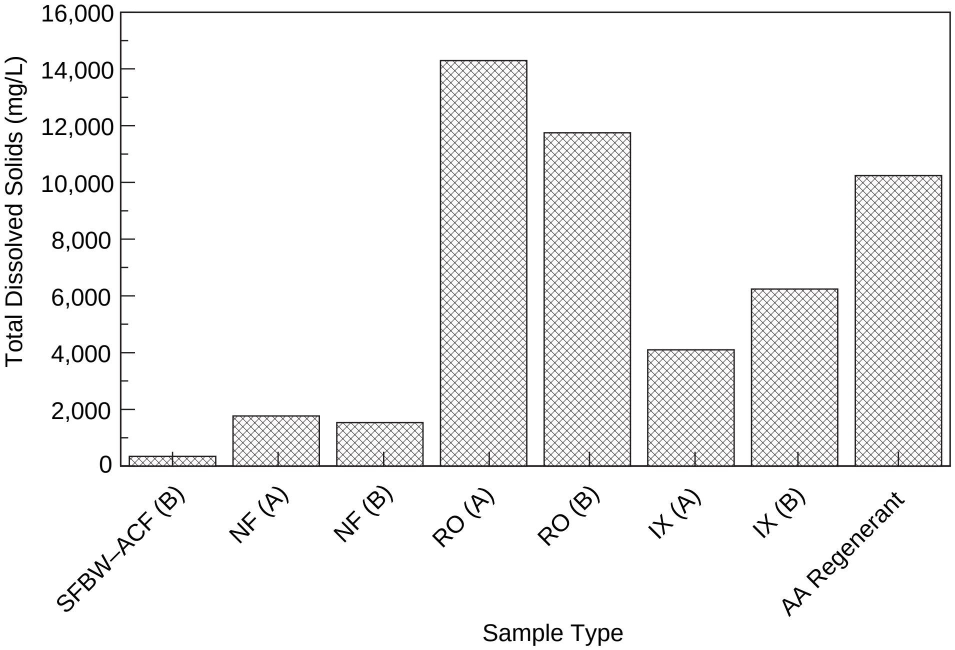
The highest levels of total hardness were measured for the NF and RO processes (in the range of 800 to 1,800 mg/L as $CaCO_3$). The IX and AA regenerant streams exhibited much lower hardness levels than the other residuals streams (<100 mg/L as $CaCO_3$) (Figure 8.7).

TDS levels of the liquid waste streams before treatment are plotted in Figure 8.8 (MacPhee, Charles, and Cornwell 2001). The highest levels of TDS (approximately 10,000 to 15,000 mg/L) were found in the AA regenerant and RO concentrate streams. SFBW TDS levels were at the low end of the spectrum at approximately 300 to 500 mg/L. TDS levels in the NF concentrate and IX regenerant streams fell approximately in the 2,000- to 6,000-mg/L range.



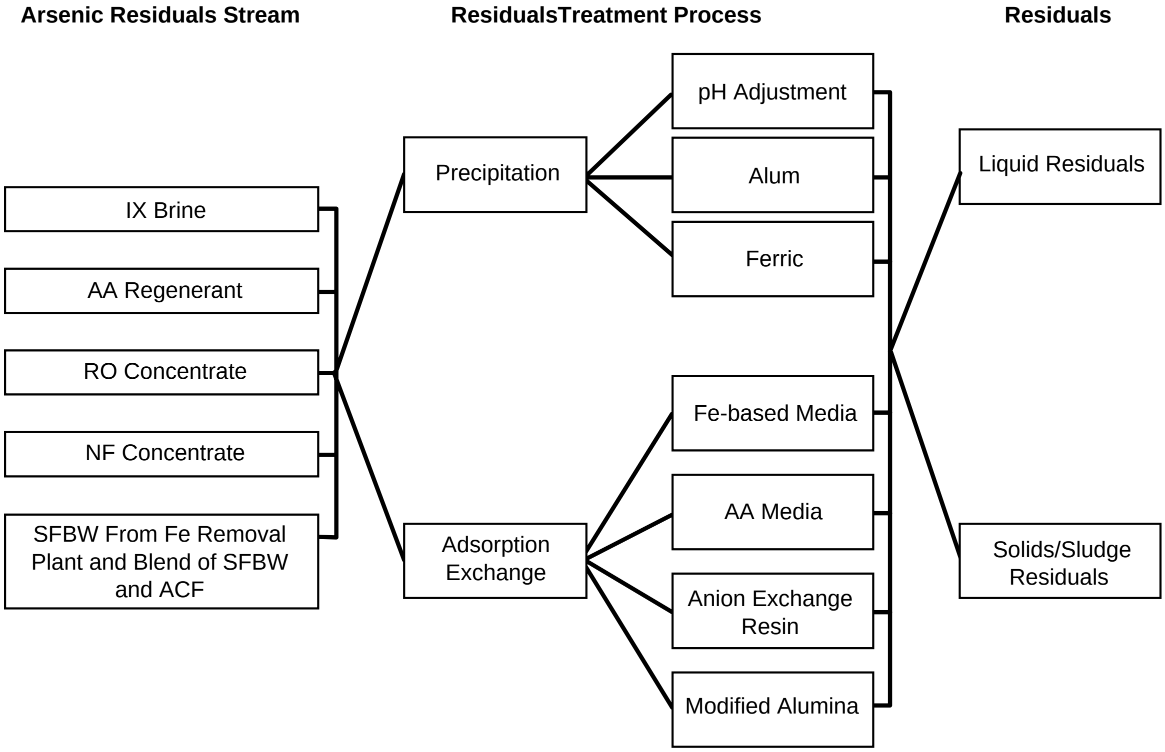
Source: Adapted from MacPhee, Charles, and Cornwell 2001.

Figure 8.7 Alkalinity, total hardness, and pH of untreated liquid arsenic residuals



Source: Adapted from MacPhee, Charles, and Cornwell 2001.

Figure 8.8 TDS concentrations of untreated liquid arsenic residuals



Source: Adapted from USEPA 2000.

Figure 8.9 Arsenic residuals treatment options

TREATMENT OF ARSENIC-CONTAINING LIQUID RESIDUALS

Liquid wastes from arsenic removal processes often have arsenic levels higher than allowed for discharge to streams or land. In this case, the arsenic must be removed prior to discharge. Figure 8.9 illustrates some of the possible treatment processes for brines generated from AA and IX regenerants as well as for concentrates from NF and RO. The treatment processes for the liquid wastes are essentially the same as for treatment of the source water: adsorption, exchange, or precipitation. The following sections discuss results of precipitation and adsorption testing that were prepared by USEPA (2000) for treating these liquid wastes.

A summary of the best treatment technology determined for each residuals sample is presented in Table 8.6. Only three residuals streams (AA regenerant, IX [A], and IX [B]) could not be treated below the common State Pollutant Discharge Elimination System total arsenic concentration limit of 0.05 mg/L. The results show that, overall, the iron-based coagulant and adsorption media resulted in greater arsenic reductions than the aluminum-based coagulant and adsorption media.

Precipitation to Treat Liquid Residuals

Precipitation test results achieved for the various brines are presented in Table 8.7. The table lists the coagulant and coagulation conditions that yielded the greatest reduction in arsenic. Ferric chloride precipitation was effective for removing 88 percent to more than 99 percent of

Table 8.6
Summary of treatment results for removing arsenic from liquid arsenic waste

Sample type	Best treatment conditions determined from testing	Total arsenic remaining (mg/L)
AA regenerant	None	0.154
IX (A)	None	1.28
IX (B)	None	18.7
RO (A)	Ferric chloride precipitation	0.041
RO (B)	Iron media adsorption	0.018
NF (A)	Ferric chloride precipitation, iron-based media, or AA adsorption	0.009, 0.030
NF (B)	Iron media adsorption, ferric chloride precipitation	<0.002, 0.005
SFBW (A) (settled)	Ferric chloride precipitation	0.013
SFBW–ACF (B) (unsettled)	Gravity settling (no chemical addition)	0.043
(settled)	Iron media, IX, or AA adsorption	<0.002

Source: Adapted from MacPhee, Charles, and Cornwell 2001.

arsenic from all residuals samples except IX (B). Total arsenic concentrations remaining in the supernatant ranged from 0.007 to 0.078 mg/L for all samples, except for the AA regenerant and IX regenerant streams. The IX and AA regenerants had much higher initial total arsenic concentrations, so comparable arsenic reductions of 94 and 88 percent for the AA and IX (A) resulted in much higher supernatant arsenic concentrations (0.154 mg/L and 1.28 mg/L). The precipitation test results obtained using alum and ferric chloride are detailed in the following paragraphs for the various treatments (MacPhee, Charles, and Cornwell 2001).

Activated Alumina Brines

Only ferric chloride was used for precipitation testing conducted with the AA regenerant, which contained 113 mg/L Al. With no chemical addition (gravity settling only), 84.5 percent of the total arsenic was removed from the AA regenerant waste, leaving 0.386 mg/L As in the supernatant. Arsenic removal increased to about 94 percent with the addition of 25 to 50 mg/L $FeCl_3$ (Fe:As ratio of 4.4 to 8.8). Corresponding supernatant arsenic levels in those tests were approximately 0.15 mg/L.

Ion-Exchange Brines

For the IX (A) composite sample, a ferric chloride dose of 100 mg/L, equivalent to a molar ratio of Fe:As of 4.4:1, yielded an arsenic removal of approximately 79 percent at ambient pH 7.9 (pH resulting from coagulant addition alone) compared to about 88 percent at reduced pH 6.2.

Table 8.7
Summary of precipitation treatment of liquid arsenic wastes

Residuals stream		Best precipitation conditions						
Sample type	Total arsenic (mg/L)	Coagulant type	Dose (mg/L)	Fe:As* molar ratio	Polymer (mg/L)	Coagulation pH	Supernatant water arsenic (mg/L)	Arsenic removed (%)
AA regenerant	2.63	$FeCl_3$	50	8.8	0	7.0	0.154	94.1
IX (A)								
Composite	10.5	$FeCl_3$	100	4.4	0	6.2	1.28	87.8
Brine	33.2	$FeCl_3$	100	1.4	0	8.8	4.35	86.9
IX (B)	24.8	$FeCl_3$	200	3.7	0.5†	6.2	18.7	29.4
RO (A)	0.546	$FeCl_3$	150	127	2†	6.0	0.041	92.5
RO (B)	0.663	$FeCl_3$	100	70	0	6.2	0.078	88.2
NF (A)	0.523	$FeCl_3$ or Alum	150	133	4†	5.0 7.0	0.060	88.4
NF (B)	0.486	$FeCl_3$	150	143	4†	6.2	0.005	98.9
SFBW(A)								
Composite	1.41	$FeCl_3$	50	16.4	4‡	7.1	0.013	99.1
Settled composite	1.41	None	0	None	0	7.6	0.007	99.5
SFBW–ACF (B)								
Composite	1.74	$FeCl_3$	75	19.9	0	6.2	0.070	96.0
Settled composite	0.043	$FeCl_3$	100	1,075		6.5	0.011	74.4

Source: Adapted from USEPA 2000.

*Based on Fe added as coagulant (does not consider Fe in the untreated wastewater).

†Cationic Percol LT 22S (Allied Colloids).

‡Anionic A3040 LTR (Stockhausen)

Alkalinity was also lowered at the reduced pH condition. Corresponding supernatant arsenic concentrations were 2.36 and 1.28 mg/L. The same ferric chloride dose applied to the brine component of the composite, which contained about three times as much arsenic, achieved 87 percent arsenic removal at ambient pH 8.8 compared to 57 percent removal at pH 6.4. Alum tests yielded much poorer arsenic reductions (11 to 43 percent).

Alum and ferric chloride precipitation tests for the IX (B) composite sample were conducted at ambient pH 9.9 and reduced pH 6.2, with alum and ferric chloride dosages ranging from 50 to 200 mg/L. In ferric chloride precipitation tests, increasing the ferric chloride dosage from 50 mg/L to 200 mg/L increased arsenic removal from 0 to 25 percent without polymer, and to about 30 percent with polymer at reduced pH 6.2. The corresponding improvement in arsenic reduction was less than 10 percent for ferric tests at ambient pH 9.9, and about the same for alum tests conducted at 6.2.

Clifford, Ghurye, and Tripp (1998) addressed removal of arsenic from spent IX brine containing 3,450 µg/L As using ferric hydroxide coagulation followed by filtration through a 0.22-µm filter. Ferric chloride dosages ranging from 1 to 50 mol Fe/mol As in the brine were evaluated and pH was varied from 5.5 to 8.5. At a pH of 5.5, a molar ratio of 20:1 was required to lower the arsenic concentration by 99.5 percent to 20 µg/L. At pH 6.2, a molar ratio of 50:1 was required to attain similar results. Molar ratios of 20:1 and 50:1 are approximately equivalent to $FeCl_3$ dosages of 150 to 375 mg/L.

RO and NF Membrane Concentrates

In tests conducted using RO (A), increasing the ferric chloride dose from 25 to 150 mg/L resulted in a dramatic increase in arsenic removal from <10 percent to >90 percent. Addition of polymer in those tests had little impact on arsenic removal, whereas depressing the coagulation pH from 7.5 to 6.0 yielded a dramatic reduction in arsenic levels remaining from nearly 0.4 mg/L to <0.1 mg/L (equivalent to arsenic removals of 30 and 80 percent). The significant improvement in arsenic removal may be due to the reduction in alkalinity brought about by the reduction in pH, and the associated reduction in carbonate complexing in the highly alkaline concentrate stream. For the dose and pH conditions evaluated, alum yielded no arsenic removal. This result is consistent with results of alum precipitation tests conducted with IX (B). For RO (B), ferric chloride and alum dosages of 50 and 100 mg/L, or Fe:As molar ratios of 35 and 70 for ferric chloride and Al:As molar ratios of 19 and 30 for alum, were evaluated. A ferric chloride dose of 100 mg/L resulted in supernatant arsenic concentrations of 0.078 at pH 6.2 and 0.132 mg/L at pH 7.2. For the alum coagulation conditions tested, the best arsenic reduction attained was about 57 percent.

In ferric chloride precipitation tests conducted using NF (A), lowering the coagulation pH from about 6.5 to 5.0 increased arsenic reduction by 4 to 12 percent, depending on ferric chloride dose. Ferric chloride dose had little impact on arsenic removal at ambient pH 6.5; however, at pH 5.0, arsenic removal increased from 82 percent with 75 mg/L $FeCl_3$ to 98 percent with 200 mg/L. Addition of polymer at pH 5.0 and 150 mg/L $FeCl_3$ increased arsenic removal from 76 to 88 percent (corresponding to supernatant arsenic levels of 0.117 and 0.061 mg/L). The impact of polymer addition was more significant in alum tests. With 200-mg/L alum at pH 6.6, arsenic reductions with and without polymer were 94 and 69 percent, respectively. Increasing alum dose increased arsenic removal from about 60 percent at 75 mg/L to 94 percent at 200 mg/L. Reducing the coagulation pH from about 7.0 to 6.0, however, had little effect on removal of arsenic. Although a marginal improvement in arsenic reduction of 5 percent was observed when ferric chloride dose was increased from 50 mg/L to 150 mg/L, reducing the pH from 6.5 to 5.0 had no impact in tests conducted with NF (B). All ferric chloride precipitation tests reduced total arsenic to <0.05 mg/L. The effect of pH was similar in alum tests; however, the dose effect was much more significant. Arsenic removals of 40 to 50 percent were attained with 50 mg/L alum, and a dose of 150 mg/L reduced arsenic by 93 percent.

As observed for the other types of liquid residuals, on a weight basis, ferric chloride yielded greater reductions in arsenic than equivalent dosages of alum. On the basis of moles of metal applied per mole of arsenic, however, comparable molar ratios yielded similar results using the two coagulants. For example, for NF (A), a molar ratio of Fe:As of 72:1 with polymer reduced arsenic by 86.4 percent to 0.071 mg/L, compared to an arsenic removal of 86.0 percent at an Al:As molar ratio of 78:1.

Iron Removal Plant Residuals

Arsenic removals of 93 percent or greater were achieved in precipitation tests conducted with SFBW (A) using both ferric chloride and alum at dosages of 25 and 50 mg/L. Neither coagulant dose nor coagulation pH impacted arsenic removal significantly. Polymer also had no impact on arsenic removal. Supernatant arsenic levels were reduced to 0.06 mg/L in all ferric chloride tests, and generally <0.05 mg/L. Supernatant arsenic levels were 0.1 mg/L in all alum tests.

In tests conducted with SFBW–ACF (B), increasing the ferric chloride dose from 25 mg/L to 100 mg/L increased arsenic removal from 91 to 96 percent, and lowered the supernatant arsenic concentration from 0.152 mg/L to 0.075 mg/L. Reducing the coagulation pH from about 7.0 to 6.0 had little impact on arsenic removal. For the range of alum conditions tested (two dosages at two pH levels), arsenic removals of 86 to 89 percent were achieved.

The iron concentration of untreated SFBW (A) was 78.5 mg/L. The molar ratio of background iron plus iron applied as coagulant to background arsenic concentration was much higher (5 to 10 times) than the molar ratio of iron applied in the coagulant to untreated arsenic concentration. Similarly for SFBW–ACF (B) in which the background iron level was 45.9 mg/L, molar Fe:As ratios were 2 to 6 times higher when the background iron was included. Background iron concentrations were only considered in residuals collected at iron removal facilities where iron levels in residuals were 45 mg/L or greater.

Adsorption to Treat Liquid Residuals

Arsenic removal from liquid residuals streams through media adsorption was investigated (MacPhee, Charles, and Cornwell 2001). A summary of the best adsorption conditions for each wastewater sample tested, along with the lowest arsenic concentration achieved, is presented in Table 8.8. The table shows that none of the media tested were successful at removing arsenic from either of the IX regenerant wastewaters.

The maximum removal achieved was 64 percent for the IX (A) composite; however, the resulting effluent arsenic concentration was nearly 4 mg/L.

Adsorption worked best for removing arsenic from the two NF concentrates and one of the RO concentrate samples. The iron-based sorbent media and the AA were equally effective for treating the concentrates from NF (A) and NF (B), whereas the iron-based media worked best for removing arsenic from the RO (B) concentrate. For all three of these samples, the arsenic concentration was reduced to <0.05 mg/L. Adsorption yielded the poorest arsenic removal for the IX samples and RO (A), which were the three residuals sample with the highest alkalinity, suggesting that alkalinity was an interference. The adsorption test results to assess the potential of various media to removed arsenic are detailed in the following sections for the various treatments (MacPhee, Charles, and Cornwell 2001).

Ion-Exchange Regenerants

Adsorption tests were conducted using both IX (A) brine and composite regenerant samples. The adsorption media used for testing included an iron-based medium and an AA medium. Two adsorption tests were conducted for the IX (A) composite regenerant sample using iron media at EBCTs of 1.5 and 3 minutes. The iron-based media removed 60 percent of the arsenic from the raw water up to 100 BV for both the 1.5- and 3-minute EBCT tests. The corresponding arsenic concentration after 100 BV was 3.80 mg/L for the 3-minute EBCT test. After 100 BV, the arsenic removal significantly decreased. The effluent arsenic concentration from the 1.5-minute EBCT test was 7.02 mg/L after 240 BV. The IX (A) brine sample was also treated using the AA media (1.5-minute EBCT). The results from these tests demonstrated that neither

Table 8.8
Summary of adsorption treatment of liquid arsenic wastes

Residuals stream		Best precipitation conditions					
Sample ID	Total As (mg/L)	Adsorption media	EBCT (min)	No. of BVs treated	pH	Arsenic* (mg/L)	Arsenic removed (%)
IX (A)	10.5	Iron-based media	3.0	100	9.0	3.81	63.7
IX (B)	24.8	Iron-based media	3.0	120	9.9	22.3	10.0
RO (A)	0.546	Iron-based media	3.0	80	7.5	0.119	77.4
RO (B)	0.663	Iron-based media	3.0	120	7.3	0.018	97.3
NF (A)	0.523	Iron-based media or AA	3.0	120	7.1	0.030	94.0
NF (B)	0.486	Iron-based media or AA	3.0	120	6.6	<0.002	99.8
SFBW (A)	1.41	Iron-based media	4.5	80	7.6	1.06	24.8
SFBW–ACF (B) (settled blend)	0.043	Iron-based media or AA or IX	3.0	120	8.1	<0.002	97.8

Source: Adapted from USEPA 2000.

*Arsenic concentration remaining in column effluent sample collected after the number of BVs listed had passed through the media.

medium was effective for removing arsenic from the IX (A) brine, perhaps because of the very high alkalinity of the sample. The total arsenic concentration remaining after 100 BV with AA adsorption was 11.5 mg/L.

Adsorption tests were conducted using only the IX (B) regenerant composite sample that had an arsenic content of 24.8 mg/L. Two adsorption tests were conducted using the iron-based media and AA media at a 3-minute EBCT. The iron media adsorption test was conducted for a total of 6 hours (120 BV), and samples were collected after each hour of operation. The results indicate that arsenic removal from the composite sample using the iron adsorption media was poor. After only 40 BV, the arsenic removal was less than 35 percent, and 16.7 mg/L of arsenic was measured in the column effluent. Arsenic reduction declined to <10 percent after 120 BV. The AA adsorption test was also conducted for 120 BV, or 6 hours of operation at the 3-minute EBCT. The AA media removed <10 percent of the arsenic concentration after 40 BV, whereas only 3 percent of the arsenic was removed after 120 BV.

RO and NF Concentrates

RO (A) concentrate was treated using both the iron-based media at 1.5- and 3-minute EBCTs and AA media at a 3-minute EBCT. Results indicate that the iron-based media with a 3-minute EBCT provided the greatest arsenic removal from the RO (A) concentrate. In that test, the total arsenic concentration was reduced by 77 percent to 0.119 mg/L up to 80 BV. The corresponding arsenic concentration for the 1.5-minute EBCT test was 0.211 mg/L after 80 BV. By comparison, AA adsorption at a 3-minute EBCT lowered the total arsenic concentration by just 26 percent to 0.389 mg/L after 80 BV of water was passed through the column.

The effectiveness of the iron-based media and AA were also evaluated for removing arsenic from RO (B) concentrate. Adsorption tests were conducted using EBCTs of both 1.5 and 3 minutes for each of the two adsorption media. Results indicate that the iron-based media outperformed the AA media, resulting in arsenic reductions of 95 percent at 120 BV. Arsenic reduction for the 1.5-minute EBCT test decreased to 84 percent after 240 BV when the arsenic level in the column effluent increased to 0.106 mg/L. The AA was also effective for arsenic removal from the RO (B) concentrate. AA adsorption at a 3-minute EBCT reduced the effluent total arsenic concentration by 89 percent to 0.071 mg/L after 120 BV, compared to 56 percent at a 1.5-minute EBCT. For both test media, increasing the EBCT increased arsenic removal from the RO (B) concentrate.

The greatest difference in the quality of the two untreated RO concentrate samples was alkalinity. Whereas the alkalinity of RO (B) was 600 mg/L (as $CaCO_3$), the alkalinity of RO (A) was more than four times as high at 2,800 mg/L. The much poorer arsenic removal attained in adsorption tests with RO (A) may be due to interference from the alkalinity.

A total of six adsorption tests were performed on the concentrate from NF (A): iron-based media (3- and 6-minute EBCTs), IX (3-minute EBCT), AA (3- and 6-minute EBCTs), and modified alumina medium (3-minute EBCT). The spiked total arsenic concentration was 0.486 mg/L prior to treatment. Both the iron media and the AA media tests with 3-minute EBCTs provided >90 percent removal of arsenic up to 120 BV treated, with corresponding effluent arsenic levels of 0.021 mg/L and 0.034 mg/L, respectively. The arsenic removal provided by the AA and iron media were also very similar using a 6-minute EBCT; after 60 BV, the effluent total arsenic concentration was less than 0.007 mg/L for both. After 40 BV passed through the column, the IX resin was exhausted. The modified alumina medium was also ineffective for removing arsenic from the concentrate. This medium only achieved 28 percent removal after 120 BV of sample were treated.

NF (B) concentrate was also treated using the same four test adsorption media and EBCTs as used for the NF (A) concentrate. The NF (B) concentrate had a spiked total arsenic concentration of 0.486 mg/L. The test results showed that both the iron media and AA media were able to remove >99 percent of the arsenic, achieving arsenic levels below the detection limit of 0.002 mg/L, using either a 3- or 6-minute EBCTs. The IX resin and modified alumina media removed <10 percent of the arsenic up to 120 BV of sample treated.

Iron Removal Plant Residuals

SFBW (A) (mixed and unsettled sample) was treated using both the iron-based sorbent (1.5- and 4.5-minute EBCTs) and AA medium (1.5-minute EBCT). Results indicate that neither medium was effective for removing arsenic from SFBW (A). No removal was achieved using the AA medium, and only a limited amount of removal (24 percent after 80 BV) was achieved using the iron medium (4.5-minute EBCT).

Following these tests, the test procedure was modified to include settling prior to adsorption tests for high solids waste streams. The settled SFBW–ACF (B) water arsenic concentration applied to the adsorption column was less than 0.15 mg/L. Ion exchange, iron media, and AA were used to treat the settled SFBW at an EBCT of 3 minutes. The test results show that close to 100 percent of the arsenic remaining was removed by each medium tested. All measured arsenic concentrations were less than the detection limit of 0.002 mg/L, which is well below an instream arsenic limit of 0.05 mg/L that is in place in some states.

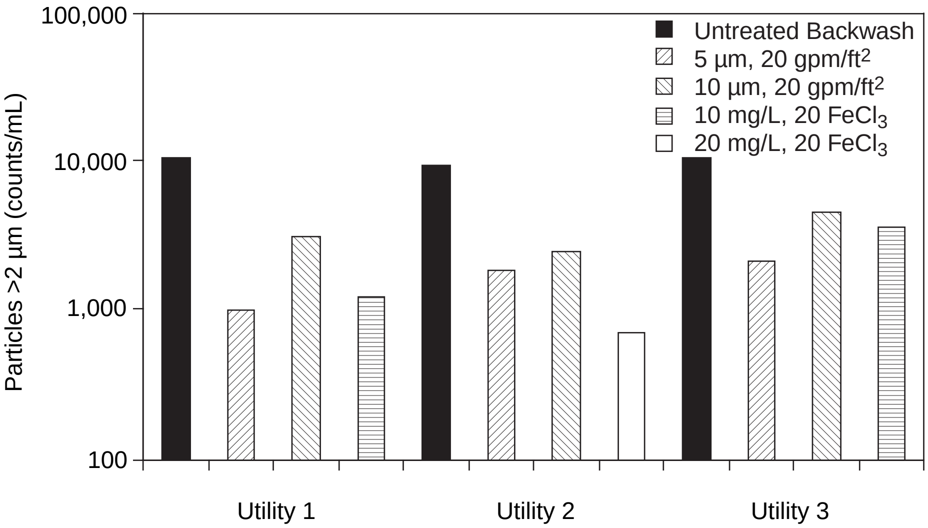
SPENT BACKWASH FROM IRON-BASED SORBENTS

Backwash minimization by introducing a prefiltration step to trap particles, or treatment and recycle of spent backwash, was studied for its ability in reducing the total volume of waste generated during absorber operation (Min et al. 2004). The recycle option would also have the benefit of eliminating NPDES issues associated with direct discharge of untreated spent backwash. The study found that cartridge filters placed before the sorbents (i.e., as prefiltration) did not absorb soluble arsenic, and that particulate arsenic, which was partially retained by cartridge filters, stemmed from arsenic absorption onto iron particles. This means that disposal of cartridge filters in a landfill may not be difficult, depending on the TCLP and/or CALWET results. Concerning spent backwash treatment and recycle, backwash operation anomalies may occur in real-world operations, and the report showed that the spent backwash was highly turbid because of sorbent particles carryover, as evidenced by elevated total iron concentration in the spent backwash. The high turbidity from the spent backwash quickly saturated the cartridge filters employed for treating the wastewater prior to recycle. As such, an alternative backwash treatment technique, such as coagulation–sedimentation, would be better suited (see Figure 8.10). Additionally, it would also contribute to a further decrease in total arsenic concentration in the treated backwash compared to a filtration technique, as shown in Figure 8.11. However, the coagulation–sedimentation technique is more operator-intensive than cartridge or bag filtration, implies high capital costs for utilities without such existing facilities, and requires further treatment of generated residuals, especially if tests indicate arsenic leaching beyond the regulatory limit (Min et al. 2004). If significant amounts of particulate arsenic pass through the spent backwash treatment step, the mixed influent (i.e., the combination of raw water and recycle streams) will be affected. This is important for the reason that each backwash recycle event could significantly increase the total arsenic concentration in the sorbent effluent because of arsenic accumulation in the influent.

STABILIZATION OF SOLID RESIDUALS FROM COAGULATION PROCESSES

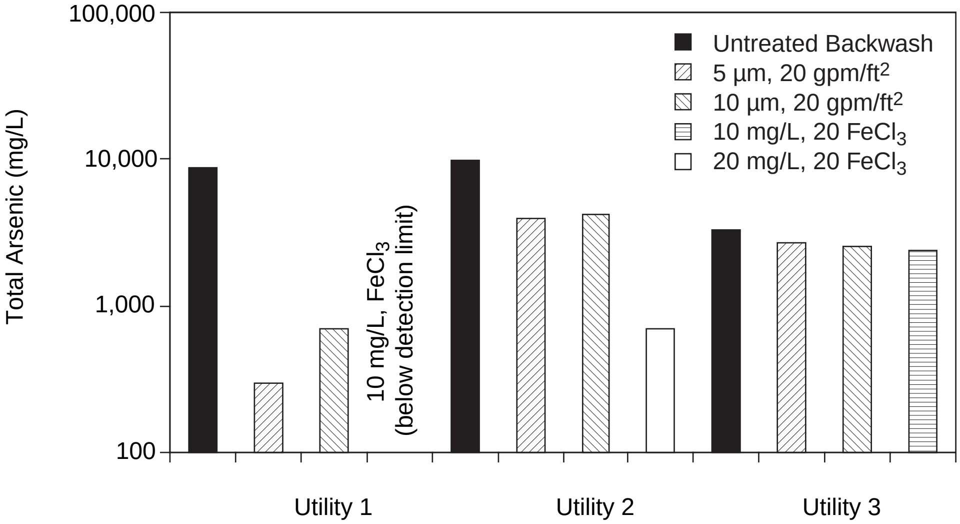
Arsenic removal by chemical precipitation generates arsenic containing sludge. Stabilization methods intend to minimize sludge arsenic leaching. One stabilization method involved the addition of lime (Cornwell et al. 2003) while another included cement with or without a superplasticizer to fix the arsenic. The CALWET (California EPA, N.d.), which is more aggressive than the TCLP, was used to compare the stabilization methods (Min et al. 2004).

As the ratio of lime, cement, or cement with superplasticizer to sludge increases, the release of arsenic decreases (Figure 8.12). For samples with 10 percent fixative (by weight), arsenic releases were above 5 mg/L, ranging from 5.42 to 6.46 mg/L in the case of lime and cement associated with Sikament (Sika Corporation, Lyndhurst, N.J.), respectively. With 20 percent fixative (by weight), all arsenic releases were below 5 mg/L and cement was the most effective of the additives in limiting arsenic release (only 1.31 mg/L). With 50 percent fixative (by weight), lime was the most effective in limiting arsenic release, with 0.18 mg/L. Cement with Sikament increased arsenic release by about 0.7 and 0.75 mg/L compared to cement only, when the fixative quantity represented 10 and 20 percent (by weight) of the stabilized sludge, respectively. When the quantity of fixative increased to 50 percent (by weight) of the stabilized sludge,



Source: Min et al. 2004.

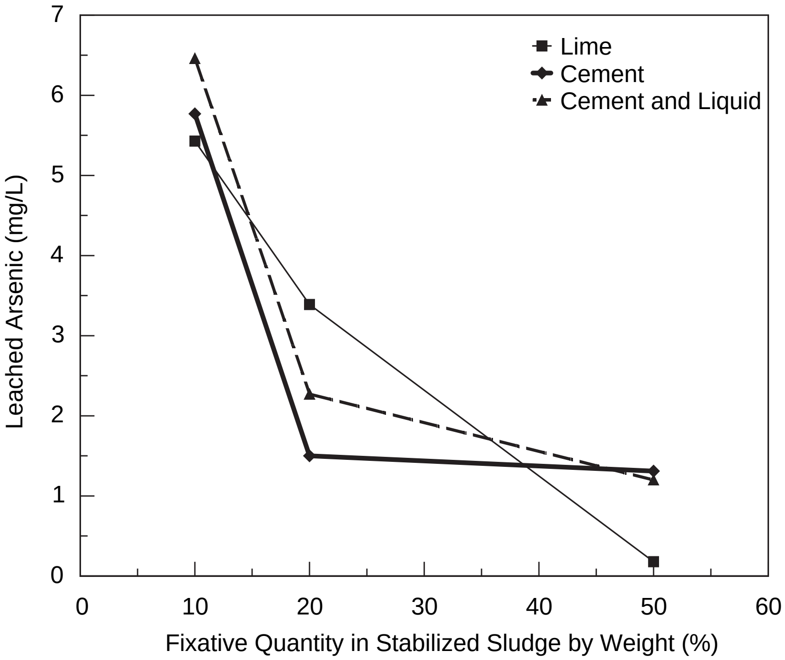
Figure 8.10 Particle removal in arsenic removal spent backwash water by bag filtration and coagulation–sedimentation



Source: Min et al. 2004.

Figure 8.11 Arsenic removal in spent backwash water by bag filtration and coagulation–sedimentation

the quantity of released arsenic was essentially the same for cement with Sikament and for cement only. This is likely because the superplasticizer effect was lessened in the cement–Sikament fixative when the cement quantity was increased to 50 percent (by weight) of the stabilized sludge (the superplasticizer quantity remained constant at 5 mL as cement amounts were increased). Gallup (2002) had obtained a dramatic reduction in arsenic release from an oily sludge exhibiting



Source: Min et al. 2004.

Figure 8.12 Comparison of arsenic release with various sludge fixatives using CALWET

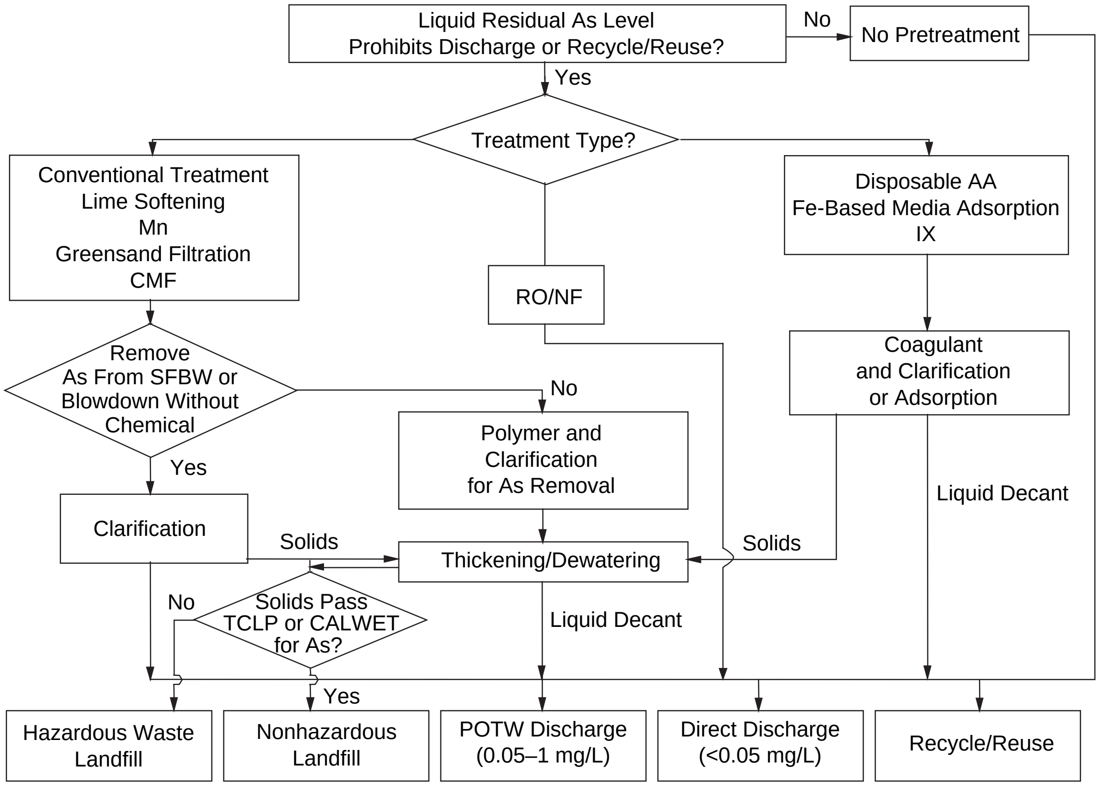
a TCLP of about 1,600 mg/L by using a mortar made of 50 percent residuals, 25 percent cement, 25 percent sand, and 2.5 mL Sikament. The difference in the superplasticizer effectiveness with the Min et al. (2004) study may be due to the type of arsenic leachability test employed (i.e., TCLP versus CALWET), as well as the possibility that no significant further reduction in arsenic release can be achieved when arsenic leaching from the untreated residual is already low.

Based on CALWET conditions, arsenic leaching from a ferric chloride residual conditioned with 10 percent of lime or cement (weight basis) was reduced by 70 percent (from 19.6 mg/L to <6 mg/L). However, the quantity of either fixative would have to be increased to about 15 percent to meet the CALWET regulatory limit of 5 mg/L of leached arsenic (Min et al. 2004).

ULTIMATE DISPOSAL OF ARSENIC-CONTAINING RESIDUALS

The effective removal of arsenic from WTP liquid residuals streams results in a supernatant of effluent streams that may meet regulatory criteria for reuse, stream discharge, or sewer disposal and a sludge or medium waste that contains a concentrated amount of total arsenic. As discussed in the federal regulatory review, final land disposal of solid residuals is dependent on the TCLP arsenic leaching (mg/L) and total arsenic concentration (mg/kg), as well as other TCLP or nonmetal contaminants regulated by USEPA.

If a waste material is found to exceed the TCLP arsenic concentration of 5 mg/L, the liquid or solid material would be considered hazardous and would require disposal in hazardous-waste-handling facilities. If the material is determined to be nonhazardous, the following disposal options may applied for liquid or solid media wastes.



NOTES: Spent media disposed of in nonhazardous waste landfill. AA and iron-based media adsorption backwash waters expected to meet POTW direct discharge or recycle arsenic criteria.

Source: Cornwell et al. 2003.

Figure 8.13 Arsenic residuals handling and disposal decision tree

Liquid/semiliquid wastes:

- Stream discharges (NPDES permit would probably require solids removal)
- Sewer disposal to WWTP
- Land application
- Municipal solid waste landfilling (requires dewatering)

Solid media:

- Land application
- Landfilling
- Regeneration/Reuse

Landfill disposal, sewer disposal, land application, and stream discharge regulations vary from state to state. Some states have adopted the federal regulations for these disposal applications, though others have developed their own specific guidelines for disposal.

Arsenic Residuals Decision Tree

Figure 8.13 illustrates the basic flow pathway for the precipitation process that is used to remove suspended or dissolved arsenic from the liquid residuals and to concentrate it in solid form. Generation of a more concentrated solid will then occur. If a utility elects not to remove arsenic from the liquid residuals, the only other option would be to dewater the residuals in an evaporation lagoon. An evaporation lagoon would only be practical for utilities generating small volumes of liquid residuals because of the large land footprint required for drying. In the schematic, it is assumed that only AA, IX, RO, and NF processes could potentially use the evaporation lagoon process because of the typically low volumes of residuals generated and lower concentrations of suspended solids present.

REFERENCES

Amy, G., M. Edwards, P. Brandhuber, L. McNeill, M. Benjamin, F. Vagliasindi, K. Carlson, and J. Chwirka. 2000. Residuals Generation, Handling and Disposal. In *Arsenic Treatment Options and Residuals Handling Issues*. Denver, Colo.: AwwaRF and AWWA.

AWWA (American Water Works Association). 1990. *Water Quality and Treatment: A Handbook of Community Water Supplies*. 4th ed. New York: McGraw-Hill.

Benefield, L.D., and J.S. Morgan. 1990. Chemical Precipitation. In *Water Quality and Treatment: A Handbook of Community Water Supplies*. 4th ed. New York: McGraw-Hill.

California EPA (Environmental Protection Agency). N.d. Waste Extraction Test Procedure. Hazardous Materials Library, SOP-910M. Berkeley, Calif.

Chen, A.S.C., and V.L. Snoeyink. 1994. Activated Alumina Adsorption of Dissolved Organic Compounds Before and After Ozonation. *Jour. Environmental Science & Technology*, 21(January): 83–90.

Clifford, D., G. Ghurye, and A. Tripp. 1998. Arsenic Ion Exchange Process With Reuse of Spent Brine. In *Proc. of the AWWA Annual Conference*. Denver, Colo.: AWWA.

Clifford, D.A. 1999. Ion Exchange and Inorganic Adsorption. In *Water Quality and Treatment: A Handbook of Community Water Supplies*. 5th ed. New York: McGraw-Hill.

Clifford, D.A., and C.C. Lin. 1986. *Arsenic Removal From Groundwater in Handford, California—A Summary Report*. Houston, Texas: University of Houston.

Clifford, D.A., and C.C. Lin. 1991. *Arsenic (III) and Arsenic (V) Removal From Drinking Water in San Ysidro, New Mexico*. Cincinnati, Ohio: USEPA.

Cornwell, D.A., M.J. MacPhee, R. Mutter, J. Novak, and M. Edwards. 2003. *Disposal of Waste Resulting From Arsenic Removal Processes*. Denver, Colo.: AwwaRF and AWWA.

DPRA, Inc. 1993. *Draft Final Water System Byproducts Treatment and Disposal Cost Document*. Prepared for USEPA, Office of Ground Water and Drinking Water. Washington, D.C.: USEPA.

Driehaus, W., M. Jekel, and U. Heldebrandt. 1998. Granular Ferric Hydroxide: A New Adsorbent for the Removal of Arsenic From Natural Water. *Jour. Water Supply: Research and Technology—Aqua*, 47(1):30–35.

Federal Water Pollution Control Act, as amended by the Clean Water Act of 1977, Public Law 92-500, Oct. 18, 1972. 86 Stat. 816, U.S.C. 1251 *et seq.*

Fields, K.A., A. Chen, and L. Wang. 2000. *Arsenic Removal From Drinking Water by Coagulation/Filtration and Lime Softening Plants*. EPA/600/R-00/063. Cincinnati, Ohio: USEPA.

Gallup, D.L. 2002. UNOCAL. Personal communication.

Lauf, G.F., and M.A. Waer. 1993. Arsenic Removal Using Potassium Permanganate. In *Proc. of the Water Quality Technology Conference*. Denver, Colo.: AWWA.

MacPhee, M.J., G.E. Charles, and D.A. Cornwell. 2001 *Treatment of Arsenic Residuals From Drinking Water Removal Processes*. EPA/600/R-01/033. Cincinnati, Ohio: USEPA.

McNeill, L.S., and M. Edwards. 1997a. Arsenic Removal During Precipitative Softening. *Jour. of Environmental Engineering*, (May): 453–460.

McNeill, L.S., and M. Edwards. 1997b. Predicting As Removal During Metal Hydroxide Precipitation. *Jour. AWWA*, 89(January): 75–86.

Min, J., L. Boulos, J. Brown, D. Cornwell, T. LeGouellec, E. Coppola, S. Baxley, J. Rine, J. Hering, and N. Vural. 2004. *Innovative Treatment Alternatives to Minimize Residuals Containing Nitrate, Perchlorate, and Arsenic*. Denver, Colo.: AwwaRF and AWWA.

SAIC and HDR (Science Applications International Corporation and HDR Engineering, Inc.). 1994. *Summary of Arsenic Treatment Workshop January 18, 1994*. Prepared for USEPA, Office of Ground Water and Drinking Water. Washington, D.C.: USEPA.

Scott, K.N., J.F. Green, H.D. Do, and S.J. McLean. 1995. Arsenic Removal by Coagulation. *Jour. AWWA*, 87(April): 114–126.

Simms, J., and F. Azizian. 1997. Pilot-Plant Trials on Removal of Arsenic From Potable Water Using Activated Alumina. In *Proc. of the Water Quality Technology Conference*. Denver, Colo.: AWWA.

Sorg, T.J. 1993. Removal of Arsenic From Drinking Water by Conventional Treatment Methods. In *Proc. of the Water Quality Technology Conference.* Denver, Colo.: AWWA.

Sorg, T.J., and G.S. Logsdon. 1978. Treatment Technology to Meet the Interim Primary Drinking Water Regulations for Inorganics: Part 2. *Jour. AWWA*, 70(July): 54.

USEPA (U.S. Environmental Protection Agency). 1983. 40 CFR Part 270. The Hazardous Waste Permit Program. Apr. 1, 1983. 48 *Fed. Reg*. 14228. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1989. 40 CFR Part 131.36. Toxic Criteria for Those States Not Complying With Clean Water Act Section 303(c)(2)(B). July 6, 1989. 54 *Fed. Reg*. 28625. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1991a. 40 CFR Parts 257 and 258. Solid Waste Disposal Facility Criteria. 56 *Fed. Reg*. 50978. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1991b. 40 CFR Part 266. Standards for the Management of Specific Hazardous Wastes and Specific Types of Hazardous Waste Management Facilities. 56 *Fed. Reg*. 50978. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1993a. 40 CFR Part 503. Standards for the Use or Disposal of Sewage Sludge, Subpart B. Feb. 19, 1993. 58 *Fed. Reg*. 9387. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1993b. *Treatment and Occurrence—Arsenic in Potable Water Supplies*. Prepared by Malcolm Pirnie, Inc., for the Office of Ground Water and Drinking Water. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1998. *Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, SW-846*. 3rd ed., Update 2B. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 2000. *Regulations on the Disposal of Arsenic Residuals From Drinking Water Treatment Plants*. EPA/600/R-00/25. Washington, D.C.: USEPA.

Wang, L., A. Chen, and K. Fields. 2000. Arsenic Removal From Drinking Water. EPA/600/R00/088. Washington, D.C.: USEPA.

Waypa, J.J., M. Elimelech, and J.G. Hering. 1997. Arsenic Removal by RO and NF Membranes. *Jour. AWWA*, 89(10):102–114.

CHAPTER 9
RADIOACTIVITY IN WATER TREATMENT PLANT RESIDUALS

The principal radioactive elements that are found in WTP residuals include radon (although it has a very short half life), uranium, radium, and lead-210.

The decay series is shown in Table 9.1. From this table, it can be seen that groundwater exposed to rock formations having uranium or radium present will also contain radon. Radon is relatively short-lived, having a half-life of 3.82 days. It has three short-lived progeny: polonium-218 (Po-218), lead-214 (Pb-214) and bismuth-214 (Bi-214). Lead-210 (Pb-210) is also formed and has a long half-life—over 20 years. Uranium, radium, radon, and Pb-210 are referred to as naturally occurring radionuclide materials, or NORM, to distinguish them from man-made radionuclide materials.

REGULATIONS

No federal agency currently has the legislative authority to regulate the disposal of wastes generated by water treatment facilities on the basis of the residuals' naturally occurring radionuclide content. The U.S. Nuclear Regulatory Commission (USNRC) regulates licensed radionuclide materials such as would be generated from a nuclear power plant. The Atomic Energy Act (AEA) of 1954 and its subsequent amendments provided the legislative authority for this agency to do so. The USNRC regulates the handling and disposal of both high-level and low-level radioactive wastes that are generated by various man-made processes. High-level radioactive waste includes such things as spent fuel rods from nuclear power plants. Low-level radioactive waste includes such things as clothing and building materials used in nuclear processes. The regulations governing both high-level and low-level radioactive wastes deal only with man-made radioactivity (Hahn 1988).

The determination of high-level or low-level radioactive wastes under AEA is not generally based on a measurement of radioactivity but on how the material was generated.

No federal agency has the specific legislative authority to regulate the handling and disposal of NORM such as radium and uranium. This type of regulation was left up to the states, and some states do address the disposal of wastes containing certain NORM isotopes.

USEPA regulates the disposal of hazardous solid waste under the RCRA. But, radioactivity in solid wastes is not one of the classifications used by the RCRA to determine whether a particular substance is "hazardous." Activated carbons and sludges from water treatment could not be categorized as hazardous under the RCRA because of radionuclide activity. No other federal law specifically regulates disposal of waste containing elevated levels of NORM.

In June of 1994, the USEPA Office of Drinking Water issued its revised *Suggested Guidelines for Disposal of Drinking Water Treatment Wastes Containing Radioactivity* (USEPA 1994). USEPA had originally issued guidelines in 1990. USEPA presented these guidelines in part because the agency recognized that NORM wastes were not regulated elsewhere. The agency drew from guidelines that had been written previously to address the safe handling and disposal of uranium mill tailings. In these suggested guidelines, the agency specified recommendations for the disposal of water plant wastes. The guidance document also specified that progeny of NORM should be considered in the recommended guidelines. This means that radon, as well as all of its decay products, including Pb-210, would be addressed by these guidelines.

Table 9.1
Abbreviated uranium decay series

Chemical notation	Historical name	Half-life	Radiation emitted upon decay
$^{238}_{92}\mathrm{U}$ ↓	Uranium	4.5×10^9 years	α
$^{234}_{90}\mathrm{Th}$ ↓	Thorium	24.1 days	β, γ
$^{234}_{91}\mathrm{Pa}$ ↓	Protactinium	1.17 minutes	β, γ
$^{234}_{92}\mathrm{U}$ ↓	Uranium	2.47×10^5 years	α, γ
$^{230}_{90}\mathrm{Th}$ ↓	Thorium	8.0×10^4 years	α, γ
$^{226}_{88}\mathrm{Ra}$ ↓	Radium	1,602 years	α, γ
$^{222}_{86}\mathrm{Rn}$ ↓	Radon	3.82 days	α
$^{218}_{84}\mathrm{Po}$ ↓	Polonium	3.05 minutes	α
$^{214}_{82}\mathrm{Pb}$ ↓	Lead	26.8 minutes	β, γ
$^{214}_{83}\mathrm{Bi}$ ↓	Bismuth	19.7 minutes	β, γ
$^{214}_{84}\mathrm{Po}$ ↓	Polonium	164 μsec	α
$^{210}_{82}\mathrm{Pb}$ ↓	Lead	21 years	β, γ
$^{210}_{83}\mathrm{Bi}$ ↓	Bismuth	5.01 days	β
$^{210}_{84}\mathrm{Po}$ ↓	Polonium	138.4 days	α
$^{206}_{82}\mathrm{Pb}$ ↓	Lead	Stable	No emissions

Source: Kinner et al. 1989.

The document states that the purpose of this guidance document is to help guide water treatment facilities toward safe and responsible waste management practices for WTP wastes containing radioactivity at levels in excess of natural background levels.

“This guidance provides assistance where gaps in existing regulations may exist. States presently lacking guidelines may wish to consider these guidelines as a starting point for the development of their own policies regulating the disposal of WTP wastes containing naturally occurring radionuclides” (USEPA 1994).

The USEPA document contains suggested guidelines that at present have no federal regulatory requirements.

The USEPA guidelines specify different disposal recommendations based on the level of naturally occurring radioactivity present in the water plant residuals. The concentration of each radionuclide as well as the combined level of radioactivity in the waste residual are considered.

The recommendations are outlined as follows:

1. Solid waste containing <3 pCi/g (dry weight) of radium and <30 pCi/g (dry) of uranium may be disposed in landfills if it is first dewatered and then mixed with other wastes.
2. Solid wastes containing 3 to 50 pCi/g (dry) of radium or 30 to 75 pCi/g of uranium-238 can also be disposed of in landfills but should be covered to prevent release of radon into the air. Sludges should be dewatered, and disposal in an RCRA hazardous waste facility should be considered.
3. Disposal of solid wastes containing between 50 and 2,000 pCi/g (dry) of radium should be handled on a case-by-case basis. Disposal methods that comply with USEPA's standards for the disposal of uranium mill tailings should be considered, along with disposal at an RCRA hazardous unit or a low-level radioactive waste facility.
4. Solid wastes containing 75 to 750 pCi/g (dry) of uranium can be disposed of under the recommendations listed in the previous three cases, as determined on a case-by-case basis.
5. Wastes with natural radioactivity concentrations >2,000 pCi/g (dry) of radium and >750 pCi/g of uranium should be disposed of only as permitted by state regulations, probably in a low-level waste facility.

In the original 1990 guidelines, Pb-210 was discussed, but in the 1994 guidelines, specific guidance for Pb-210 was not given because Pb-210 does not release radon. Instead, USEPA "advises caution regarding Pb-210 bearing wastes. In cases where Pb-210 tainted wastes may occur, the USEPA advises thorough review of water treatment and waste disposal plans by appropriate State agencies" (USEPA 1994). As noted, these disposal criteria are only guidelines, not regulations.

Most states do not specifically regulate water plant wastes on the basis of elevated levels of radioactivity. A few states have enacted restrictions on the disposal of solid wastes containing specific naturally occurring radionuclides.

The states of Illinois and Wisconsin have developed disposal criteria of WTP wastes containing radium. Other states, including New Hampshire, have disposal criteria for wastes containing high levels of NORM radiation from uranium and radium.

Most states will most likely deal with this situation on an individual basis. For example, Hunton and Williams in a regulatory review for AWWA interviewed California officials and found that the California Water Code 13265(a) (1990), Section 5.A, states that the mere presence of radioactivity does not cause residuals to be deemed "hazardous." Officials then indicated that residuals would be handled on an ad hoc case-by-case basis where a radiological and safety concern was presented. In the absence of specific regulations or guidance, water suppliers would be required to dispose of radioactive residuals in accordance with existing solid waste or, where applicable, hazardous waste requirements (Hunton and Williams 1992).

Table 9.2
Disposal requirements of certain states

<table>
<tr><td colspan="4">New Hampshire</td></tr>
<tr><td></td><td>Material</td><td colspan="2">Low-level waste facility</td></tr>
<tr><td></td><td>Pb-210</td><td colspan="2">Not regulated</td></tr>
<tr><td></td><td>Radium</td><td colspan="2">>0.444 pCi/g</td></tr>
<tr><td></td><td>U-238</td><td colspan="2">>58.4 pCi/g</td></tr>
<tr><td colspan="4">Illinois</td></tr>
<tr><td>Material</td><td>Landfill</td><td>Permitted landfill</td><td>Case by case</td></tr>
<tr><td>Total radiation</td><td><5 pCi/g</td><td>5 to 50 pCi/g</td><td>>50 pCi/g</td></tr>
</table>

Source: State of New Hampshire 1983, Cornwell et al. 1987.

The state of Illinois formalized the regulation of water plants generating radium-bearing sludges through a memorandum of understanding (MOU) between the Illinois EPA and the Illinois Department of Nuclear Safety. The MOU requires that water treatment facilities and landfills receiving the radium-bearing sludges be licensed as “radiation installations.” The MOU also specifies disposal criteria depending on the level of radioactivity due to the radium. “In general, sludge containing less than five picocuries/g (dry weight) may be disposed of in a permitted landfill.” Sludge with radioactivity levels between 5 and 50 pCi/g may also be disposed of in a permitted landfill, but under more stringent conditions. If the radioactivity exceeds 50 pCi/g, the method of disposal must be reviewed and determined in advance by the regulatory agency. In that situation, the basic standard is that there must be “reasonable assurance that the exhalation rate of radon to the atmosphere or into a dwelling will not exceed an average rate of five picocuries per square meter per second and reasonable assurance against accidental intrusion into the sludge in the future” (Hunton and Williams 1992).

New Hampshire currently imposes disposal limitations on water plant residuals having high levels of radioactivity due to some NORMs, such as radium. Table 9.2 summarizes certain states’ criteria.

Wisconsin has set the following criteria for landfilling of residuals containing radium:

- Solid waste containing 2 pCi/g (dry) or less of Ra-226 can be landfilled in approved sanitary landfills.
- Solid waste containing >2 pCi/g but ≤50 pCi/g of Ra-226 can be disposed of in selectively approved sanitary landfills. The waste must be mixed with stabilizing solid waste so that the concentration of Ra-226 averaged over any area of 100 m^2 will not exceed background levels by >5 pCi/g, averaged over any 15-cm-thick soil below the surface.
- Solid waste containing >50 pCi/g requires specific agency review.
- The radium-containing waste should be disposed of in its own trench with a separate liner and leachate collection/treatment system.

Illinois also addressed land application of lime sludge containing radium. Illinois requires that the sludge be mixed with the soil so as to not increase the radium by >0.1 pCi/g. The maximum allowable application rate can be found by the following equation:

$$AR = 1{,}390\ Sd\frac{Ra}{R} \tag{9.1}$$

where AR = maximum allowed lime sludge application (tons/acre, dry weight)
S = specific gravity of soil (for example = 2)
d = depth of sludge/soil mixing (ft)
Ra = allowed radium increases (pCi/g)
R = radium in sludge (pCi/g dry weight)

This is illustrated in the following example for a lime sludge containing 22 pCi/g radium, mixed with 6 in. (15 cm) of soil:

$$AR = 1{,}390(2)(0.5)\frac{0.1}{22} \tag{9.2}$$

$$AR = 6.3 \text{ tons/acre} \tag{9.3}$$

which exceeds the approximate desired loading for effective liming of 3 tons/acre. Therefore, this waste could be land applied without increasing the background radium by >0.1 pCi/g.

The U.S. Department of Transportation (DOT) regulates the shipment of any radioactive waste. DOT is a possible regulatory authority if the waste is shipped offsite for disposal. The waste can be considered radioactive by DOT if (1) a state authority has designated the waste as radioactive, or (2) the radioactivity exceeds DOT-established levels. DOT defines a radioactive waste as a material that has a specific activity of >2,000 pCi/g. Also, if a state designates a waste as radioactive, then DOT regulations apply. In such cases, shipment must be according to 49 CFR Part 172 (USDOT 1976), which requires that the waste be packaged in leak-proof containers with acceptable levels of external radiation and transported in appropriately marked vehicles.

RADIOACTIVE PROPERTIES

The parent elements of radium are generally insoluble in water so that radium is often the first radioactive element that is found in drinking water supplies. Ra-226 is the predominant species and has a half-life of 1,500 years. Ra-226 emits alpha and gamma rays, decaying to Rn-222 (see Table 9.1). Alpha, beta, and gamma rays emitted from various intermediate products are all forms of ionizing radiation. In tissue, ionization can produce harmful cellular changes. Radon, with a half-life of only 3.8 days, is a gas and thus will be released from water or sludge solids containing radium. The radon gas can then be inhaled by humans. The major threat to human health from radium daughters comes from breathing air containing radon and its very short lived daughters, which can accumulate as solids in the lungs. This exposes the lungs and other internal organs to continuous radiation. In addition, the residuals can directly expose humans to gamma radiation from the decay of radium. However, for the most part, the safe handling and disposal of radium-containing wastes involves the prevention of radon exposure.

Measurement of radioactive components is expressed in curies or picocuries (10^{-12} Ci). A curie is the official unit of radioactivity, defined as exactly 3.70×10^{10} disintegrations per second. This decay rate is nearly equivalent to that exhibited by 1 g of radium in equilibrium with its disintegration products. About 0.001 mL Ra/d at standard temperature and pressure is released from 1 g of radium.

Radium itself in the residuals can emit gamma rays when landfilled. However, the major concern is for controlling the release of radon gas. Landfill design for radium residuals should at least keep radon emanation to <2 pCi/m^2•sec, the standard that has been set for uranium mill tailings. Radon release is a function of radium concentration, depth of residuals applied, and depth of cover material. Another important factor is the moisture content of the residuals and cover material. Water has the effect of inhibiting radon flux. Detailed design considerations can be found in the USNRC report on uranium mill tailings (Rogers and Nielson 1981, USNRC 1990). The following equation estimates the radon flux from a landfill:

$$F = RpE\,(LD)^{0.5} \tanh\left((L/D)^{0.5}\, d\right) 10^4 \tag{9.4}$$

where
F = radon flux (pCi/m^2•sec)
R = concentration of Ra-226 in the residuals (pCi/g dry weight)
p = composite density of residuals (g/cm^3)
E = emanation coefficient = 0.2
L = decay constant of radium = 2.1×10^{-6} sec^{-1}
D = bulk diffusion coefficient of radon (cm^2/sec)
= $0.106 \exp(-0.261\, m)$
where m = moisture content (percent)
d = residuals depth (cm)

Recall that for tanh

$$x = \frac{e^x - e^{-x}}{e^x + e^{-x}} \tag{9.5}$$

Take, for example, lime residuals at 40 percent moisture content containing 20 pCi/g dry weight. The density of lime residuals for this example is 1.3 g/cm^3. The radon flux can be calculated for a 6-ft (183-cm) residuals depth as follows:

$$\begin{aligned} D &= 0.106 \exp(-0.261[40]) \\ &= 3.1 \times 10^{-6} \end{aligned}$$

$$\begin{aligned} F &= 20\,(1.3)\,(0.2)\,[(2.1 \times 10^{-6}\,(3.1 \times 10^{-6})]^{0.5} \tanh\,[(2.1 \times 10^{-6}/3.1 \times 10^{-6})^{0.5}\,183]\,10^4 \\ &= 5.2\,(2.55 \times 10^{-6})\,(0.59)\,(10^4) \\ &= 0.08 \text{ pCi/m}^2\text{•sec} \end{aligned}$$

The radium concentration in most lime residuals as produced would be low enough that the radon emanation would be below 2 pCi/m^2•sec. In cases where the radium is concentrated to higher levels, and as the residuals dry, the radon release may exceed acceptable levels. In this case, the waste may need to be disposed of in shallower depths or a sufficient cover material added to reduce the release of radon gas.

An extreme example for a radium-containing lime residual would be the disposal of completely dried residuals (0 percent moisture) at an infinite depth. The following calculation applies to an infinitely deep residual sample. Considering a residuals sample containing 22 pCi/g Ra-226:

$$\begin{aligned} F &= 10^4\, RpE\, (LD)^{0.5} \\ &= 10^4\, (22)\, (1.3)\, (0.2)\, (2.1 \times 10^{-6}\, (1.1 \times 10^{-1}))^{0.5} \\ &= 27.5 \text{ pCi/m}^2\text{•sec} \end{aligned}$$

Therefore, as lime residuals dry more completely, the radon release could exceed a guidance limit of 2 pCi/m^2•sec.

The amount of soil cover required to reduce the release to <2 pCi/m^2•sec can be calculated from the following equation (this equation has been simplified by assuming that the porosity of the sludge and cover material are the same):

$$d_c = \left(\frac{D_c}{L}\right)^{0.5} \left\{ \ln\left(\frac{2F}{F_d}\right) - \ln\left(\left[1 + \left(\frac{D}{D_c}\right)^{0.5}\right] + \left[1 - \left(\frac{D}{D_c}\right)^{0.5}\right]\left(\frac{F_d}{F}\right)^{0.5}\right)\right\} \tag{9.6}$$

where d_c = depth of cover required (cm)

D_c = bulk diffusion coefficient of radon release through cover material = 2.1×10^{-3} cm^2/sec for clay at 15 percent moisture based on Equation 9.6 for D

F = radon release flux from residuals

F_d = desired radon release flux

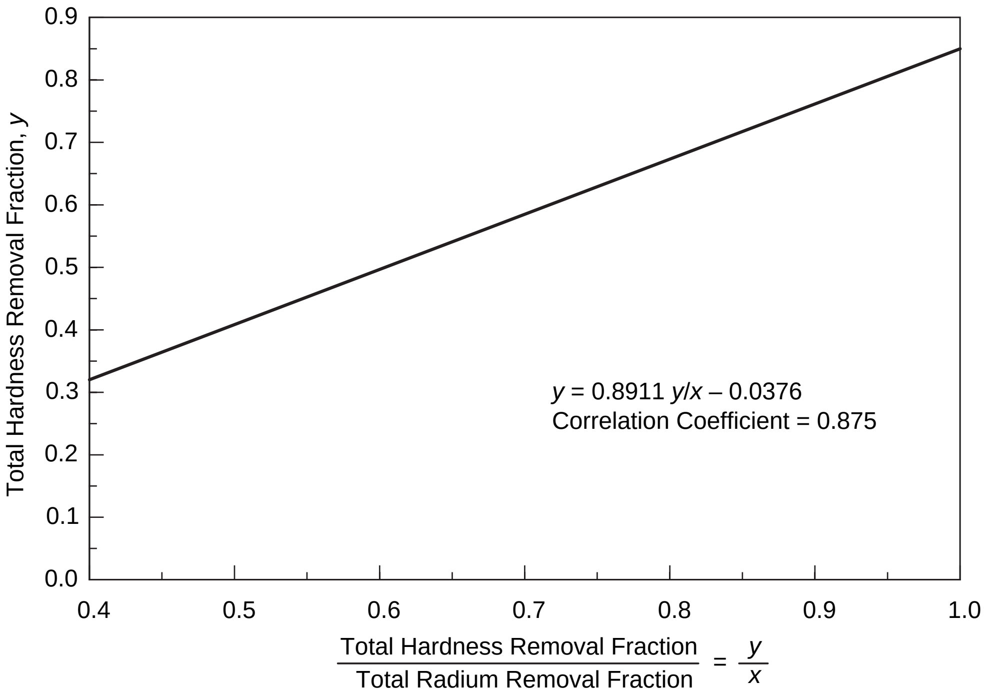
therefore,

$$d_c = \left(\frac{2.1 \times 10^{-3}}{2.1 \times 10^{-6}}\right)^{0.5} \left\{ \ln\left(\frac{2(27.5)}{2}\right) - \ln\left(\left[1 + \left(\frac{6.8 \times 10^{-2}}{2.1 \times 10^{-3}}\right)^{0.5}\right] + \left[1 - \left(\frac{6.8 \times 10^{-2}}{2.1 \times 10^{-3}}\right)^{0.5}\right]\right)\left(\frac{2}{27.5}\right)^{0.5}\right\} \tag{9.7}$$

and calculated so that

$$\begin{aligned} d_c &= 31.6\, [(3.31) - \ln(6.69 - 4.69 \times 10^{-3})] \\ &= 31.6\, (3.31 - 1.90) \\ &= 44.7 \text{ cm} \end{aligned}$$

These equations can then be used for different radium concentrations, depths of application, and types of cover material to determine the proper landfill methods.



Source: Snoeyink et al. 1984b.

Figure 9.1 Proposed correlation between Ra-226 and total hardness removal fraction for lime-softening plants

LIME RESIDUALS

Radium is increasingly removed by lime softening as the percentage of hardness is removed, as shown in Figure 9.1. Data reported by Snoeyink et al. (1984a) for radium concentrations in various lime-softening wastes are shown in Table 9.3. The sludge concentrations of Ra-226 range from 1,000 to 11,000 pCi/L of sludge; Ra-228 varies from 200 to 12,000 pCi/L. Because the radium is associated with the sludge solids, its concentration in the liquid stream is a function of the solids concentration. The concentration per gram of solids is 10 to 20 pCi/g for Ra-226 and 1 to 11 pCi/g for Ra-228. Backwash water concentrations for Ra-226 range from 6 to 50 pCi/L. Again, since this radium is associated with the solids, it could be settled into a sludge waste. Snoeyink and colleagues showed calculations to estimate the radium concentration in the sludge. The removal of radium would first be calculated by using Figure 9.1 or determined by using an actual mass balance within the WTP. The equations presented in chapter 2 for estimating quantities of lime sludge can then be used for total sludge production. The estimated picocuries per gram can then be determined. Table 9.4 shows estimates calculated by Snoeyink et al. (1984a) for four different wastes. The difference between measured and calculated wastes are –37 to +37 percent. Considering the inaccuracy of obtaining a representative clarifier grab sample and the assumption made for calculating theoretical sludge production, the differences are reasonable.

Table 9.3
Summary of radium concentrations in lime-softening residuals and SFBWs

Location	Percent solids	Ra-226 (pCi/L)	Ra-228 (pCi/L)	Ra-226 (pCi/g [dry])	Ra-228 (pCi/g [dry])
West Des Moines, Iowa					
Lagoon sludge	37.6	5,159	596	10.8	1.3
Clarifier sludge*	1.6	<20	<40	<0.2	<0.4
Lagoon sludge	NA†	2,300	NA	NA	NA
Backwash water	NA	6.3	NA	NA	NA
Bushnell, Ill.					
Clarifier sludge	19	4,577	<45	21.6	<0.21
Colchester, Ill.					
Clarifier sludge	12.6	2,038	236	15.0	1.7
Backwash water	0.23	<20	<39	NA	NA
Webster City, Iowa					
Sludge	NA	980	NA	NA	NA
Backwash water	NA	50	NA	NA	NA
Peru, Ill.					
Backwash water	NA	36.9	NA	NA	NA
Elgin, Ill.					
Active lagoon sludge	57.3	9,642	9,939	11.3	11.7
Inactive lagoon sludge	67.1	11,686	12,167	10.9	11.3
Clarifier sludge	10.3	948	873	8.6	8.0
Backwash water	0.051	<20	<40	<0.2	<0.4
Sludge*	NA	6,100	NA	NA	NA
Backwash water	NA	18.3	NA	NA	NA

Source: Snoeyink et al. 1984a.
*Assume sp gr = 1.0.
†NA = not available.

ION EXCHANGE

In addition to removal of hardness, IX can be used for the removal of specific trace inorganic ions such as radium and barium.

Singley et al. (1977) reported radium removal by IX to range from 65 to 85 percent for plants with incomplete regeneration and 95 percent removal or better for plants practicing complete regeneration of their exchange resin. His survey of eight IX plants (four in Iowa, three in Illinois, and one in Florida) indicated radium removals of 84 to 97 percent with raw water Ra-226

Table 9.4
Theoretical sludge quantities and theoretical and measured Ra-226 concentrations

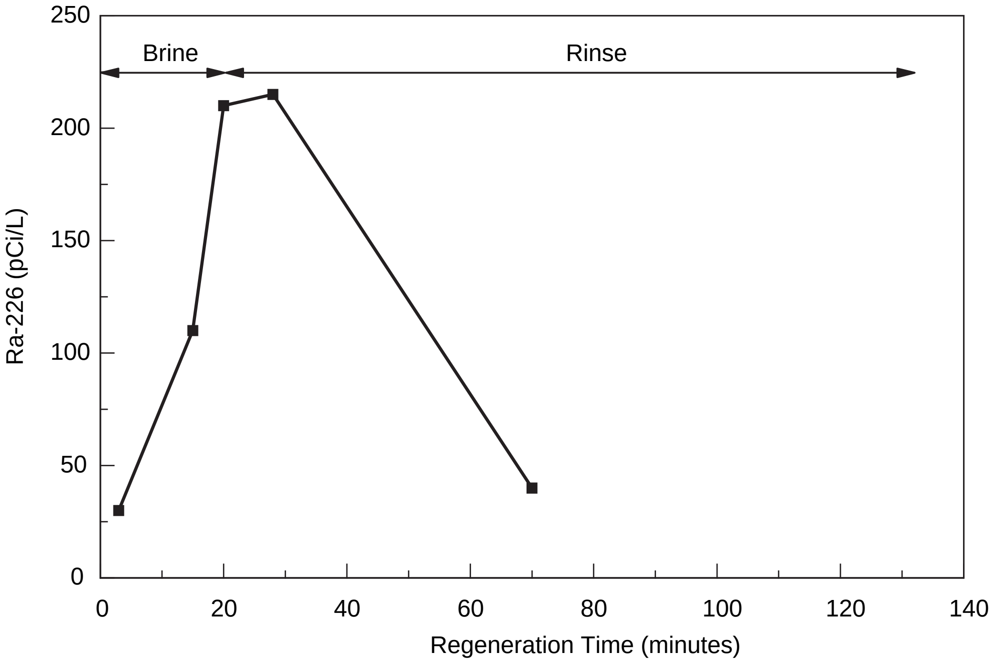
Plant	Theoretical sludge production (kg solids/d)	Theoretical Ra-226 concentration (pCi/g [dry])	Measured Ra-226 concentration (pCi/g [dry])	Percent difference
West Des Moines	3,955	10.0	10.8*	8
Colchester	323	24.0	15†	–37.5
Bushness	712	15.8	21.6†	+36.7
Elgin	8,090	NA‡	0.9†	NA

Source: Calculated from data by Snoeyink et al. 1984a.
*Based on lagoon sludge.
†Based on concentrations measured in the clarifier sludge.
‡NA = not available.

concentrations ranging from 3.3 to 49 pCi/L. Brinck et al. (1978) found radium removals of 81 to 97 percent at seven IX plants in Iowa and Illinois with raw water Ra-226 concentrations ranging from 3.3 to 43 pCi/L. Schliekelman (1976) performed an extensive study of Iowa WTPs and found that IX removed 93 to 96 percent of Ra-226 from raw waters containing 5.7 to 49 pCi/L. The exact concentration of radium in the waste will vary significantly from plant to plant depending upon the regeneration practices. For a given radium concentration, as the hardness decreases, generally the radium concentration in the waste increases. This is because less waste volume per kilogram of hardness removed is produced as the hardness decreases. Snoeyink et al. (1984b) reported data on the release of radium during the regeneration process from one plant, as shown in Figure 9.2. Although during this regeneration the plant was not using sufficient salt for complete regeneration, it does show the profile and indicates that the majority of the radium was released during the rinse phase for this particular operation. Table 9.5 shows data on radium average and peak concentrations in the IX waste from four plants in Iowa.

Clifford (1999) reported that during the normal sodium IX softening process, radium is completely (>95 percent) removed; thus, softening is an effective technique for meeting the radium MCL. Recently completed pilot studies on groundwater containing 18 pCi/L total radium and 275 mg/L total hardness in Lemont, Ill., have resulted in the following conclusions regarding sodium IX softening for radium removed (Subramonian, Clifford, and Vijjeswarapu 1990):

1. On the first exhaustion run to radium breakthrough, hardness breakthrough occurred at 300 BV, although radium did not break through until 1,200 BV.
2. On the second and subsequent exhaustion runs following salt regeneration at 15 lb NaCl/ft^3 (256 kg/m^3) resin, radium broke through simultaneously with hardness at 300 BV. This long, first run to radium breakthrough, followed by shorter subsequent runs, was not an anomaly but was repeated with other strong acid softening resins.
3. When the resin bed was operated in the normal fashion (i.e., exhausted downflow and never run beyond hardness breakthrough), no radium was removed during the first three cocurrent regenerations at 15 lb NaCl/ft^3 (240 kg/m^3) resin. Five exhaustion regeneration cycles were required to each steady state where radium sorption during exhaustion equaled radium desorption during regeneration.



Source: Adapted from Snoeyink et al. 1984b.

Figure 9.2 Variation of Ra-226 concentrations during regeneration (10 to 15 percent brine), Eldon, Iowa

Table 9.5
Ra-226 concentrations in IX treatment plant spent regenerant

Location	Average brine + rinse (pCi/L)	Average brine + rinse + backwash (pCi/L)	Average for peak ¼–⅓ of regeneration cycle (pCi/L)	Peak concentration (pCi/L)	Raw water concentration (pCi/L)
Eldon, Iowa	530	420	2,000	3,500	46
Estherville, Iowa	NA*	52	114	320	5
Grinnell, Iowa	110	NA	260	320	6
Holstein, Iowa	175	NA	576	1,100	13

Source: Schliekelman 1976.
*NA = not available.

4. Radium never broke through before hardness in any of the 80 experimental runs with five different resins. Furthermore, the radium concentration of the effluent never exceeded that of the influent (i.e., chromatographic peaking of radium never occurred).
5. Radium was very difficult to remove from exhausted resins, presumably because it is a very large, poorly hydrated ion that seeks the relatively inaccessible hydrophobic regions of the resin phase. Increasing the regeneration time beyond the usual 15 to 30 minutes helped only slightly to remove the radium from the spent resin.

Calcium-form resins can be used for radium removal (Myers, Snoeyink, and Snyder 1985; Subramonian, Clifford, and Vijjeswarapu 1990) when softening is not necessary. In this process, 1 to 2 M $CaCl_2$ is used as the regenerant at a level of 14.2 lb $CaCl_2/ft^3$ (227 kg/m^3), and counterflow regeneration for an extended time (60 minutes) is the preferred mode of operation. With counterflow regeneration, care must be taken not to mix the exhausted resin bed prior to or during regeneration. In theory, the run time to radium breakthrough is independent of the form (sodium or calcium) of the resin. This theoretical constant run length, irrespective of the resin's initial condition, was verified in the Lemont pilot study for the first exhaustion of a calcium-form resin, which also ran for 2,500 BV before radium reached the MCL (Subramonian, Clifford, and Vijjeswarapu 1990). The lengths of subsequent runs of calcium-regenerated resin were, however, very much a function of the regeneration conditions. The best counterflow $CaCl_2$ regeneration resulted in a typical run length of 500 BV to radium breakthrough with a continuous radium leakage prior to breakthrough of 3 pCi/L when the feed was 18 pCi/L. Cocurrent $CaCl_2$ regeneration resulted in immediate radium leakage (8 to 10 pCi/L) that continually decreased until radium breakthrough. Therefore, cocurrent $CaCl_2$ regeneration is not an acceptable way to operate this radium removal process. When calcium-form resins are utilized for radium removal, only countercurrent regeneration should be employed. Furthermore, extreme care must be taken to keep traces of radium-contaminated resin away from the column exit during exhaustion.

If one is consistent with existing practices regarding the disposal of radium-contaminated brines, disposal into the local sanitary sewer should be allowed (Clifford 1999). For example, in many midwestern communities where water is being softened on both a residential and municipal scale, radium removal is also taking place and the radium-contaminated brines are being disposed of in the usual fashion (i.e., by metering into the local sanitary sewer). To determine the radium concentration in waste-softener brines, a simple calculation can be done. In a hypothetical situation in which the raw water contains as much as 20 pCi/L of radium and the softening run length is 300 BV, the 5 BV of spent regenerant contains an average of 1,200 pCi/L of radium. This is a concentration factor of 60 (300/5) and is typical of softeners in general.

GRANULAR ACTIVATED CARBON

Radioactivity on GAC

GAC removes radon from water by two processes—adsorption and decay. During the initial stage of operation, radon is adsorbed onto the carbon, and this process is the predominant one for the first 2 weeks of operation. After this time, the radon achieves a steady state on the carbon in which the rate of radon adsorption is approximately equal to the radon decay. Because of the constant radon decay, radon can be indefinitely removed by GAC.

The amount of GAC needed for a specific application is determined by the following equation:

$$C_t = C_o e^{-K_{ss} t} \tag{9.8}$$

where C_t and C_o refer to the concentration of radon in the finished and raw water, respectively; e is the exponential; t = contact time of the water with the GAC; and K_{ss} is a rate constant for a specific GAC for a specific contaminant, in this case, radon. The rate constant can range from 2 to nearly 5 depending upon the GAC type (Cornwell, Kinner, and McTigue 1999).

Table 9.6
Measured Pb-210 activity on GAC samples

Influent radon (pCi/L)	GAC volume (m^3)	Effluent (pCi/L)	Water flow (gpd)	Measured Pb-210 activity (pCi/g)
220,000 ± 52,000	0.57 (contactor 1) 0.76 (contactor 2)	4,750 to 68,400	10,000	3,430, 1,940, 1,800, 757, 428, 383
49,500 ± 11,200	0.85	Not reported	15,000	297, 259, 251
35,600 ± 6,700	0.1	79% to 99% removal	270 to 500	235
Up to 1,000,000 pCi/L	0.1	90% to 99% removal	100 to 500	300,000,000

Source: Kinner et al. 1989.
NOTE: gpd × 1.38 = m^3/year.

Radon decays to its progeny, as shown in Table 9.1. These progeny, including Pb-210 and Bi-214, have varying half-lives and emit different types of radioactive particles. The half-lives of the progeny are relatively short until the decay series reaches the product Pb-210. This material has a half-life of 22 years. Most of the Pb-210, therefore, will remain for the life of the GAC contactor. It is this constant generation of the beta-particle-emitting Pb-210 material that has caused concern. If the radioactivity builds to >2,000 pCi/g, then transportation rates for radioactive wastes govern and some states may require disposal of the GAC in a low-level radioactive-waste facility.

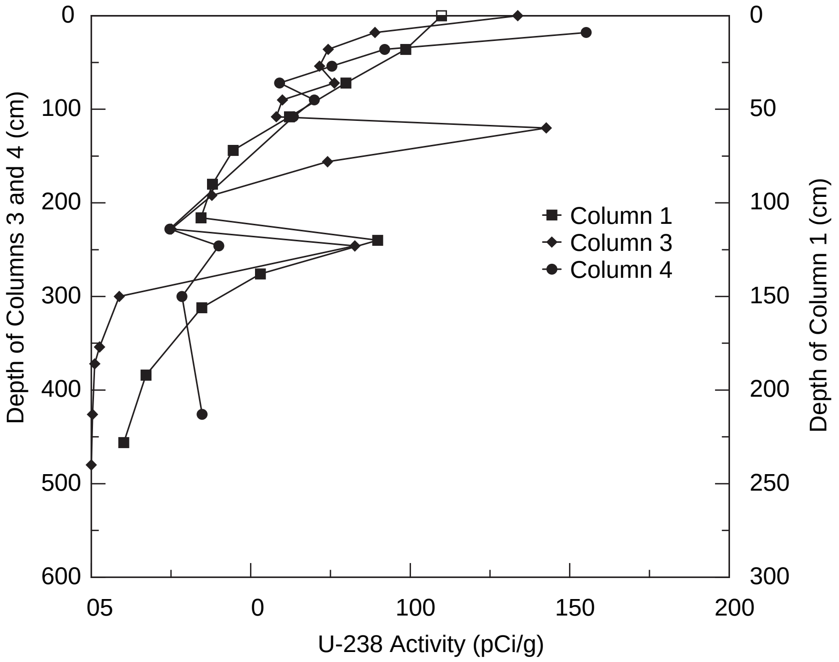
Kinner, Lowry, and others have reported values for several GAC samples that they collected from contactors and measured for Pb-210 activity (Kinner 1990; Kinner, Malley, and Clement 1990; Lowry, Lowry, and Toppan 1988). These data, along with influent radon concentrations, hydraulic loading, and quantity of carbon, are shown in Table 9.6. Some of these samples exceeded the 2,000-pCi/g level. This characterization could result in restrictions being placed on the spent GAC disposal, which could become a burden for the water utility.

In addition to potential health concerns regarding the handling of spent GAC, there have also been worker safety concerns involved with other progeny from radon decay. Some of these progeny particles are gamma radiation emitters. The buildup of these progeny on the carbon bed can present a health risk to employees who work near the GAC units. This issue is separate from that of the buildup of Pb-210 activity.

Cornwell, Kinner, and McTigue (1999) conducted pilot studies to evaluate the buildup of Pb-210, uranium, and radium on GAC when used for radon removal. A New Hampshire water that was tested had a radon concentration of between 10,000 and 20,000 pCi/L and an influent iron concentration of 2 to 4 mg/L. Raw water uranium concentrations were 4.2 to 9.0 pCi/L, and the influent radium varied between 0.03 and 0.87 pCi/L.

Primarily because of the iron buildup on the GAC, the contactors had to be periodically backwashed. The spent backwash water contained approximately 200 pCi U-238 per gram of dry-weight backwash residuals. Uranium concentrations at this level could be a disposal concern.

Radium is, generally, poorly adsorbed by GAC. However, radium can be adsorbed by iron precipitates; therefore, because of the high iron accumulation in the GAC contactors, radium



Source: Cornwell, Kinner, and McTigue 1999.

Figure 9.3 Measured uranium activity on GAC used for radon removal

buildup also occurred in the contactors. The backwash water contained 50 to 60 pCi/L Ra per gram of dry-weight backwash residual. The decay product Pb-210 was also found in the spent backwash, in this case, at levels of 350 pCi/g.

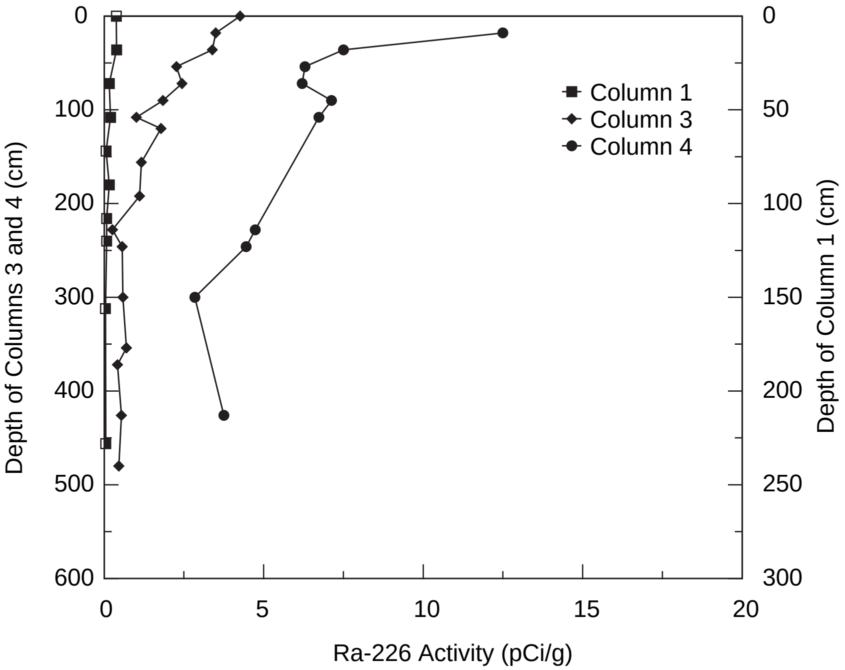
Because of the presence of the radioactivity in the backwash water, it would be justified to remove the iron prior to the GAC contactor in order to eliminate the need to backwash the contactors and thereby eliminate this residual.

The researchers also evaluated the buildup of radioactivity on the GAC itself. Figure 9.3 shows the uranium buildup on the GAC as a function of contactor depth. Near the surface, the uranium level was in the 100-to-150-pCi/g range, although it was well under 50 pCi/g at the contactor lower depths. The researchers presented methods to estimate the composite or average buildup on the GAC in order to preclude reaching uranium regulatory thresholds.

A radium profile is shown in Figure 9.4. The radium levels were generally <5 pCi/g except for column 4, where iron was present.

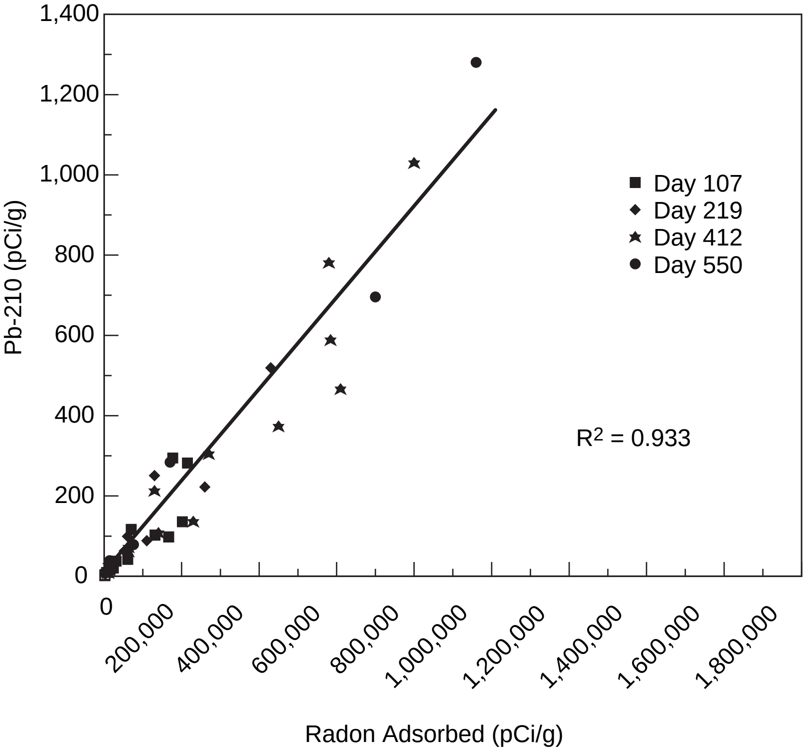
Pb-210 buildup is directly a function of the amount of radon removal. Figure 9.5 shows the direct relationship found by Cornwell, Kinner, and McTigue (1999). Therefore, the longer the GAC stays in service, the higher the Pb-210 levels will be. The increase will not be technically linear because of the decay of Pb-210. However, with a half-life of 22 years, this decay was insignificant in the pilot study.

McTigue and Cornwell (1994) developed a simplified procedure for predicting the buildup of Pb-210 on carbon. In order to determine the average radioactivity per gram of carbon, it was assumed that the Pb-210 was uniformly distributed. This assumption would also be valid if, when the contactor is taken out of service, the carbon is mixed and composited prior to analysis. In the case of the multiple contactors in service, the first contactor may contain higher radioactivity than



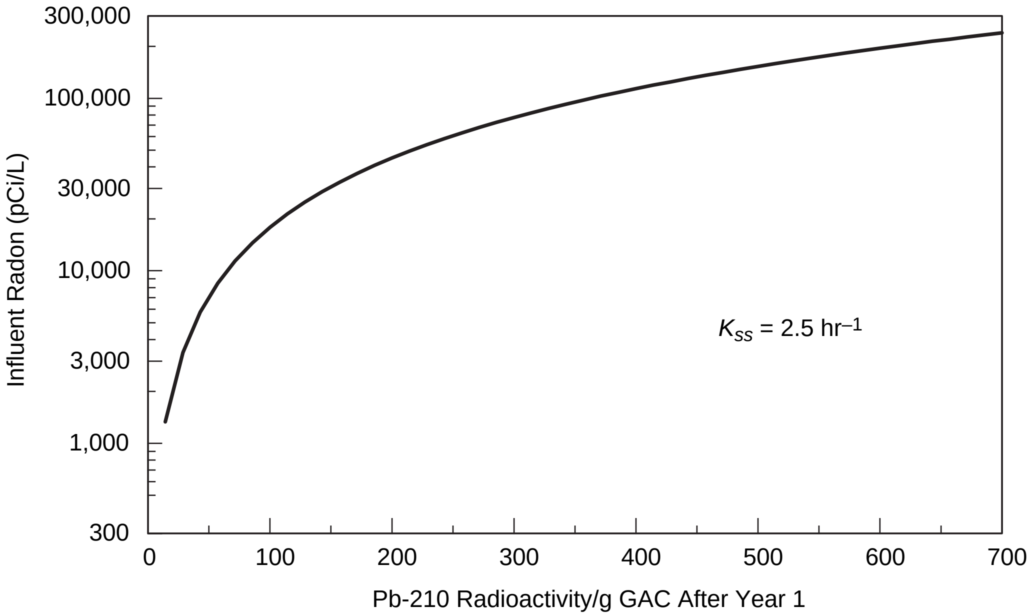
Source: Cornwell, Kinner, and McTigue 1999.

Figure 9.4 Measured radium activity on GAC in columns used for radon removal



Source: Cornwell, Kinner, and McTigue 1999.

Figure 9.5 Linear regression of the radon adsorbed onto carbon and the corresponding Pb-210



Source: McTigue and Cornwell 1994.

Figure 9.6 Estimate of Pb-210 radioactivity/g of GAC after 1 year in service for removing radon

the subsequent contactors; however, that situation is not addressed by the assumption of uniform distribution. Using the uniform distribution assumption, the total radioactivity per gram of carbon after year 1 is found by the following equations:

$$\text{Pb-210 radioactivity/g carbon after year 1} = \frac{-2.57 \times 10^{-1} C_R K_{ss}}{d \ln(C_f / C_o)} \tag{9.9}$$

For a typical bulk density (d) of 30 lb/ft^3,

$$\text{Pb-210 radioactivity/g carbon after year 1} = \frac{-8.6 \times 10^{-3} C_R K_{ss}}{d \ln(C_f / C_o)} \tag{9.10}$$

where
C_R = Radon removal ($C_o - C_f$)
C_o = initial radon concentration
C_f = final radon concentration
K_{ss} = removal rate constant
d = density of GAC

Figure 9.6 shows a plot of the concentrations of Pb-210 radioactivity per gram of carbon for a K_{ss} value of 2.5 hr^{-1} after 1 year of service. In order to determine the levels for years after year 1, the value obtained by Equations 9.9 and 9.10, or Figure 9.6 can be multiplied by the multiplier values given in Table 9.7. This multiplier adjusts for the decay of Pb-210.

Table 9.7
Multipliers used to calculate effect of Pb-210 decay on service life of GAC

Years of service	Multiplier
2	1.97
3	2.91
4	3.82
5	4.70
6	5.55
7	6.38
8	7.18
9	7.96
10	8.71
11	9.44
12	10.20
13	10.80
14	11.50
15	12.10
16	12.80
17	13.40
18	14.00
19	14.50
20	15.10
21	15.60
22	16.10
23	16.60
24	17.10
25	17.60

Source: McTigue and Cornwell 1994.

These methods provide a simplified method for predicting what the Pb-210 radioactivity per gram of carbon will be at various influent radon concentrations. In order to use Equation 9.10, a water system owner only needs to know the influent and desired effluent radon concentration. The K_{ss} of the particular carbon to be used should also be known, but a K_{ss} of 2.5 hr^{-1} can be used as an estimate.

For example, if a water source has an influent concentration of 10,000 pCi/L of radon, and a contactor with sufficient contact time to produce 300 pCi/L in the finished water is used with the

GAC having a K_{ss} of 2.5 hr^{-1}, the Pb-210 radioactivity per gram of carbon after 1 year of service would be 59 pCi/g. This is calculated as follows:

$$\text{Pb-210 radioactivity/g carbon after year 1} = \frac{-8.6 \times 10^{-3} C_R K_{ss}}{\ln(300/C_o)} \tag{9.11}$$

where
C_o = influent radon concentration (10,000 pCi/L)
C_R = radon removed (9,700 pCi/L)
K_{ss} = 2.5 hr^{-1}

For this example, the utility's GAC would have a Pb-210 radioactivity concentration of 59 pCi/g of GAC after 1 year. To determine the Pb-210 radioactivity concentration after a longer time—for example, 20 years—the multiplier in Table 9.7 is used:

$$59 \text{ pCi/g} \times 15.1 = 890 \text{ pCi Pb-210 radioactivity/g of GAC} \tag{9.12}$$

Using this method, it can be seen that if a water system uses the amount of GAC required by the EBCT, it would not exceed the 2,000-pCi/g transportation limit of Pb-210 radioactivity after 1 year of service unless its influent radon were 700,000 pCi/L or higher. It would exceed the 2,000-pCi/g limit after 5 years if the influent radon level were 125,000 pCi/L or higher. GAC left online for 10 years would surpass the 2,000-pCi/g limit if the influent radon level exceeded 55,000 pCi/L. An estimate of the GAC service life required at different influent radon concentrations in order to accumulate >2,000 pCi/g of carbon is shown in Figure 9.7.

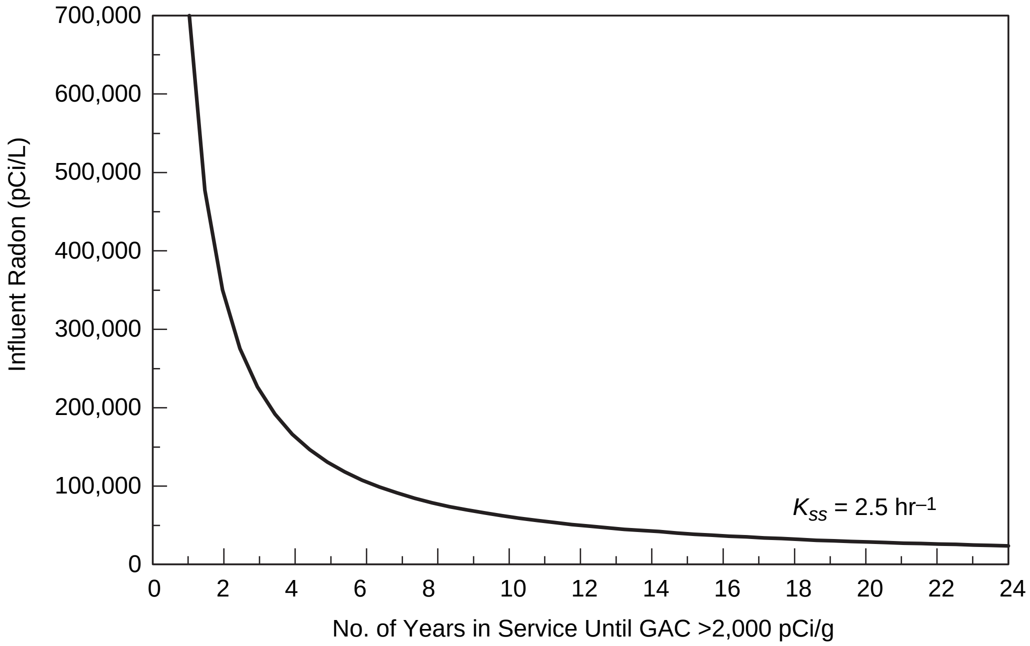
The predictive method described here, and the resulting data and figures shown in this section, assumes that the amount of GAC specified by the EBCT is used in the contactor and that the carbon removes the radon as predicted by the adsorption decay equations. This method can be used by system owners to predict whether GAC in particular situations could accumulate enough radioactivity to be classified as a low-level radioactive waste.

Water system operations using GAC for radon removal or for removal of other contaminants from radon-containing water might be able to minimize the potential for carbon to become characterized as low-level radioactive waste. Because of the problems associated with disposing of this type of waste, system operators should consider operational modifications that can reduce the concentration of radioactivity.

Using the simple predictive method, a system operator can judge if the carbon needed to remove radon will accumulate above a regulatory threshold. It may be economically prudent to remove the carbon for disposal before reaching this limit. Even though the carbon may still have capacity for radon removal, it will probably be more economical to replace the carbon "prematurely" than to dispose of it as a radioactive waste.

Similarly, an operator can use more GAC than specified by the EBCT, thereby lowering the total Pb-210 radioactivity concentration per gram of carbon. Again, this practice would entail an increase in capital costs for the system, but the overall cost of the system may be lower than paying for the disposal of spent GAC in a radioactive waste facility.

A utility may also consider lowering the radon to a level higher than the limit and then blending this water with a radon-free source. If feasible, this option would reduce the amount of radioactivity that would end up on the GAC.



Source: McTigue and Cornwell 1994.

Figure 9.7 Estimated service life of GAC until Pb-210/g radioactivity exceeds 2,000 pCi/g when removing radon

Finally, if a system owner finds through the predictive method described here that the GAC will become significantly contaminated with Pb-210 and low-level radioactive waste categorization cannot be avoided, then alternative radon treatment methods could be evaluated. Realistic cost-of-treatment comparisons among the different technologies available, such as GAC and aeration, must in this case include the cost of low-level radioactive GAC disposal.

Gamma Radiation Emissions

Once radon is captured on the GAC units, it will decay to its progeny. Some of these decay products emit gamma radiation, which can pose a health risk to workers in the area.

For WTP personnel, worker safety exposure limits are discussed in *Radiation Protection Guidance to Federal Agencies for Occupational Exposure* (USEPA 1987). USEPA cites this document in its 1994 suggested guidelines (USEPA 1994). The agency's discussion of its recommendations on worker exposure follows:

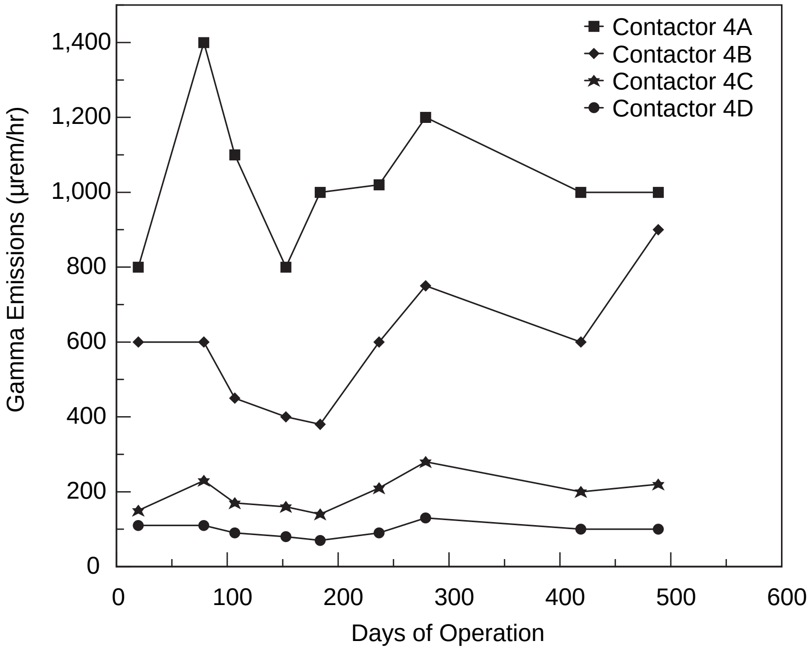
> **Radiation Protection Guidance for Workers in Water Treatment Facilities**
>
> In determining appropriate radiation exposure limits for WTP workers, the relevant guidance is "Radiation Protection Guidance to Federal Agencies for Occupational Exposure", approved by the President in January of 1987. That guidance limits doses to workers to an upper bound of 5,000 mREM/year and further recommends that (a) doses be as low as reasonably achievable (ALARA) below this limit, and (b) doses not approach the limit for substantial portions of a working lifetime. The guidance further recommends that

> *administrative control levels* which take into account ALARA, be developed below the limiting values for specific categories of workers or work situations.
>
> In general, individuals in the United States receive an average radiation dose of approximately 360 mREM/yr from all sources including radon. The International Commission on Radiological Protection (ICRP) recommends that additional man-made exposure of members of the general public due to exposure from all sources excluding medical exposure and background, be limited to 100 mREM/yr, and that no single source provide a large fraction of this limit.
>
> Personnel in water treatment facilities removing naturally occurring radionuclides from drinking water may be exposed to higher-than-background radiation levels. The doses that these workers would receive, however, as discussed in the next section, are normally very much lower than the upper bounds for workers in radiation facilities. As there is no need to allow these workers to receive radiation doses up to the occupational limits, and since radiation doses should be maintained as low as reasonably achievable, it is appropriate and feasible to limit these workers to much less than 5,000 mREM/yr.
>
> Based on the radiation levels that treatment plant workers may be exposed to and taking into account the recommendations of the Radiation Protection Guidance for occupational exposure of workers, it appears a reasonable objective to keep treatment plant workers' doses to well within the levels recommended for the general public (i.e., 100 mREM/yr) with respect to man-made sources of radiation. Based on very limited data available for these exposure conditions, it may be possible to limit occupational dose levels to as low as 25 mREM/yr for external and committed effective dose equivalent. However, *final determinations on whether levels below 100 mREM/yr are reasonably achievable at WTPs must await accumulation of further experience. In the interim, 100 mREM/yr is that recommended maximum administrative control level that should be adopted for use.*

In addition to direct radiation exposures, WTP workers can be exposed indirectly to harmful radon gas through inhalation. Radon screening indoors may be performed easily and inexpensively by use of simple, commercially-available radon detectors. USEPA recommends that homeowners take some action to permanently reduce radon levels in their homes as much as reasonably achievable. Based on the ease of achieving low levels for DWTP workers, it is recommended that action be taken to reduce airborne radon levels in water treatment facilities as much as possible. Because of the different conditions in work places, the times over which radon in air concentration is measured and averaged may be less than 1 year if appropriate (chosen to correspond to normal working hours and conditions) (USEPA 1994).

In addition to annual units, the USNRC regulations require designation of radiation areas when the exposure at 30 cm from a source exceeds 1 mrem/working day. These regulations do not apply to WTPs, but USEPA did suggest clearly marking areas in treatment facilities where unusually high levels of radiation occur (USEPA 1994).

Cornwell, Kinner, and McTigue (1999) measured gamma emissions from pilot-plant columns. Gamma readings obtained outside the pump house at a distance of 20 ft (6 m) were very low (about 15 μrem/hr). Although comparisons of gamma emissions could only be made qualitatively because of the proximity of the contactors, the patterns were similar to those observed in previous studies (Lowry and Brandow 1985; Kinner et al. 1989; Kinner, Malley, and Clement 1990; Rydell, Keene, and Lowry 1989). The column that removed the most radon (contactor 4A)



Source: Cornwell, Kinner, and McTigue 1999.

Figure 9.8 Gamma and beta emissions at GAC contactor surface used for radon removal

exhibited the highest readings (800 to 1,300 µrem/hr) (Figure 9.8). There was a decreasing trend in gamma emissions through each column. These results are consistent with those reported by Kinner et al. (1989) and support the use of the first-order sorption/decay model.

Predictions of gamma emissions were made using an empirically derived equation developed by Lowry and Brandow (1985) and were found to be very similar to the actual readings:

$$\gamma_{\mathrm{MAX}} = \frac{Rn_{inf}}{17.8\frac{\mu\mathrm{rem/hr}}{\mathrm{pCi/L}}} \tag{9.13}$$

where γ_{MAX} = maximum gamma emission (µrem/hr)
Rn_{inf} = influent radioactivity (pCi/L)

Based on an average radon concentration of 17,110 ± 4,580 pCi/L, Equation 9.13 yields 960 ± 260 µrem/hr.

Maximum gamma emissions of about 1,000 µrem/hr would translate into a 2,080-mrem/year exposure (assuming an 8-hr/d, 5-d/week, 52-week/year exposure). This exceeds the <100-mrem/year exposure recommended for the general public but does not exceed the allowable occupational exposure of <5,000 mrem/year. It also exceeds the suggested limit of 100 mrem/year for water treatment personnel (USEPA 1994). However, the 2,080-mrem/year estimate assumes that the individual is ≤1 in. (2.54 cm) away from the top surface of the GAC bed continuously during working hours, which is a very unlikely scenario. Gamma emissions exposure is also very dependent on the distance from the source and geometry of the source. This estimate also assumes that workers would be in a pump house for entire work shifts all year, which is unlikely. Hence, extrapolation from the New Hampshire pilot-plant data to other sites would likely overestimate worker exposure.

Dosimeter badges worn by researchers during all activities at the pilot plant (about 10 to 15 hr/month) detected emissions levels below the monthly measurable doses of 10 mrem for gamma emissions and X rays, and 40 mrem for beta emissions.

GAC contactors can be designed to prevent gamma rays from passing through their walls by using either lead liners or water jackets. It is also possible to limit unnecessary body exposure by physical barriers, such as caging, around contacts at 3-ft (1-m) distances. Access to the contactors would still be possible, but casual contact would be prevented.

Caution should be advised when the GAC itself is to be handled—for example, during GAC replacement operations. In these situations, strict safety measures should be followed. USEPA (1990) makes the following recommendations:

- General radiation levels in areas of WTPs removing radionuclides from drinking water should be monitored at least yearly, using gamma survey instruments. Radiation levels in the vicinity of components concentrating radioactive materials should be monitored at least quarterly.
- Radiation exposure to personnel working in a drinking water treatment facility should not exceed 25 mrem/year and should be kept as far below this level as reasonably achievable.
- If areas in a treatment plant are identified where an individual working could receive a short-term exposure that would be a significant fraction of the previously mentioned limit, such as 1 mrem/d, those locations should be boldly marked “Caution—Radiation” and restricted to specified personnel.
- Persons working in areas marked “Caution—Radiation” should have appropriate radiation protection training, and their radiation exposure should be monitored through area or personnel monitoring, as appropriate.
- Any utility using GAC contactors on source waters containing radon should be aware of both the USEPA and USNRC guidelines and take appropriate measures to protect workers.

REFERENCES

Brinck, W.L., R.J. Schliekelman, D.L. Bennett, C.R. Bell, and I.M. Markwood. 1978. Radium Removal Efficiencies in Water Treatment Processes. *Jour. AWWA*, 70(1):31.

Clifford, D.A. 1999. Ion Exchange and Inorganic Adsorption. In *Water Quality and Treatment: A Handbook of Community Water Supplies*. 5th ed. New York: McGraw-Hill.

Cornwell, D.A., M. Bishop, R. Gould, and C. Vandermeyden. 1987. *Handbook of Practice: Water Treatment Plant Waste Management.* Denver, Colo.: AwwaRF and AWWA.

Cornwell, D.A., N. Kinner, and N.E. McTigue. 1999. *Assessment of GAC Adsorption for Radon Removed*. Denver, Colo.: AwwaRF and AWWA.

Hahn, N.A. 1988. Disposal of Radium Removed From Drinking Water. *Jour. AWWA*, 80(7):71–78.

Hunton and Williams. 1992. *Residuals Regulatory Review: Residuals Management TAW.* Denver, Colo.: AWWA.

Kinner, N.E. 1990. *Radon Removal Using Point-of-Entry Water Treatment Techniques*. Report Prepared for USEPA, Office of Research and Development, Risk Reduction Engineering Lab. EPA/600/2-90/047. Cincinnati, Ohio: USEPA.

Kinner, N.E., J.P. Malley Jr., and J.A. Clement. 1990. *Radon Removal Using Point-of-Entry Water Treatment Techniques*. EPA 600/D-89/249. Cincinnati, Ohio: USEPA.

Kinner, N.E., J.P. Malley Jr., J.A. Clement, P.A. Quern, and G.S. Schell. 1989. *Radon Removal Techniques for Small Community Public Water Supplies*. EPA/600/2-90/047. Cincinnati, Ohio: USEPA.

Lowry, J.D., and J.E. Brandow. 1985. Removal of Radon From Water Supplies. Environmental Engineering. *Jour. ASCE*, 111:511–527.

Lowry, J.D., Lowry, S.B., and Toppan, W.C. 1988. New Developments and Considerations for Radon Removal From Water Supplies. In *Proc. of the USEPA Symposium on Radon and Radon Technology*. Washington, D.C.: U.S. Government Printing Office.

McTigue, N.E., and D. Cornwell. 1994. *The Hazardous Potential of Activated Carbons Used in Water Treatment*. Denver, Colo.: AwwaRF and AWWA.

Myers, A.G., V.L. Snoeyink, and D.W. Snyder. 1985. Removing Barium and Radium Through Calcium Cation Exchange. *Jour. AWWA*, 77(5):60.

New Hampshire, State of. 1983. *New Hampshire Rules for the Control of Radiation*. Concord, N.H.: State of New Hampshire, State Department of Health and Welfare.

Rogers, V.C., and K.K. Nielson. 1981. *A Handbook for the Determination of Radon Attenuation Through Cover Materials*. NUREG/CR-2340. Washington, D.C.: U.S. Nuclear Regulatory Commission.

Rydell, S., B. Keene, and J.D. Lowry. 1989. Granular Activated Carbon Water Treatment and Potential Radiation Hazards. *Jour. New England Water Works Association*, 103(7):234–248.

Schliekelman, R.J. 1976. *Determination of Radium Removal Efficiencies in Iowa Water Supply Treatment Processes*. EPA-ORP/TAD-76-1. Cincinnati, Ohio: USEPA.

Singley, J.E., B. Beaudet, W. Bolch, and J. Palmer. 1977. *Cost of Radium Removal From Potable Water Supplies*. EPA-600/2-77-073. Washington, D.C.: USEPA.

Snoeyink, V.L., C.K. Jongeward, A.G. Myers, and S.K. Richter. 1984a. Characteristics and Handling of Wastes From Groundwater Treatment Systems. In *Proc. of the AWWA Annual Conference*. Denver, Colo.: AWWA.

Snoeyink, V.L., J. Pfeffer, D. Snyder, and C. Chambers. 1984b. Barium and Radium Removal From Groundwater by Ion Exchange. EPA/600/2-84/093. Washington, D.C.: USEPA.

Subramonian, S., D.A. Clifford, and W. Vijjeswarapu. 1990. Evaluating Ion Exchange for the Removal of Radium From Groundwater. *Jour. AWWA*, 82(5):61–70.

USDOT (U.S. Department of Transportation). 1976. 49 CFR Part 172. Hazardous Material Transportation Uniform Safety Act. April 15, 1976. 41 *Fed. Reg*. 15996. Washington, D.C.: USDOT.

USEPA (U.S. Environmental Protection Agency). 1987. *Radiation Protection Guidance to Federal Agencies for Occupational Exposure*. Recommendations approved by the President. 52 FR 2822. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1990. *Suggested Guidelines for the Disposal of Drinking Water Treatment Wastes Containing Naturally-Occurring Radionuclides*. Washington, D.C.: USEPA, Office of Drinking Water.

USEPA (U.S. Environmental Protection Agency). 1994. *Suggested Guidelines for Disposal of Drinking Water Treatment Wastes Containing Radioactivity*. Washington, D.C.: USEPA, Office of Ground Water and Drinking Water.

USNRC (U.S. Nuclear Regulatory Commission). 1990. *Severe Accident Risks: An Assessment for Five U.S. Nuclear Power Plants*. Vol. 1. NUREG-0933. Washington, D.C.: USNRC.

CHAPTER 10
GRANULAR ACTIVATED CARBON

Another waste product that is occasionally produced at WTPs is spent granular activated carbon. GAC is sometimes used in the filters or postfiltration of a conventional plant and used as a separate absorber on groundwater. When it is used for taste and order removal, the carbon is disposed of after its capacity is exhausted. When it is used is for continuous low-level organics removal, then the carbon is usually regenerated onsite or returned to the vendor. GAC can also be used for trace organic removal. Federal regulations that could apply to the disposal of GAC primarily address hazardous waste properties. These regulations are described in chapter 5. Radioactivity associated with GAC is addressed in chapter 9. This chapter addresses GAC used for organic removal and the potential buildup of organics on the GAC to a level of regulatory concern as well as issues associated with carbon regeneration.

TESTING CARBONS FOR HAZARDOUS PROPERTIES

There have been limited literature citations of spent GAC being categorized as hazardous solid waste under the criteria of Subpart C of the RCRA regulation. It is possible that a spent GAC could become categorized as a hazardous waste due to toxicity, because of the adsorption mechanism and affinity of GAC for certain contaminants that have been listed in the toxicity criteria.

The TCLP, as applied to GAC, is described in subsequent sections. As discussed in chapter 5, toxicity of a waste is determined by an analysis of the extraction fluid that has been in contact with the waste for at least 18 hours. The extraction fluid is analyzed for several contaminants, and if any of the contaminants exceeds a certain level, then the waste is considered to be hazardous because of its toxicity.

The list of the contaminants analyzed in the TCLP, and the level at which the individual contaminants are considered to be present in toxic amounts (regulatory levels), is shown in Table 5.9. The list includes many chemicals that are present in source waters treated with GAC. If the GAC adsorbs sufficient levels of these chemicals that its extract exceeds the TCLP limit, then the GAC would be categorized as hazardous.

Theoretically, all the contaminant present on the GAC will desorb. However, the extraction procedure may be rate limiting and the contaminants may have low solubility in the extraction fluid so that incomplete desorption takes place.

GAC can adsorb many different organic contaminants, and in water treatment, GAC does, in fact, adsorb many of the organic contaminants present in the source water. However, not all of the contaminants on the TCLP list would be expected to be present on the GAC at levels that would trigger a “toxic” categorization.

In general, GAC will not remove much of the dissolved metals that may be in source waters. Some removal may be noted in the metals concentration, often because of the filtration of the GAC particles, due to the adsorption of neutral metal species or organic–metal complexes, but the amount of metal removal that could occur would be limited.

One area of concern regarding metals was cited by work done by USEPA concerning the amount of metals that are present in the GAC itself (Koffskey and Lykins 1987). Because of the manufacturing process used to make activated carbons, some metals may be present in the virgin GAC. USEPA researchers subjected samples of virgin GAC to acid desorption for 24 hours.

Table 10.1
Heavy metals found on virgin GAC after acid digestion

Metal	Carbon 1 (mg/kg C)	Carbon 2 (mg/kg C)
Arsenic	108	3.5
Barium	50.4	54.5
Cadmium	1.2	—*
Chromium	7.2	2.7
Lead	16.8	—
Mercury	1.9	0.2
Selenium	—	—
Silver	—	—

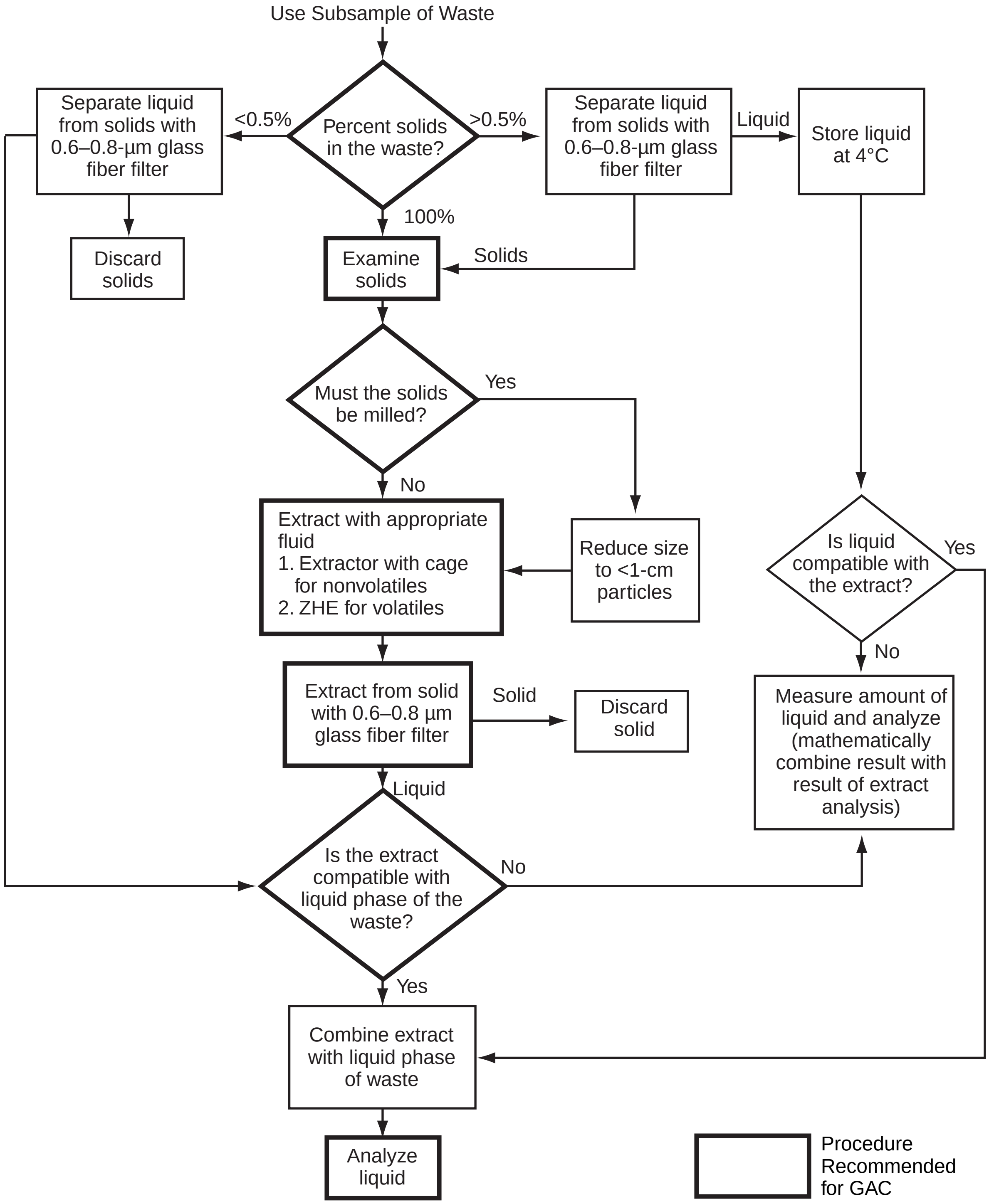
Source: Koffskey and Lykins 1987.
*No data.

Table 10.1 shows the concentrations of selected metals regulated by the TCLP found in this work. The leachate was analyzed for the metals, and the results are reported in milligrams per kilogram of carbon. These values, when converted to milligrams per liter in the extract, are well below the TCLP regulatory levels even if all the metal were extracted. But it is clear from this work that the GAC medium itself could potentially contribute to the concentration of metals in the TCLP leachate. However, it would be unlikely that a spent GAC would fail the TCLP test because of high levels of any of the metals on the TCLP list.

The TCLP list contains several volatile and semivolatile organics that GAC has been shown to remove from source waters. It would not be unusual for the GAC to remove concentrations of these chemicals many times the background levels. Over time, if the organic chemicals are present in the source water, the GAC could easily adsorb the “regulatory threshold” level of the chemical. The question again is how much of the adsorbed organic compound will desorb in the TCLP.

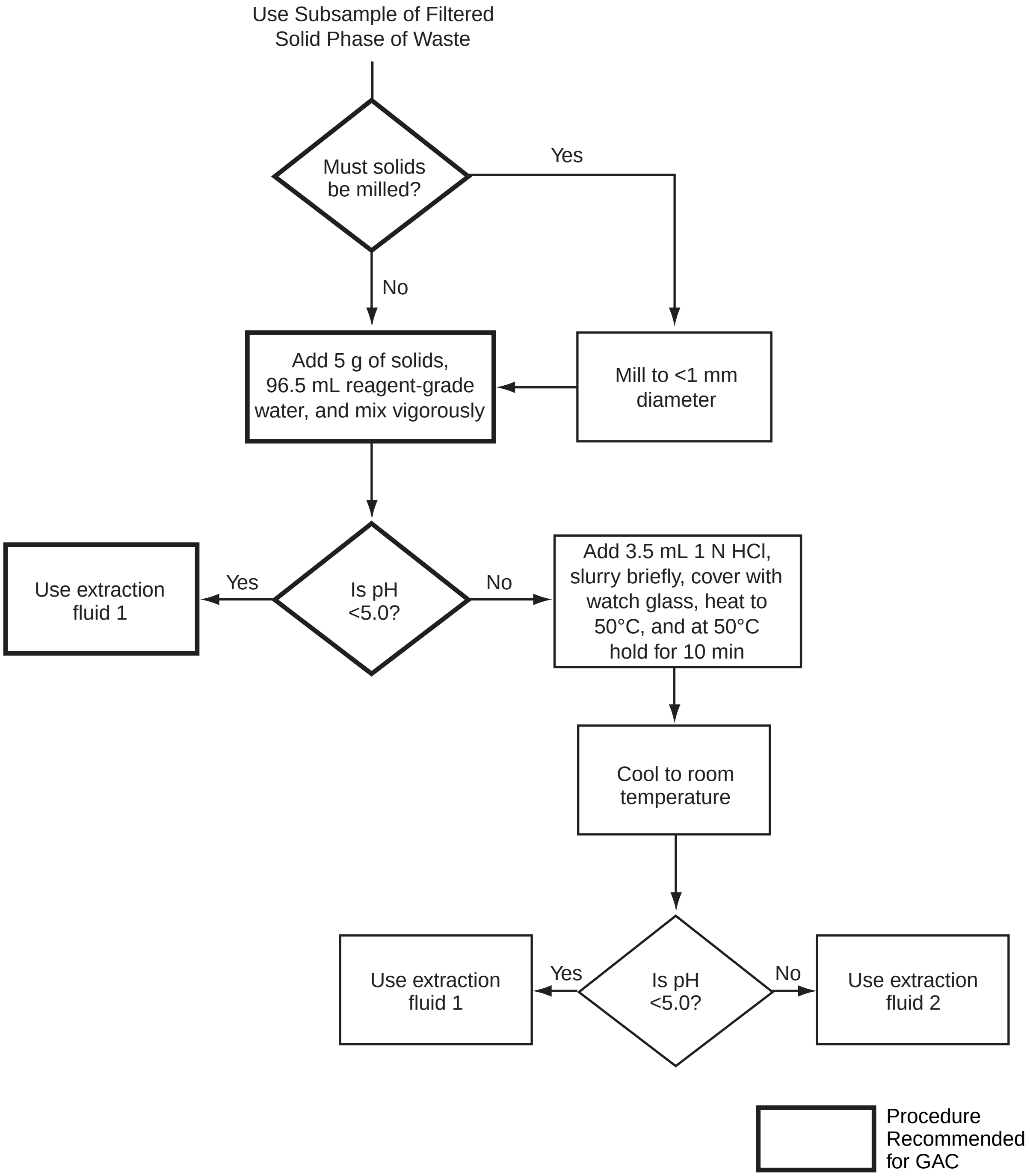
It is clear that GAC users need to have information regarding the potential of GAC to pass the TCLP test. In the absence of such information, the Colorado Department of Health (CDH) determined that GAC being used to remove TCE from contaminated groundwater at South Adams County Water and Sanitation District, Colo., would be categorized as an RCRA hazardous waste (Michael, Crittenden, and Snoeyink 1991). This determination was based solely on the presence of TCE in the source water, as no exhausted GAC was available for TCLP analysis at the time of CDH’s determination. Subsequently, South Adams County conducted TCLP extractions on GAC samples from pilot columns at the site and analyzed them for TCE. These data, referred to as the TCE–TCLP data on the exhausted GAC, did not exceed the TCLP toxicity limit.

The general procedure for conducting a TCLP test is shown schematically in Figures 10.1 and 10.2. Figure 10.1 also highlights the procedure that is used specifically for the analysis of GAC. The procedure contains four stages: sample collection, sample preparation, the leaching procedure, and the analysis for listed contaminants.



Source: McTigue and Cornwell 1994.

Figure 10.1 General procedure for TCLP extraction from GAC



Source: McTigue and Cornwell 1994.

Figure 10.2 Procedure for selecting type of extraction fluid for TCLP test of GAC

The RCRA regulation specifies that representative samples of the solid waste be analyzed. There is no regulatory obligation to test the worst-case, most-contaminated carbon. The complete contents of the contactor can be mixed, and a composite sample that is representative of the complete batch to be disposed of can be collected for analysis.

Approximately 500 g should be collected from each location in order to provide sufficient quantity of the GAC for preliminary evaluation for extraction fluid determination, for inorganic and organic analysis, and for backup.

In order to conduct the actual leaching procedure, the GAC must be prepared. In general, if the material to be analyzed is a liquid with <0.5 percent dry solids material, the liquid itself is considered to be the extract and is analyzed. Solid samples must be "100 percent solid." This criterion is defined in the TCLP regulation as a material that yields no liquid when subjected to pressure filtration (50 psi [345 kPa]). GAC samples would generally be considered to be 100 percent solid, but in reality, the GAC pores do contain water; so, when weighed, the weight includes some water. The presence of the moisture means that less GAC per volume of extract fluid is being used, and, therefore, there is less contaminant present per volume of fluid. There is no requirement to dry the GAC in order to obtain a true 100 percent dry-weight sample. The GAC samples should be crushed to ensure that particles are smaller than 0.4 in. (1 cm) in their narrowest dimension, as required.

Before the leaching procedure can be performed, a determination must be made regarding the type of extraction fluid to use. The general procedure for determining which fluid to use is shown in Figure 10.2. Two different extraction fluids, one having a pH of ~5 and the other a pH of ~3 are used in the TCLP test. In TCLP extractions to be used to analyze for volatile constituents, and in extractions for nonvolatile analyses in which the pH of the sample after being mixed with water and stirred vigorously is less than 5, extraction fluid no. 1 (pH 4.93) is specified. If the pH of the waste sample after being stirred is greater than 5, acid is added to the mixture, which is subsequently heated to 122°F (50°C), held at the temperature for 10 minutes, and allowed to cool. Only if the pH is less than 5 after completion of this test should fluid no. 1 be used for the extraction; otherwise, extraction fluid no. 2 should be used.

For GAC samples, McTigue and Cornwell (1994) determined that extraction fluid no. 1 should be used for extracting both volatile and nonvolatile organics. After being mixed vigorously with a specified volume of water, GAC slurry pH values were measured to be less than 5. Resulting pH values of <5 (pH dropped to <3) dictated that fluid no. 1 be utilized in the extraction of nonvolatiles; fluid no. 1 is to be used unconditionally in obtaining extracts to be used for analysis of volatile compounds.

Separate leaching procedures are generally carried out for each class of compounds of interest. The bottle extraction procedure is recommended for metals and semivolatile organics, and the ZHE procedure is recommended for volatile organics. It is acceptable, however, to use the ZHE procedure for both types of extractions (Cornwell et al. 1992).

Extraction of the solid-phase GAC samples first require the addition to the solid of an amount of extraction fluid equal to 20 times its weight followed by their combination in an extraction vessel. When obtaining an extract to be used for analysis of nonvolatiles (metals, etc.) only, a minimum waste size of 100 g is needed primarily to ensure that an adequate volume of extract is obtained for analysis. However, when an extract is to be used to evaluate the volatiles, the maximum amount of solid that can be accommodated by the ZHE vessel is determined by the volume of the extractor (500 mL). In order to maintain the extraction-fluid-to-waste ratio at 20:1 and still avoid head space, 25 g of crushed GAC is used.

The solids and the extraction fluid are placed in the extractor and extracted for 18 hours. Following the extraction, the solid and liquid phases are separated using a 0.6- to 0.8-µm glass fiber filter. The liquid extract is added to the liquid (if any) from the original separation for analysis.

GAC USED IN SURFACE WATER TREATMENT

Carbons used for surface water treatment are generally used to remove low levels of DBP precursors, to remove organic compounds that cause taste and odor problems, or to remove synthetic organic compounds (SOCs). McTigue and Cornwell (1994) analyzed GAC used at five surface water plants for hazardous properties. The first carbon sample analyzed was obtained from the Greater Cincinnati Water Works' (GCWW) Richard Miller Water Treatment Plant.

The treatment goal of GAC for GCWW is twofold—to provide a barrier between the industrialized Ohio River and the consumers' taps and to remove DBP precursors to minimize formation of THMs and other by-products in the distribution system. The Ohio River source water contains trace levels of several synthetic organics, but their presence and levels vary.

The carbon for the work reported by McTigue and Cornwell (1994) was taken from the top (influent) of a deep bed (15 ft [4.6 m]) experimental contactor that was treating chlorinated, filtered water. The influent TOC to the contactors was in the range of 2 to 3 mg/L, and the effluent was about 1 mg/L. Cincinnati considers GAC to be exhausted when the effluent from the contactor has a TOC value of about 2 mg/L. The experimental contactors are typically online for about 8 months to 1 year to achieve full exhaustion based on TOC and DBP precursors.

A complete TCLP was run on the sample collected form the experimental contactor. These results are shown in Table 10.2. No organic chemicals were found to have levels greater than the detection limits for volatiles, extractables, and herbicides. Similarly, no pesticides were detected at detection limits. Some metals were detected in the extract. Arsenic, barium, lead, and selenium were detected, but at levels well below the regulatory threshold.

The second sample collected by the researchers was from a contactor at Manchester, N.H., Water Company (MWC). MWC also uses a surface water source and has installed GAC to lower TOC in the finished water and to reduce the taste and odors in the finished water. Typically, MWC reactivates the GAC in its contactors every 3 to 4 years. In addition, the contactors may have fresh GAC added to them at any time to replace GAC that has been lost. The contactors are backwashed regularly to distribute the GAC particles uniformly through the beds. MWC reactivates the GAC onsite at its own facility. A sample of GAC was collected from a contactor that had been online for approximately 3 years.

The TCLP results for the GAC samples are shown in Table 10.2. Contaminants were not present in the TCLP extract at levels above the regulatory level. The only contaminant detected was chloroform at 0.29 ppm. The TCLP limit is 6 ppm.

Carbon samples were also collected from two Pennsylvania American Water Company facilities using GAC filter media for taste and odor control. In both cases, Aldrich Station and Hayes Mine, Pa., the utilities used GAC as a filter media replacement at a depth of 3 ft (0.9 m). Both GACs analyzed in the research had been online for about 4 years and were considered to be exhausted for taste and odor control. The TCLP data are shown in Table 10.2. TCLP contaminants at any level were not detected for either system.

Table 10.2
TCLP results for GAC used on a surface water (in ppm)

Constituent	GCWW	Manchester, N.H.	Hayes Mine, Pa.	Aldrich Station, Pa.
Metals				
Arsenic	0.03	<0.10	<0.10	<0.10
Barium	0.59	<1.0	<1.0	<1.0
Cadmium	<0.005	<0.10	<0.10	<0.10
Chromium, total	<0.01	<0.50	<0.50	<0.50
Lead	0.02	<0.50	<0.50	<0.50
Mercury	<0.002	<0.010	<0.010	<0.010
Selenium	0.08	<0.10	<0.10	<0.10
Silver	<0.01	<0.10	<0.10	<0.10
Volatile Organics				
Benzene	<0.01	<0.10	<0.10	<0.10
Carbon tetrachloride	<0.01	<0.10	<0.10	<0.10
Chlorobenzene	<0.01	<0.10	<0.10	<0.10
Chloroform	<0.01	0.29	<0.10	<0.10
1,2-Dichlorethane	<0.01	<0.10	<0.10	<0.10
1,1-Dichloroethylene	<0.01	<0.10	<0.10	<0.10
2-Butanone	<0.01	<1.0	<1.0	<1.0
PCE	<0.01	<0.10	<0.10	<0.10
TCE	<0.01	<0.10	<0.10	<0.10
Vinyl chloride	<0.01	<0.10	<0.10	<0.10
Extractable Organics				
Pyridine	<0.01	<0.10	<0.10	<0.10
o-Cresol	<0.01	<0.10	<0.10	<0.10
m-Cresol	<0.01	<0.10	<0.10	<0.10
p-Cresol	<0.01	<0.10	<0.10	<0.10
1,4-Dichlorobenzene	<0.01	<0.10	<0.10	<0.10
2,4-Dinitrotoluene	<0.01	<0.10	<0.10	<0.10
Hexachlorobutadiene	<0.01	<0.10	<0.10	<0.10
Hexachloroethane	<0.01	<0.10	<0.10	<0.10
Nitrobenzene	<0.01	<0.10	<0.10	<0.10
Pentachlorophenol	<0.01	<0.10	<0.10	<0.10
2,4,5-Trichlorphenol	<0.01	<0.10	<0.10	<0.10
2,4,6-Trichlorphenol	<0.01	<0.10	<0.10	<0.10
Hexachlorobenzene	<0.01	<0.10	<0.10	<0.10

(continued)

Table 10.2 (Continued)

Constituent	GCWW	Manchester, N.H.	Hayes Mine, Pa.	Aldrich Station, Pa.
Pesticides				
Chlordane	<0.01	<0.015	<0.015	<0.015
Endrin	<0.005	<0.010	<0.010	<0.010
Heptachlor	<0.0004	<0.005	<0.005	<0.005
Heptachlorepoxide	No data	<0.005	<0.005	<0.005
Lindane	<0.01	<0.20	<0.20	<0.20
Methoxychlor	<0.01	<1.0	<1.0	<1.0
Toxaphene	<0.01	<0.25	<0.25	<0.25
Herbicides				
2,4,-D	<0.01	<5.0	<5.0	<5.0
2,4,5-TP (Silvex)	<0.01	<0.5	<0.5	<0.5

Source: McTigue and Cornwell 1994.

PAC USE IN SURFACE WATERS

PAC is used by many utilities to remove taste and odor causing substances. It can also be used to reduce NOM or low levels of SOCs including pesticides, benzene, and toluene. The spent PAC will naturally end up in the sludge produced by the water plant and can represent a significant portion of the sludge. For example, a plant that uses 40 mg/L alum and 10 mg/L PAC to treat raw water with 30 mg/L SS would produce a sludge that is approximately 17 percent by dry weight PAC. Any contaminant on the PAC could therefore have an effect on the sludge chemical characteristics. As with any sludge, in order for the mixed PAC and coagulant sludge to be classified as hazardous, the mixed sludge would need to fail the TCLP test. There are two possible sources of contamination on the PAC sludge. The first source would be contaminants present in the virgin PAC. These contaminants would primarily be inorganic constituents. The second source would be contaminants adsorbed onto the PAC during the treatment process. These contaminants would primarily be organic contaminants.

Cornwell and Koppers (1990) have reported results of testing virgin PAC for the presence of inorganic constituents. Cornwell et al. (1992) have also reported values for the amount of contaminants that could be present in sludge and still be classified as nonhazardous by the TCLP regulation even if all the contaminant leached out during the extraction procedure. Table 10.3 compares the analyses of PACs for inorganic contaminant concentrations to the minimum concentration needed to exceed the TCLP for a sludge that is 22 percent solids concentration and all the solids were PAC. As shown in Table 10.3, the concentrations of inorganic constituents present in PAC are orders of magnitude below the minimum levels necessary to fail the TCLP test. It is therefore highly unlikely that a mixed PAC and coagulant sludge waste would be classified as hazardous by the TCLP test because of the presence of inorganic contaminants in virgin PAC.

PAC could also adsorb organic contaminants that are regulated by the TCLP test. As with the inorganic compounds, it is possible to calculate the minimum amount of an organic contaminant that

Table 10.3
Comparison of PAC inorganic analyses to minimum concentrations necessary to fail the TCLP test

Constituent	Minimum concentration in PAC to exceed the TCLP level at 22% solids concentration (mg/kg)*	Total concentration in PAC (mg/kg)†	Total concentration in PAC (mg/kg)‡
Arsenic	455	108	<5
Barium	9,090	50	45 to 78
Cadmium	90	1	<5
Chromium	455	7	<5
Lead	455	17	<5
Selenium	90	<5	<5

*****Data from Cornwell et al. 1992.
†Data from Clark and Lykins 1989.
‡Data from Cornwell and Koppers 1990.

would have to be adsorbed onto the PAC in order to fail the TCLP test if all the contaminant was absorbed during the extraction procedure. However, in this case, organic compounds can load to levels that would exceed the minimum amount required to fail the TCLP test. Therefore, whether the sludge containing PAC will fail the TCLP test because of buildup of organic compounds is determined by the amount of the compound that leaches during the TCLP test.

Hayes Mine, Pa., uses PAC to reduce low levels of benzene, toluene, and phenol but has never failed the TCLP test on its sludge. The Pennsylvania American Water Company reports that they have tested sludge at 21 water plants, of which 3 use significant amounts of PAC. No organic compounds have been detected or extracted from any of the TCLP tests. Princeton, W.Va., uses PAC and alum treatment. No organic compounds have been detected on TCLP testing of their sludge. Newport News, Va., conducted one TCLP test on its sludge when PAC was not being used and another during PAC use. No differences in results for the TCLP test were found, and no organic compounds were detected.

It is theoretically feasible for PAC to adsorb enough of an organic compound that it would fail the TCLP test. However, this would require a very high raw water concentration of the contaminant, a situation that is unlikely to occur. There are no known reports of a mixed PAC and coagulant sludge failing the TCLP test.

CARBON USED ON GROUNDWATER SOURCES

Carbons used by groundwater systems are typically meant to remove one or two specific contaminants. Generally, GAC has been used to remove specific VOCs from well supplies.

McTigue and Cornwell (1994) collected samples from several utilities using GAC for volatile organic treatment. Complete TCLP analyses were gathered for some of the systems, but for most of the systems, only those contaminants known to be present at high levels in the source water were analyzed for in the TCLP.

Table 10.4
TCLP results for GAC removing TCE at South Adams County, Colo.

Influent TCE (μg/L)	TCLP value for TCE (μg/L)
8	32
12	34
35	120

Source: Adapted from Michael, Crittenden, and Snoeyink 1991.

Carbon samples were collected from utilities that were using GAC to remove TCE and PCE. These contaminants were chosen for evaluation because they are commonly found volatile organics in groundwater sources.

Michael, Crittenden, and Snoeyink 1991 reported on spent GAC characteristics from the South Adams County (Colo.) Water and Sanitation District. South Adams County treats groundwater that is primarily contaminated with TCE with GAC contactors. TCE was present in the raw water at levels of 2 to 37 μg/L. South Adams County reported TCLP results for TCE (TCE–TCLP) based on carbons used to treat three different average influent TCE concentrations. The results they obtained are shown in Table 10.4.

The four utilities that were analyzed by McTigue and Cornwell (1994) were St. Anthony, Minn.; New Brighton, Minn.; Park Ridge, N.J.; and Lansdale, Pa. TCE levels in the raw water ranged from 5 to 350 μg/L. In some cases, multiple carbon samples were obtained. The multiple samples were obtained when possible from parallel contactors treating the same influent source. In other cases, the utility had contactors on different sources allowing evaluation of different influent TCE concentrations.

A summary of the results of the testing conducted are shown in Table 10.5. The table contains average influent TCE concentrations ranging from 5 to 350 μg/L. The TCLP values ranged from 17 to 1,140 μg/L. Three of the samples exceeded the 500-μg/L TCLP regulatory level for TCE.

A plot of the TCLP results (TCE–TCLP) from the carbon systems previously reported is shown in Figure 10.3. Although the carbons treated a variety of water sources, with different NOM characteristics, the relationship between influent TCE and TCLP values for TCE in the extract is clear. From this plot, it appears that a GAC TCLP extract could fail the TCLP test for TCE (>500 μg/L) if the influent TCE level was greater than 150 μg/L.

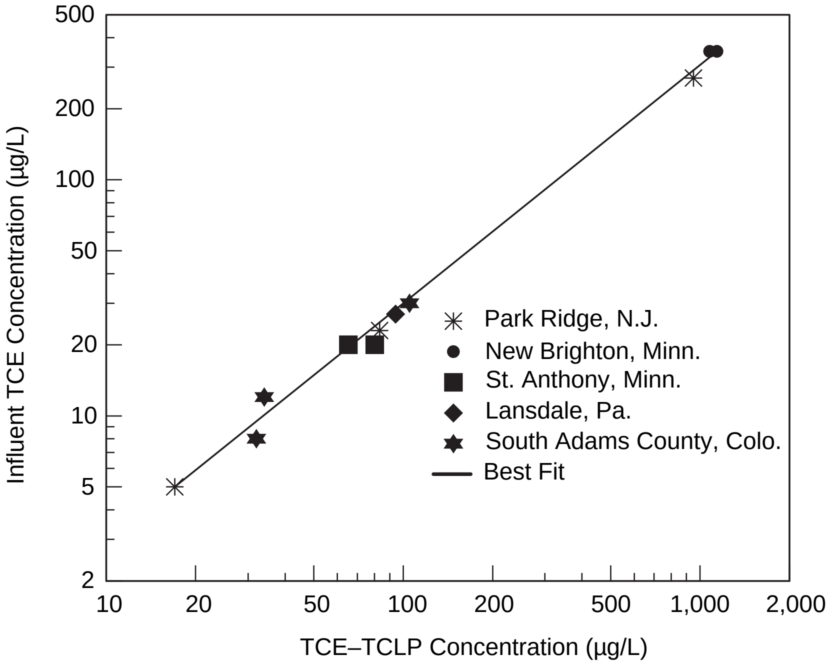
Two GAC samples were collected from a facility in Palmyra, N.J. GAC is used at that utility to remove PCE. The GAC contactor sampled is generally able to operate for 2 years before the bed is exhausted for PCE removal. TCLP extraction was conducted on the GAC samples, and PCE was analyzed. The carbons had been online about 15 months when the samples were collected. The two samples collected were from contactors that were in series, and so received water with different PCE influent concentrations. These concentrations, and the PCE–TCLP concentrations, are shown in Table 10.6.

PCE was also analyzed for in the TCLP extract from Park Ridge, N.J., and data are available for South Adams County water (Michael, Crittenden, and Snoeyink 1991). None of the values exceed the PCE–TCLP threshold of 700 μg/L.

Table 10.5
TCLP test results for GAC used to remove TCE

Plant	Average influent TCE (µg/L)	TCLP analysis for TCE (µg/L)
Park Ridge, N.J.		
Sample no. 1	5	17
Sample no. 2	23	83
Sample no. 3	270	949
New Brighton, Minn.		
Sample no. 1	350	1,140
Sample no. 2	350	1,078
St. Anthony, Minn.		
Sample no. 1	20	65
Sample no. 2	20	80
Lansdale, Pa.		
Sample no. 1	27	94

Source: McTigue and Cornwell 1994.



Source: McTigue and Cornwell 1994.

Figure 10.3 Relationship between influent TCE concentration and TCLP extract concentration from carbon

Table 10.6
Palmyra, N.J., GAC TCLP–PCE results

Location	Average influent PCE (μg/L)	TCLP analysis for PCE (μg/L)
Filter no. 1	44.75	21.2
Filter no. 2	25.75	12.8

Source: McTigue and Cornwell 1994.

All of these data are plotted in Figure 10.4. Again, although the data were collected from significantly different water sources using different types of carbon, the relationship between influent PCE concentration and PCE–TCLP levels is described by a single function. The PCE data were described by the following equation:

$$\text{raw water PCE, } \mu\text{g/L} = 2.55\ (\text{PCE–TCLP, } \mu\text{g/L})^{-9.3} \tag{10.1}$$

with a correlation coefficient of 0.96. Equation 10.1 could be used to determine the raw water PCE concentration at which the TCLP test would be failed:

$$\begin{aligned}\text{raw water PCE, } \mu\text{g/L} &= 2.55\ (700)^{-9.3} \\ &= 1{,}775\ \mu\text{g/L}\end{aligned} \tag{10.2}$$

This indicates that the raw water PCE level would have to exceed 1,775 μg/L in order for the GAC to fail the TCLP test. However, data were only collected at raw water levels below 50 μg/L, so it is not known if this relationship is valid at the high concentrations.

Equation 10.1 can be rearranged in order to estimate the PCE concentration in the TCLP extract if the raw water PCE concentration is known:

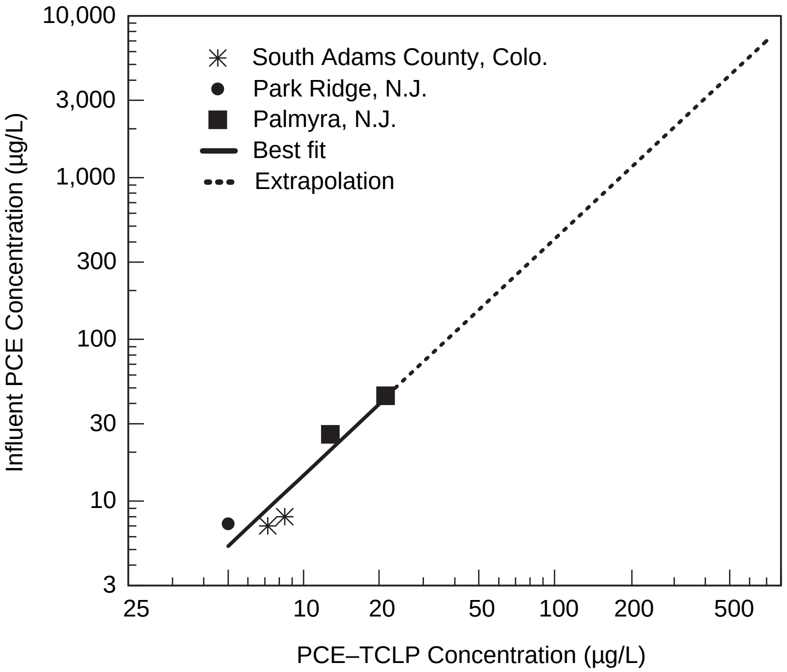
$$\text{PCE–TCLP, } \mu\text{g/L} = 0.38\ (\text{raw water PCE, } \mu\text{g/L})^{+3.92} \tag{10.3}$$

GAC DISPOSAL OPTIONS

Ultimately, GAC used in water treatment must be disposed of or regenerated. GAC has a limited useful life, the length of which depends on the specific contaminant being removed and the unique water quality characteristics of the source water. Once GAC is exhausted, a utility can choose to

- Return it to the vendor for regeneration
- Dispose of it in a landfill
- Bring it to a regional regeneration facility
- Regenerate it onsite
- Store it onsite indefinitely

Whichever option is chosen, because GAC is considered to be a solid waste, the utility, under the RCRA, must determine if it is also a hazardous solid waste.



Source: McTigue and Cornwell 1994.

Figure 10.4 Prediction of TCLP results from carbon removing PCE

Return to Vendor

Many utilities arrange for a carbon supplier to remove spent GAC as part of the original purchase agreement, replacement purchase agreement, or as a separate work item. All carbon suppliers will require that a TCLP test be run on the carbon, although it may be possible for the utility to have the carbon manufacturer conduct this test. The test must be completed prior to transport to determine proper handling methods. If the material is characterized as "nonhazardous," then the transport and handling is relatively straightforward and not subject to RCRA Subpart C restrictions. In the United States, all water treatment carbons returned for regeneration are subsequently used for applications other than water treatment or food production. Only virgin carbon is supplied by manufacturers for use in water treatment.

If a GAC is characterized as hazardous because of TCLP toxicity or state requirements, only carbon manufacturers permitted as an RCRA transport, storage, and disposal (TSD) facility and having both Part A and Part B of the TSD permit can accept this material for regeneration; not all companies have this permit. These permitted companies can accept RCRA hazardous waste, unless PCBs, dioxins, or dibromochloropropanes (DBCPs) are present. PCBs, dioxin, and DBCPs are not RCRA-listed hazardous wastes. However, PCBs are regulated under the TSCA (40 CFR 760; USEPA 1976), and their presence requires that the waste be incinerated before being handled by an RCRA TSD facility. The Federal Insecticide, Fungicide, and Rodenticide Act forbids the storage or handling of any waste containing dioxin. The California Occupational Safety and Health Administration limits the exposure to DBCP. Because of these restrictions by regulations more stringent than the RCRA, TSDs cannot handle wastes containing PCBs, dioxins, or DBCPs.

Once a hazardous classification is made, the water system is considered to be a generator of hazardous wastes, and so is bound by the requirements of the cradle-to-grave manifest system and the transportation requirements of the RCRA and DOT.

A generator or transporter of hazardous waste is not only required to follow the RCRA regulations previously discussed, but also must comply with DOT regulations under the Hazardous Materials Transportation Act (HMTA). Under these regulations, the generator is required to identify and classify wastes according to DOT's Hazardous Materials Table (HMT; 40 CFR 172.101) (USDOT 1976). DOT considers hazardous wastes to be a subset of hazardous materials; consequently, the RCRA designated that hazardous waste will always be subject to the requirements of the HMTA. In addition, the generator is required to comply with all packaging, marking, and labeling requirements in addition to providing appropriate placards to the transporter. The transporter is responsible for verifying that the shipment is properly identified, packaged, marked, labeled, and not leaking. The transporter is also required to apply appropriate placards and comply with DOT incident reporting rules.

The transportation of hazardous waste is governed by DOT, and the requirements are extensive. A good discussion of the requirements can be found in *Hazardous Waste Identification and Classification Manual* by Wagner (1990). The following section is summarized from that work. Title 49 CFR, Section 171.3 (USDOT, 1976), states that "no person may offer for transportation or transport a hazardous waste (defined as any material subject to the manifest requirements of 40 CFR Part 262) in interstate or intrastate [shipment] except in accordance with the requirements of this subchapter."

When a utility offers a hazardous waste for transportation, the utility is considered the shipper and must comply with specific regulations for identification, classification, and shipping documentation, as well as for labels, packages, markings, and placards. This information is contained in DOT's HMT. Each of these actions is designed primarily to protect the health of humans who may be exposed to such wastes during transportation.

Typically, a water utility would contract with the RCRA-permitted carbon manufacturer to transport as well as regenerate the carbon. Whatever company, or person, moves the waste from the generator to the TSD facility is known as the transporter. Some vendors maintain a fleet of RCRA-permitted vehicles that can handle the material. Commercial trucking companies can also be used for this transport, if they are similarly licensed under the RCRA. However, it is the responsibility of the generator (the water utility) to initiate the manifesting process itself. Although the transporter and TSD facility can offer assistance to the utility based on their experience and familiarity with the requirements, the utility is responsible for the correct preparation of the manifest.

Disposal in a Landfill

A utility may choose to dispose of its GAC in a landfill, although this is not typically done, because it is usually more cost-effective to have the carbon regenerated. Landfill disposal can be used, however, for both nonhazardous and hazardous GACs. In either case, the GAC has to meet the criteria of having no free water associated with it, based on a paint filter liquid test.

Utilities often dispose of water treatment residuals in municipal and industrial solid waste facilities. No federal mandates are currently in place that would prohibit the disposal of spent GAC deemed nonhazardous in a MSWLF. Disposal of nonhazardous wastes in MSWLFs is governed at the federal level by RCRA regulations. It should be noted, however, that more-stringent individual state regulations disallowing disposal of GAC in MSWLFs would override RCRA rules. Particularly where municipal solid waste or industrial waste landfills are located in close proximity to small WTPs that generate relatively small amounts of residuals, codisposal could be an option for spent GAC disposal.

Most municipal solid waste facilities have quality criteria that solid waste would have to meet in order to be accepted, including the requirement that no free water be present as demonstrated by the paint filter liquid test (chapter 5). Spent GAC would have to be dewatered to this level, but simple drainage and drying should accomplish this. If the spent GAC passed the TCLP test, it should be accepted in this type of landfill.

If a GAC sample fails a TCLP test, or is classified as hazardous by the state-regulating agency, it will have to be disposed of in an RCRA-permitted landfill if landfill disposal is chosen by the utility. In this case, the same cradle-to-grave manifest system and transportation requirements that were discussed earlier would apply. Again, the water utility would be considered the generator and therefore would be responsible for the GAC until it is delivered to the landfill. This responsibility includes creating and maintaining the waste manifest and arranging for the GAC to be handled and disposed of in an RCRA-permitted TSD facility.

It would generally be less expensive and more convenient for a utility to return a hazardous GAC to a manufacturer for regeneration than to dispose of it in an RCRA landfill. Depending on the distance of the utility from the RCRA landfill, the transportation costs can be considerable. However, manufacturers may only accept their own carbons for regeneration, and only some carbon manufacturers are permitted to handle RCRA wastes. If a water utility's hazardous GAC was not originally obtained from one of these RCRA-permitted manufacturers, then disposal in an RCRA landfill may be that utility's only alternative.

Regional Regeneration

This option may be utilized by utilities under the same conditions as described in the Return to Vendor section. If the spent GAC fails the TCLP test, then the carbon could be sent to a regional regeneration facility only if that facility was permitted under the RCRA as a TSD facility.

Onsite Regeneration or Storage

Both onsite regeneration and storage eliminate the need to transport the GAC over public roadways. However, RCRA regulations still apply. If the GAC is nonhazardous, the water utility would not need to take any special precautions. However, the utility must conduct the TCLP tests to prove that it is not hazardous. If the GAC failed the TCLP test, the utility would need to apply for a special RCRA permit for operation as a TSD facility. Because transportation costs are eliminated in onsite regeneration, the cost of regenerating hazardous GAC onsite may be less than regenerating it offsite, depending upon the amount of carbon used.

Storing or regenerating hazardous GAC onsite would, however, add considerable complexity to a utility's operation of a GAC system. The water utility would have to apply for, and, in a reasonable period of time, be given a permit to operate as a TSD facility under the RCRA. Because, in this case, the water utility would be the generator, transporter, and TSD facility, a waste manifest would not be required. Instead, the permit would require certain reporting requirements to USEPA or the state RCRA agency. The requirements of obtaining a TSD designation are as follows (from Wentz 1989):

> Under RCRA, there are two categories of TSD facilities: those that have a full permit and those that do not hold a regular permit but have an interim status facility permit. TSD facilities operating under interim status must meet only a portion of those requirements of a permitted facility. Interim status permits primarily ensure that the TSD facility is following good housekeeping practices. The standards for fully accredited TSD facilities, on the other hand, are based upon the design and operating criteria specified in the TSD facility permit. A[n] RCRA operating permit is obtained by all TSD facilities through a standardized process:
>
> - Submittal of the permit application
> - Regulatory review of the application
> - Preparation of the draft permit
> - Request for public comment
> - Permit finalization
> - Maintenance and termination of permit

The TSD facility is required to submit a permit application that covers the design, operation, and maintenance of the facility. This permit application is divided into Parts A and B. Part A requests general information, including a description of the activities to be conducted at the facility. Part B requires highly detailed and technical information, such as analyses of hazardous wastes to be managed at the facility.

The administrative review process initially focuses on the completeness of the permit application. Once the application has been deemed complete, a review process by USEPA determines either approval or denial of the application. This lengthy process may take as long as 3 years.

After the application has been approved, a draft permit, which incorporates technical operating requirements and other conditions, is prepared to ensure compliance. When the draft permit has been completed, USEPA is required to solicit written public comments for 45 days. A public hearing may be held during this same time period. Following the public comment period, USEPA issues the RCRA permit decision.

Issuance and maintenance of the permit is based on compliance with numerous federal laws, including the Endangered Species Act, the Wild and Scenic Rivers Act, the National Historic Preservation Act, the Fish and Wildlife Coordination Act, and the Coastal Zone Management Act.

Each TSD facility must have a USEPA identification number. These facilities must also properly identify and handle their wastes and ensure that personnel are properly trained in hazardous waste management. The TSD facility must conduct waste analyses prior to treatment, storage, and disposal of the material. The TSD facility must have security barriers in place to prevent people from unauthorized entry into the facility. Employee training must be provided to reduce the potential for operational errors that might threaten human health or the environment (Wentz 1989).

Clearly, the procedure required to obtain a TSD permit is complex and lengthy and would likely add to the cost of onsite regeneration and storage.

RESIDUALS FROM ONSITE REGENERATION OF GAC

When GAC is regenerated onsite, both water waste streams and air off-gases are produced. Water waste streams are produced from the transport of spent GAC from the contactors to the regeneration facility and from the carbons returning to the contactors after regeneration. A water waste stream can also be produced from scrubber water if used in treating the regeneration furnace off-gases. The off-gases themselves are produced during the carbon regeneration, and these gases may require treatment prior to discharge to the atmosphere.

Regeneration of spent carbon may be accomplished by thermal, chemical, hot-gas, solvent, or biological methods. Thermal regeneration is universally employed for this purpose. Thermal regeneration is carried out in the following steps:

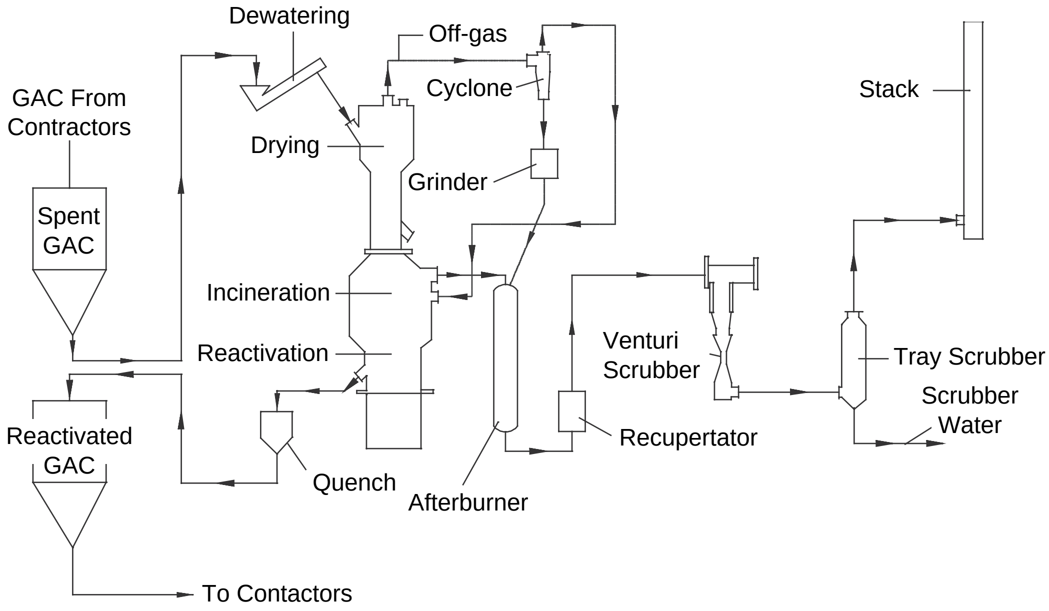
1. Dewatering
2. Drying
3. Pyrolysis or adsorbate
4. Activation
5. Quenching
6. Pollution control

Different types of thermal regeneration furnaces for GAC are available, such as multiple hearth, fluidized bed, rotary kiln, and infrared reactivator. The types of regeneration facilities vary in certain operational aspects, and the specific characteristics of the GAC and source water dictate the best choice of furnace. The fluidized bed reactivator at the GCWW and the infrared reactivator at Jefferson Parish, La., are discussed in detail in the following sections.

The basic components of the reactivation system are a storage bin for the spent carbon; a screw conveyor to dewater and convey the GAC; and the furnace, complete with pollution control equipment. The regenerated GAC is quenched and educted back to the contactors or to a GAC storage bin. The general reactivation schematic used by GCWW is shown in Figure 10.5.

The reactivator can be divided into four zones where the four stages of regeneration occur. The bottom, or combustion, zone is the area where stoichiometrically balanced ratios of air and fuel are burned at temperatures of 1,900°F (1,037°C).

The reactivation and the incineration zones are actually one integral chamber. The bottom half of this chamber is referred to as the reactivation chamber and is where the fluidized bed of GAC is maintained. Fluidization gas enters this chamber through nozzles mounted on a plate that separates this chamber from the combustion zone. Additional air and fuel are added using secondary burners to incinerate organics that desorb during reactivation. The temperature in this zone is between 1,500° and 1,800°F (815° and 980°C). Off-gas from the reactivator and incinerator passes directly to a venturi scrubber and multiple tray scrubber prior to being vented to the atmosphere through a stack. The final zone, the dryer, is separated from the reactivation and incinerator chamber by a perforated steel plate with nozzles. The fluidization gas is provided by the reactivation and incineration chamber, and the temperature is maintained at 300°F (150°C) by spraying water. Off-gas from the dryer is passed through a cyclone to remove the dust particles, and the particle-free gas is recycled back to the incineration zone.



Source: Demarco, Miller, and Hartman 1988.

Figure 10.5 GAC reactivation system with afterburner

Transport Waters

Data on GAC transport (transport of GAC from a filter absorber) water quality collected by the American Water Works Service Company (AWWSC, Voorhees, N.J.) are shown in Table 10.7; water contained high levels of solids, with turbidity and particle counts up to nearly 500 ntu and 2×10^6 counts/mL, respectively. Some metals were present in high concentrations also (e.g., aluminum, 25.8 to 108.0 mg/L; iron, 5.2 to 9.0 mg/L; and manganese, 4.2 to 49.2 mg/L). DBP levels approached Stage I D/DBP Rule limits for both THMs and HAAs. GCWW collected data on their GAC backwash water and GAC transport water from a postfilter absorber. These results are shown in Table 10.8. Both of these waters showed good water quality.

Regeneration Off-Gases

The potential airborne contaminants of concern in regeneration off-gases include various forms of dioxin and furan. These by-products have been found in stack effluent gas and particulates, cyclone catch, and scrubber water in trace quantities. These contaminants could be formed as by-products of reactivation because of the presence of certain adsorbed organics and the temperatures in the furnaces.

The GCWW conducted a series of experimental runs that looked at air emissions from the generation facility with and without an afterburner. In addition, the GAC regenerated in these experiments had been exhausted by means of feedwater that did not contain chlorine. Previous work had been conducted using GAC that had been fed with chlorinated water.

Table 10.9 summarizes the impact of chlorination of the GAC influent water on the formation of tetrachlorinated dibenzo dioxin (TCDD) and tetrachlorinated dibenzo furan (TCDF). Total organic halides (TOX) (milligrams per kilogram of GAC) were lower when prechlorination was

Table 10.7
Contaminant concentrations in GAC transport water from a filter absorber

Contaminant	Units	Total concentration range	No. of samples
Inorganics			
Turbidity	ntu	168–476	*n* = 2
Particle counts		1,932,400	*n* = 2
TSS	mg/L	315–652	*n* = 2
Aluminum	mg/L	25.8–108.0	*n* = 2
Arsenic	mg/L	<0.001–0.050	*n* = 2
Barium	mg/L	0.120–0.640	*n* = 2
Chromium	mg/L	<0.01	*n* = 2
Copper	mg/L	<0.01–0.130	*n* = 2
Iron	mg/L	5.2–9.0	*n* = 2
Lead	mg/L	0.16–0.30	*n* = 2
Manganese	mg/L	4.2–49.2	*n* = 2
Mercury	mg/L	<0.0005	*n* = 2
Selenium	mg/L	<0.002	*n* = 2
Silver	mg/L	<0.001	*n* = 2
Zinc	mg/L	<0.01	*n* = 2
Chlorine residual	mg/L	0.9–3.0	*n* = 2
pH		7.35–7.62	*n* = 2
Organics			
TTHM	mg/L	0.031–0.063	*n* = 2
HAA	mg/L	0.0024–0.053	*n* = 2

Source: AWWSC 1998.

not practiced. There was also a reduction in the TCDD and TCDF concentrations in the cyclone catch and the gas-phase stack emissions.

The impact of the afterburner on the release of chlorinated dibenzo dioxin (CDD) and chlorinated dibenzo furan (CDF) is shown in Table 10.10. Levels of these substances were decreased when the afterburner was utilized.

Eliminating prechlorination and adding an afterburner to further treat the off-gas streams lowered the concentrations of the contaminants of concern. It should be noted, however, that the levels observed in the GCWW research, even in the worst-case scenario, did not present a significant health risk according to USEPA estimates. It was concluded through this research that

> ...dioxins and furans have been discovered during reactivation of GAC. The health risks posed by the levels found have been shown to be significant. The performance of two sequential phases showed that chlorinating water before GAC treatment instead of after

Table 10.8
Chemical analysis results for carbon backwash and carbon transport water from GCWW

Contaminant	Carbon backwash[*] Unfiltered	Carbon backwash[*] Filtered	Carbon transport[†] Unfiltered	Carbon transport[†] Filtered
Aluminum (mg/L)	0.382	0.189	0.414	0.159
Iron (mg/L)	0.306	0.100	0.222	0.100
Manganese (mg/L)	0.030	0.01	0.11	0.01
Turbidity (ntu)	1.41	<0.001	7.25	<0.001
VOCs (mg/L)	<0.0005	<0.0005	<0.0005	<0.0005
TOC (mg/L)	NA[‡]	1.6	NA	0.95
TSS (mg/L)	11.7	NA	3.1	NA
Silver (mg/L)	<0.02	<0.02	<0.02	<0.02
Arsenic (mg/L)	<0.005	<0.005	<0.005	<0.005
Barium (mg/L)	0.118	0.058	0.064	0.065
Cadmium (mg/L)	<0.0002	<0.0002	<0.0002	<0.0002
Chromium (mg/L)	0.003	<0.001	0.002	<0.001
Copper (mg/L)	<0.05	<0.05	<0.05	<0.05
Mercury (mg/L)	<0.0005	<0.0005	<0.0005	<0.0005
Lead (mg/L)	<0.005	<0.005	<0.005	<0.005
Selenium (mg/L)	<0.005	<0.005	<0.005	<0.005
Zinc (mg/L)	0.074	<0.05	0.05	<0.05

Source: GCWW 1998.

*Spent backwash water from a postfilter carbon contactor that has been in service for more than 60 days.

†Water that has been used to transport spent carbon from postfilter absorbers to the storage bins.

‡NA = not available.

> GAC treatment increased the levels of dioxins detected during GAC reactivation. However, even with prechlorination, the reactivation system stack emissions are well below any credible risks. Postchlorinating water after GAC treatment even further reduces any risk potential by reducing dioxin and furan levels. Finally, the use of a high temperature afterburner when reactivating GAC that received unchlorinated water provides the ultimate levels of protection available at the Cincinnati site (DeMarco and Miller 1985).

Research on off-gas emissions was also conducted at the 3-mgd (11.4-ML/d) GAC filtration and reactivation system located at Jefferson Parish, La. (Koffskey and Lykins 1990). Water was taken from the Mississippi River, treated with ferric sulfate and lime, and passed through sand filters. Water was taken from the filters and sent through post-GAC contactors during the operation of the pilot facility. The GAC was regenerated in an infrared reactivation furnace.

Table 10.9
Effect of chlorination on total TCDD and TCDF formation in carbon regeneration at GCWW

	Prechlorination			No prechlorination		
Sample	TCDD	TCDF	TOX (mg Cl/kg GAC)	TCDD	TCDF	TOX (mg Cl/kg GAC)
Spent GAC feed (ng/g)	ND*	ND	657	ND	0.04	398
Reactivated GAC (ng/g)	NS†	NS	55	ND	ND	<50
Cyclone catch (ng/g)	2.2	1.7	NS	ND	0.4	NS
Scrubber water (ng/L)	0.08	ND	NS	ND	ND	NS
Quench water (ng/L)	ND	ND	NS	NS	NS	NS
Stack emission						
Particulates (ng/L)	0.14	0.20	NS	ND	ND	NS
Gas phase (ng/L)	ND	0.04	NS	ND	0.34	NS

Source: DeMarco, Miller, and Hartman 1988.

*ND = not detected.

†NS = not sampled.

Table 10.10
Effect of afterburner with 2-sec residence time on total CDD and CDF air concentrations at GCWW

	Afterburner off		Afterburner on	
	CDD	CDF	CDD	CDF
Particulate (ng/m^3)	1.610	0.375	1.257	0.171
Gas phase (ng/m^3)	0.972	4.285	0.145	0.201

Source: DeMarco, Miller, and Hartman 1988.

The basic components of the reactivation system included a spent GAC tank, reactivation furnace, reactivated GAC tank, and makeup GAC tank. The GAC was transported horizontally on a conveyor as the heat from the infrared source regenerated the GAC thermally.

The reactivation process involved drying, volatilization, and pyrolysis of adsorbed organics and reactivation with flue gas and steam. After the spent GAC was dewatered to about 50 percent moisture by weight, it was transported at a depth of 0.75 in. (1.9 cm) through the reactivator. The drying zone of the furnace was maintained at between 500° and 600°F (216° and 316°C) by the residual heat from the pyrolysis zone. The temperature of the pyrolysis zone was maintained at 1,400°F (760°C). The final stage of the regeneration was the reactivation zone, which was maintained at 1,700°F (927°C). To provide a minimum amount of GAC agitation during the reactivation process and also to provide constant temperature, cake breakers were

Table 10.11
Average TCDD and TCDF concentrations in regeneration of GAC at Jefferson Parish, La.

Sample	2, 3, 7, 8 TCDD	Total CDD	2, 3, 7, 8 TCDF	Total CDF
Spent GAC (ng/g)	ND*	ND	ND	0.17
Reactivated GAC (ng/g)	ND	ND	ND	ND
Scrubber water (ng/L)	0.09	0.18	0.03	0.20
Quench water (ng/L)	0.05	0.05	ND	ND
Drinking water (ng/L)	ND	ND	ND	ND
Gas phase (ng/m^3)	ND	0.79	0.068	0.44

Source: Koffskey and Lykins 1990.
*ND = not detected.

located in the drying zone, pyrolysis zone, and reactivation zone. After regeneration, the GAC was quenched and educted to the regenerated carbon storage tank.

The off-gases from the furnace were drawn through the afterburner, which was maintained at 1,850°F (1,010°C) for a residence time of 18 seconds. A small amount of air was introduced to aid combustion.

As was the case with the GCWW project, low levels of dioxin and furan were observed in the gas emissions from the regeneration facility. Because the source water and type of unit differed from those in Cincinnati, the levels of these contaminants were different. Effluent streams from the furnace were monitored for CDD and CDF when virgin and spent carbons were generated. However, during the regeneration of the spent GAC, CDD and CDF were found. The TOX of spent GAC that was exposed to chloraminated water was 460 mg/kg, whereas the reactivated virgin GAC contained <50 mg/kg TOX. Table 10.11 shows the CDD and CDF results for Jefferson Parish.

Low levels of CDDs were found in the gas phase of the stack emission. CDD concentrations of 0.18 and 0.05 ng/L were found in the scrubber water and quench water, respectively. Low levels of CDF were also found in the gas phases of the stack emission. CDF concentrations of 0.17 ng/g and 0.20 ng/L were found on the spent GAC and in the scrubber water. Although these contaminants were found in the emissions, the health risk associated with the levels observed was quite low. USEPA estimated that the risk posed by the CDD and CDF concentrations was about three in one billion.

In general, CDD and CDF concentrations were lower in the Jefferson Parish off-gas than in the GCWW's off-gas. One reason was the longer contact time of the afterburner, which was 18 seconds for the infrared regenerator and 3 seconds for the fluidized bed regenerator.

REFERENCES

AWWSC (American Water Works Service Company). 1998. Internal Report. Voorhees, N.J.: AWWSC.

Clark, R.M., and B. Lykins. 1989. *Granular Activated Carbon: Design, Operation, and Cost.* Chelsea, Mich.: Lewis Publishers.

Cornwell, D.A., and H. Koppers, eds. 1990. *Slib, Schlamm, Sludge*. Denver, Colo.: AwwaRF, AWWA, and KIWA.

Cornwell, D.A., C. Vandermeyden, G. Dillow, and M. Wang. 1992. *Landfilling of Water Treatment Plant Coagulant Sludges*. Denver, Colo.: AwwaRF and AWWA.

DeMarco, J., and R. Miller. 1985. Discovery and Elimination of Dioxins From a Carbon Reactivation Process. In *Proc. of the 1985 Annual Conference*. Denver, Colo.: AWWA.

DeMarco, J., R. Miller, and D. Hartman. 1988. Discovery and Elimination of Dioxins From a Carbon Reactivation Process. *Jour. AWWA*, 80(3):66–72.

GCWW (Greater Cincinnati Water Works). 1998. Unpublished internal report. Cincinnati, Ohio: GCWW.

Koffskey, W.E., and B. Lykins. 1987. Alternative Disinfectants and GAC Effects on Trace Organic Contaminants. EPA/600/52-87/006. Washington, D.C.: USEPA.

Koffskey, W.E., and B. Lykins. 1990. GAC Adsorption and Infrared Reactivation: A Case Study. *Jour. AWWA*, 82(1):48–56.

McTigue, N.E., and D. Cornwell. 1994. *The Hazardous Potential of Activated Carbons Used in Water Treatment*. Denver, Colo.: AwwaRF and AWWA.

Michael, J.I., J. Crittenden, and V. Snoeyink. 1991. Spent GAC Hazardous Waste Characterization Results for South Adams County Water and Sanitation District. In *Proc. of the AWWA Annual Conference*. Denver, Colo.: AWWA.

USDOT (U.S. Department of Transportation). 1976. 49 CFR Parts 171 and 172. Hazardous Material Transportation Uniform Safety Act. April 15, 1976. 41 *Fed. Reg*. 15996. Washington, D.C.: USDOT.

USEPA (U.S. Environmental Protection Agency). 1976. 40 CFR Part 760. Toxic Substances Control Act. Washington, D.C.: USEPA.

Wagner, T. 1990. *Hazardous Waste Identification and Classification Manual*. New York: Van Nostrand Reinhold.

Wentz, C. 1989. *Hazardous Waste Management*. New York: McGraw-Hill.

CHAPTER 11
MARKETING WATER TREATMENT PLANT RESIDUALS

Utilities have many options available to use the residuals that are produced by the purification processes in a beneficial way. The "benefit" can be not putting the residuals in a landfill, thereby saving valuable land space, or not discharging the residuals to a water course, thereby avoiding potential impacts. The benefits can also be more direct in the improvement or lowered cost of the end product, such as replacing topsoil in turf farm applications. Chapter 5 discussed many beneficial use options that have been practiced by utilities. In many cases, an aggressive, well-planned program is needed to implement these options. This chapter, adapted from the AwwaRF report titled *Commercial Application and Marketing of Water Plant Residuals* (Cornwell, Mutter, and Vandermeyden 2000) provides a guideline that can be used to investigate, develop, and implement a successful beneficial use program.

The marketing strategy guidelines include information concerning the following tasks, which are necessary for developing a program:

- Review of local and state regulations and history of beneficial use within the state
- Performance of residuals characterization
- Screening of potential beneficial use options
- Determining the requirements for performing beneficial use
- Assessment of existing and future required facilities
- Economic and noneconomic analysis
- Regulatory permitting
- Development of marketing package
- Marketing of residuals to potential end users
- Contractual agreements
- Project development and implementation
- Compliance sampling and monitoring (if required)

The guidelines presented in this chapter could be modified to be applicable to any of the different beneficial use markets.

REGULATORY EVALUATION

The first task that should be performed by a utility when initiating a beneficial use program is to determine if any state or local regulatory guidelines exist for beneficial use of water treatment residuals. Many states have previous experience dealing with residuals regulation, and some states have guidelines that could provide a framework of how to develop a beneficial use program. Some of the federal guidelines were discussed in chapter 5; however, state guidance will be the most important in determining allowable uses, application procedures, and so forth. It is best to involve the regulatory agency that will oversee the permitting process at the beginning of the project to determine which tasks must be accomplished for establishing a successful program. Regulatory involvement can be started by providing a written summary that outlines the goals and objectives of the project. This could be followed up by a face-to-face meeting with regulators for further discussion.

If regulatory guidelines for beneficial use exist, then an effort should be made to locate utilities that have received beneficial use permits in the past and review that utility's experiences. Nearby utilities that have experience with one or more forms of residuals beneficial use could provide invaluable information on how to develop a successful program or why a certain program did not succeed.

RESIDUALS CHARACTERIZATION

The next task necessary for marketing residuals for beneficial use applications is to sample residuals and perform a complete chemical and physical analysis of the contents. The chemical and physical properties of the residuals will ultimately dictate how a particular residual could be beneficially used in a safe manner. Accurate analysis of the residuals is critical, and a utility should be aware that residuals quantities could change seasonally. Listings of recommended physical and chemical parameters that should be analyzed are shown in Tables 11.1 and 11.2. Analysis of these parameters should provide enough information to initiate the marketing of residuals to end users. Also, compliance with regulatory guidelines can be assessed at this point. Because of seasonal changes in raw water quality and changes in treatment chemicals applied during different times of the year, quarterly sampling of residuals may be necessary to fully characterize the variations in the residuals quality. Even more samples would be required for any type of statistically valid analysis.

Selection of Potential Beneficial Use Options

The selection of potential beneficial use markets should be simplified after reviewing the market descriptions and case studies provided in the AwwaRF report, as well as by investigating the practices used by nearby water utilities (Cornwell, Mutter, and Vandermeyden 2000). Beneficial use or "turnkey" contractors, if available, may also provide insight as to which applications are most promising. Ultimately, a utility should select a short list of beneficial use applications that have the greatest potential based on the following topics:

- Is this alternative available within close proximity to the treatment plant?
- Will this application potentially meet regulatory acceptance?
- Are the potential costs attractive?
- Would the end of residuals be beneficial to this market?
- Is this alternative a realistic option based on information supplied by this book or experiences of other utilities?

If these questions can be answered positively, then the particular alternative in question should be included in the short list of potential beneficial uses.

After establishing a short list, a general search can be used to identify which particular end users exist. The goal should be to establish as many agreements as possible with potential end users in order to have a variety of outlets for beneficial use in case some are unsuccessful. In order to locate potential end users, the following techniques could be used:

Table 11.1
Physical test parameters useful for beneficial use

Use	Solids concentration (%)	Color	Texture	Soil aggregation	Moisture content (%)	Grain size analysis (clay/silt/sand) (%)	Liquid limit (% solids)	Plastic limit (% solids)	Mass density (lb/ft^3 [kg/m^3])	Specific gravity	Shrinkage (%)	Specific weight (lb/ft^3 [kg/m^3])	Shear strength (lb/ft^2 [kg/m^2])	Moisture retention (cm water/cm soil depth)
Land application	x	x	x	x	x	x						x		
Cement manufacturing	x				x	x			x	x		x		
Brick making	x	x			x	x			x	x	x	x		
Turf farming	x		x	x	x	x			x	x		x	x	
Composting	x		x	x	x	x			x	x		x		x
Soil production	x	x	x	x	x	x			x	x	x	x		
Road subgrade	x				x	x			x	x	x	x	x	
Forest land application	x	x	x	x	x	x						x		
Citrus grove application	x	x	x	x	x	x						x		
Nutrient control	x			x						x		x		
Landfill cover	x				x	x						x		
Land reclamation	x	x	x	x	x	x	x	x	x	x	x	x	x	
Hydrogen sulfide (H_2S) binding	x													

- Discussions with utility staff and with neighboring cities, towns, and villages
- Search of the yellow pages to determine what businesses exist within the area
- Contact with local municipal landfills and composting facilities
- Contact with national organizations for the different markets (see Cornwell, Mutter, and Vandermeyden 2000)
- Discussions with employees at a local nursery or lawn and garden center to determine where topsoil and potting soil products are manufactured
- Discussions with local county farm extension agents, or direct contact with farmers
- Contact with private turnkey companies

A series of phone interviews will help determine whether potential end users would be seriously interested in using residuals.

Table 11.2
Chemical test parameters useful for residuals beneficial use

	Nutrients							Metals			Radionuclides			Organics			Toxicity	Other tests	
Use	TKN (lb/ton [mg/kg])	Total phosphorus (lb/ton [mg/kg])	Potassium (lb/ton [mg/kg])	Ammonia–nitrogen (lb/ton [mg/kg])	Nitrate/Nitrite–N (lb/ton [mg/kg])	Calcium (lb/ton [mg/kg])	CCE (%)	Total metals (lb/ton [mg/kg])*	TCLP RCRA metals (mg/L)†	Metal oxides (lb/ton [mg/kg])‡	Gross alpha (pCi/g)	Gross beta (pCi/g)	Radium-226 (pCi/g)	TOC (lb/ton mg/kg)	TCLP volatiles/Semivolatiles (mg/L)†	Loss of ignition (%)	Phytotoxicity –Microtoxtest	Total coliform (no./g)	pH
Land application	x	x	x	x	x	x	x	x	x		x	x	x	x			x	x	x
Cement manufacturing						x		x	x	x	x	x	x	x	x				x
Brick making						x		x		x	x	x	x	x	x				x
Turf farming	x	x	x	x	x	x	x	x	x		x	x	x	x		x	x	x	x
Composting	x	x	x	x	x	x	x	x	x		x	x	x	x		x	x	x	x
Soil production	x	x	x	x	x	x	x	x	x		x	x	x	x			x	x	x
Road subgrade									x						x				
Forest land	x	x	x	x	x	x	x	x	x		x	x	x	x			x	x	x
Citrus grove	x	x	x	x	x	x	x	x	x		x	x	x	x		x	x	x	x
Nutrient control	x	x	x	x	x	x	x	x	x		x	x	x	x			x	x	x
Landfill cover								x							x				x
Land reclamation	x	x	x	x	x	x	x	x	x		x	x	x	x		x	x	x	x

*Total metals analyses include Al, As, Ba, Cd, Cr, Cu, Fe, Pb, Mg, Mn, Hg, Ni, Se, Ag, Zn, and Mo.
†TCLP analyses is as specified by 40 CFR Part 261 [USEPA 1983]. The RCRA metals include Ag, Ba, Cd, Cr, Ph, As, Se, and Hg.
‡These include major oxides of the following elements: Al, Si, Fe, Ca, Mg, S, Na, K, and Mn.

USER REQUIREMENTS

Prior to marketing residuals for a particular application, a utility must first determine the needs of the potential end user, storage requirements, and residuals characteristic requirements. Facility requirements must also be considered and could significantly impact project costs. A utility's goal should be to fully understand the potential user's operation, how the residuals could be incorporated, and which residuals characteristics would enhance the user's product or application. Some key issues to be addressed include

- What is the optimal or desired solids concentration, and is further dewatering such as air drying required?
- What are the residuals chemical and physical properties that are most important?

- Are any additions required, such as lime, fertilizers, or other additives?
- What quantity of residuals could the end user accept?
- Are residuals storage facilities and additional equipment required by the utility or the end user?

A complete evaluation of each of these questions should be performed to determine whether each alternative is feasible. The AwwaRF report referenced provides details on many of the processes and how understanding the end users of industry can help the utility be better prepared in its marketing efforts (Cornwell, Mutter, and Vandermeyden 2000).

FACILITY ASSESSMENT

After identifying the requirements that are necessary for each potential beneficial use, a complete facility assessment should be conducted to determine what, if any, modifications are necessary. A facility evaluation should be conducted for the water treatment facility as well as for the facilities used for performing the beneficial use. The facility assessment of the WTP should include the following tasks:

- Assessment of residuals dewatering facilities to determine if the capacity and extent of dewatering is acceptable
- Assessment of onsite residuals storage facilities to determine if enough space is available
- Assessment of existing equipment to determine if residuals handling, transportation, and application is feasible
- Assessment of driveways and roads to determine if road conditions are acceptable

Any facility modifications that may be required to perform a particular beneficial use plan could impact the overall project costs and, therefore, should be thoroughly evaluated prior to reviewing the project economics.

PRELIMINARY ECONOMIC ANALYSIS

The economics associated with a beneficial use program would be an important parameter for determining project feasibility. A utility must evaluate the costs associated with performing each potential beneficial use alternative to determine whether the beneficial use alternative is economical. The probable capital and viable operations and maintenance (O&M) costs for each potential alternative should be evaluated at this point. This would serve as a reality check to determine whether further pursuit of a beneficial use alternative is economically warranted. The alternatives should be compared to a base-case disposal option such as landfilling or the existing residuals disposal method the utility is using. The key cost elements that need to be considered are presented as follows.

Capital costs

- Residuals EQ basins
- Residuals thickeners
- Residuals dewatering equipment

- Residuals air drying facilities
- Residuals storage facilities
- Residuals blending equipment
- Equipment for residuals handling and transportation

Operating costs
- Dewatering
- Residuals handling and loading
- Transportation
- User fees
- Compliance sampling and analysis

NONECONOMIC ANALYSIS

A noneconomic analysis should also be conducted to evaluate nonmonetary considerations for each beneficial use plan. An example of a noneconomic analysis for rating various alternatives is shown in Table 11.3. Results from the noneconomic analysis will further assist a utility in screening the potential beneficial use options. The results from the noneconomic analysis should be linked with the beneficial use economics in order to confirm that beneficial use is economically and noneconomically attractive relative to a base-case alternative.

REGULATORY DISCUSSIONS AND REGULATORY REPORTING

If the economic and noneconomic assessments demonstrate that a particular beneficial use option is feasible and attractive, then the next task is to determine the specific tasks that must be completed to receive regulatory approval. At this point, a formal meeting with regulators should be conducted to discuss the specific information or applications needed for regulatory review. The potential beneficial use applications should be discussed as well as the potential users. The utility staff should inquire if the potential end users have a general permit and whether or not any permit violations have occurred in the past. This information will vary from state to state. Regulatory approval tends to focus primarily on the following issues:

- Residuals quality and quantity data
- Hazard potential of residuals
- Final use for residuals
- Who is the end user(s)
- Impacts on the natural environment

Residuals quality and quantity information, at this point, should be readily available. However, demonstration studies may be required to answer the questions of how residuals will be used and what impacts residuals will have on the environment.

In order to provide background information on requirements that a utility may encounter, permit requirements from three states were reviewed and summarized in the following sections. This information can provide guidance for working with state agencies on “typical” requirements for beneficial use options.

Table 11.3
Example of noneconomic evaluation of beneficial use alternatives

Weighting parameters	Weighting factor	Beneficial use alternatives																							
		Co-use with biosolids		Composting with yard waste		Turf farming		Topsoil blending		Potting soil		Landfill cover		Land application		Brick making		Cement manufacturing		Nutrient control		Road subgrade		Land reclamation	
		IS*	WS†	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS	IS	WS
Reliability	3	3	9	2	6	3	9	3	9	4	12	5	15	4	12	4	12	3	9	2	6	3	9	2	6
Flexibility	3	3	9	3	9	2	6	4	12	2	6	5	15	4	12	4	12	4	12	3	9	4	12	4	12
Permitting and regulatory impacts	3	3	9	2	6	4	12	4	12	4	12	3	9	3	9	4	12	4	12	3	9	4	12	2	6
Liability	2	4	8	2	4	3	6	3	6	2	4	4	8	3	6	4	8	4	8	3	6	4	8	2	4
User experience and time in business	3	3	9	3	9	4	12	3	9	4	12	4	12	4	12	5	15	5	15	2	6	4	12	3	9
Operational complexity	2	3	6	2	4	3	6	4	8	4	8	5	10	4	8	3	6	3	6	4	8	5	10	2	4
Expandability	2	4	8	4	8	3	6	4	8	4	8	3	6	4	8	3	6	3	6	5	10	3	6	4	8
Onsite land requirements	3	4	12	4	12	5	15	2	6	4	12	4	12	2	6	4	12	4	12	3	9	3	9	3	9
Compatibility with existing processes	2	4	8	4	8	4	8	3	6	4	8	5	10	5	10	4	8	4	8	3	6	4	8	3	6
Environmental impacts	3	4	12	4	12	3	9	3	9	3	9	4	12	4	12	3	9	3	9	5	15	3	9	5	15
Vehicle traffic	2	3	6	3	6	2	4	2	4	2	4	2	4	2	4	2	4	2	4	2	4	4	8	3	6
Disposal volume	2	4	8	4	8	3	6	4	8	5	10	5	10	4	8	5	10	5	10	5	10	5	10	5	10
Overall score		**104**		**92**		**99**		**97**		**105**		**123**		**107**		**114**		**111**		**98**		**113**		**95**	
Alternative ranking		**7**		**12**		**8**		**10**		**6**		**1**		**5**		**2**		**4**		**9**		**3**		**11**	

Source: Cornwell, Mutter, and Vandermeyden 2000.

*IS = Individual score (1 = least favorable, 2 = less favorable, 3 = neutral, 4 = favorable, 5 = very favorable).

†WS = Weighted score (1= less important, 2 = important, 3 = very important).

New Jersey

The New Jersey Department of Environmental Protection Bureau of Pretreatment and Residuals regulates beneficial use of WTP residuals using the New Jersey Pollutant Discharge Elimination System (NJPDES). Several water utilities in New Jersey hold NJPDES permits for beneficial use of WTP residuals. The NJPDES permit is structured into three parts, indicated as follows:

- Part I: Permit Conditions Relating to Domestic Residuals Management
- Part II: Additional Conditions for Distribution and Land Application
- Part III: Residuals Quality Monitoring Criteria

The permit document also includes guidance information for land application of WTP residuals as an attachment and several worksheets for (a) application site information, (b) property owner's certification, (c) blending operation information, and (d) example monitoring and reporting forms.

Part I of the NJDPES permit outlines the criteria for residuals management at the WTP in terms of generation, dewatering and storage, and offsite transportation. This section also lists the criteria that must be followed to update the permit when changing the treatment process or expanding the plant capacity.

Part II addresses the general conditions for land-applying WTP residuals. There are 10 different sections to the general conditions of the permit. These sections are listed as follows, and the key features of each section are further summarized in the text.

1. Definitions
2. General information
3. WTP residuals quality monitoring
4. Distribution conditions and procedures
5. Provisions for land application
6. Provisions for blending
7. Contingency plan
8. Conformance with district and state solid waste planning
9. Applicability
10. Record keeping and reporting

The permit provides definitions for WTP residuals, land application, and blending. The definition of land application is "controlled discharge of materials onto the soil or into the soil surface in such a manner that the materials are treated by and/or become incorporated into and mixed with the soil to enhance soil productivity."

A permit is not required for each application site. The permittee must certify the chemical and physical quality of the WTP residuals with the state to obtain reuse approval. If there are any changes in residuals quality over time, the state must be notified of these changes.

The residuals quality parameters, acceptance limits, and frequency of residuals sampling and analysis are included in Part III. Residuals must be collected at three locations for each 100-yd^3 (75-m^3) batch distributed. A 1-qt (1-L) sample volume should be collected from each location and blended together to generate a composite sample. Analysis must be conducted in a state-certified laboratory.

Prior to land applying or blending residuals at a location, a site information form, property owner's certification, and a copy of the Soil Conservation Service plan must be submitted. Records must be maintained for each application or blending site. If residuals are to be applied outside of New Jersey, then the utility must comply with the guidelines of that particular state.

The following criteria are outlined in the permit for land-applying WTP residuals:

- Application should not exceed 20 DT/year (18,140 kg/year).
- End users must be provided with instructional literature included in permit attachment.
- Residuals should not be applied within 100 ft (30 m) of surface water, 50 ft (15 m) of drainage ditches, and 50 ft (15 m) from property lines.
- Application is not allowed to saturated soils, to snow or ice cover, or frozen ground (unless slope is less than 5 percent, 75 percent vegetative cover exists, and soils are properly drained).
- No application will be permitted to sites with a slope exceeding 12 percent.
- Residuals must be incorporated into the soil within 24 hours unless slope is less than 5 percent or vegetative cover is at least 25 percent.
- Residuals should not be stockpiled on the following sites: within 200 ft (61 m) of surface water or buildings, within 50 ft (15 m) of drainage ditches and adjacent properties, on land with a slope greater than 5 percent, or on poorly drained soils; and residuals must not be stored more than 180 days.
- The quantity of residuals hauled to application site must not exceed the amount that can be applied.

Water treatment residuals can be blended at any ratio desired; however, the permit states that when residuals are blended at greater than 50 percent the user must comply with the regulations listed in the sections on "Distribution Conditions and Procedures" and "Provisions for Land Application." If the blend is less than 50 percent residuals, then the user is not subject to these conditions. When residuals cannot be applied for an extended period of time (i.e., because of weather), residuals must be managed in a legally permitted manner.

Utility beneficial use programs must comply with the Federal Water Pollution Control Act, New Jersey Water Pollution Control Act, and New Jersey Solid Waste Management Act or the permit may be terminated.

The permit includes standard forms for record keeping and reporting and includes dates for monitoring periods and report submission. Copies of these forms will be provided as an attachment to the memorandum in order to better understand the requirements.

The monitoring frequency, reporting requirement, and sample type necessary for each parameter under the NJPDES permit are provided in Table 11.4.

Pennsylvania

WTP residuals in Pennsylvania are classified as a solid waste by the Department of Environmental Resources (DER) Bureau of Waste Management and are regulated under the Solid Waste Management Act. The state has an established regulatory permitting program for WTP residuals beneficial use. These types of permits are referred to as a General Permit for Beneficial Use of Residual Waste and allow for use of residuals as "a soil additive on agricultural lands." The special conditions listed in this permit are summarized as follows:

Table 11.4
NJPDES water treatment residuals quality monitoring and reporting criteria

Parameter	Reporting requirements	Limits	Monitoring frequency	Sampling type
Residuals production (DT/d)	30-day monthly average	Report only	1/30 days	Calculated
pH	Monthly average	Report only	1/30 days	Composite
Total solids (%)	30-day monthly average	Report only	1/30 days	Composite
Total arsenic (mg/kg)	Maximum	75	1/30 days	Composite
	Monthly average	41		
Total cadmium (mg/kg)	Maximum	85	1/30 days	Composite
	Monthly average	39		
Total copper (mg/kg)	Maximum	4,300	1/30 days	Composite
	Monthly average	1,500		
Total lead (mg/kg)	Maximum	840	1/30 days	Composite
	Monthly average	300		
Total mercury (mg/kg)	Maximum	57	1/30 days	Composite
	Monthly average	17		
Total molybdenum (mg/kg)	Maximum	75	1/30 days	Composite
	Monthly average	Report only		
Total nickel (mg/kg)	Maximum	420	1/30 days	Composite
	Monthly average	420		
Total selenium (mg/kg)	Maximum	100	1/30 days	Composite
	Monthly average	100		
Total zinc (mg/kg)	Maximum	7,500	1/30 days	Composite
	Monthly average	2,800		
TKN (mg/kg)	Monthly average	Report only	1/30 days	Composite
Total phosphorus (mg/kg)	Monthly average	Report only	1/30 days	Composite
Total potassium (mg/kg)	Monthly average	Report only	1/30 days	Composite

NOTE: The NJPDES permit states that after 2 years of reporting, the permittee can petition the state to have the parameters and sampling frequency reduced.

1. Permit only allows for application on agricultural land as a soil additive. No definition of “soil additive” is provided, leaving this open for discussion.
2. The parameters and regulatory limits are as follows:
   a. pH 5.5–8.5
   b. Arsenic 41 mg/kg
   c. Cadmium 25 mg/kg

d.	Chromium	1,200 mg/kg
e.	Copper	1,500 mg/kg
f.	Lead	300 mg/kg
g.	Mercury	17 mg/kg
h.	Molybdenum	18 mg/kg
i.	Nickel	420 mg/kg
j.	Selenium	36 mg/kg
k.	Sodium	50 mg/kg
l.	Zinc	2,800 mg/kg
m.	PCBs	2 mg/kg
n.	Man-made inerts	4 mg/kg

These determinations are to be based on the 90 percent upper confidence level using *Test Methods for Evaluating Solid Waste Physical/Chemical Methods* (EPA SW-846; USEPA 1982). The analysis must be based on the requirements of EPA SW-846.

3. A farm conservation plan is required for application area.
4. Water treatment residuals cannot be mixed with other solid waste materials (i.e., hazardous waste, municipal waste, special waste).
5. No discharge to surface waters is permitted.
6. Must comply with fugitive emissions standards.
7. State reserves the right to inspect site without search warrant or prior notice.
8. Permit violations can result in revocation or suspension of permit.
9. Contractors are subject to compliance history review.
10. A permittee must obtain a “Determination of Applicability” prior to beginning operation by submission of application forms. In order to obtain this designation, a utility must provide the Bureau of Waste Management specific information as indicated by standard forms available from the state. The list of required information is included as an attachment to the memorandum.
11. Anyone operating under the permit must notify the bureau of Water Management.
12. For any new sources of WTP residual, the analytical data must be submitted.
13. Equipment must be properly maintained.
14. Residuals can only be stored for up to 7 days at application site, and the total amount stored should not exceed the 15 DT/acre/year (3.3 kg/m^2/year) loading rate multiplied by the number of acres on which residuals are applied. No free liquids can be present in stored residuals based on the paint filter liquid test (EPA SW-846) and no runoff is allowed from storage sites.
15. Residuals cannot be stored in areas with direct contact to ground- or surface waters, and storage and transportation should not threaten public health or safety.
16. Residuals data must be maintained at the permittee’s place of business for at least 5 years, and an annual report must be submitted that identifies the application sites and residuals loading.
17. Application cannot exceed 15 DT/acre/year (0.33 g/$m^3 \cdot$year) or 5,000 gal/acre/d (4.3 L/$m^2 \cdot$d)
18. At least one background analysis of the application site soil must be collected prior to land applying residuals. This must include pH, As, Cd, Cr, Cu, Hg, Mb, Na, Ni, Pb, Se, and Zn.
19. The soil pH must be maintained at pH 6.5 or higher.

20. No application will be permitted to slopes >15 percent, or during periods of rain, snow, or frozen ground.
21. Residuals should not be applied or stored 100 ft (30 m) from a perennial stream, 300 ft (91 m) from a water source, 25 ft (7.6 m) from a sinkhole, or applied to wetlands.

The Pennsylvania DER requires that all utilities must comply with these same general permit requirements in order to reuse residuals. This general permit structure simplifies the regulatory process for the Pennsylvania DER because of the standardization for residuals reuse procedures and forms.

Virginia

Virginia classifies WTP residuals as an industrial waste and regulates beneficial use of residuals on a case-by-case basis. A Virginia Pollution Abatement (VPA) permit from the DEQ is required for beneficial use applications. This permit allows the City of Newport News, Va., to land-apply alum residuals onto loblolly pine forests within their watershed. The three sections included in the VPA permit are listed as follows:

Part I: Monitoring requirements and special conditions
Part II: Requirements for monitoring and reporting
Part III: Management requirements

The VPA permit identifies the parameters that must be monitored for the residuals, residuals application rates, soils, and groundwater. The Waterworks permit lists the parameters that must be monitored as well as the limits, reporting units, sampling frequency, and sample type.

This permit does not provide a quality limit for each residuals parameter, but instead only requires that the city report residuals quality results. The parameters monitored include the following:

- Volume generated for reuse
- Percent solids, pH
- TKN, ammonia-N, nitrate-N, plant-available nitrogen (PAN)
- CCE
- Total-P, total-K
- Al, As, Cd, Cr, Cu, Hg, Mb, Ni, Pb, Se, Zn

Each of these parameters must be measured or calculated for every application event. Residuals loading is limited to 8.2 DT/acre/year (1.8 kg/m^3•year) to prevent overapplication. Copper and zinc were identified as the two limiting metals for land application, and, as a result, cumulative loading rates were set to prevent excessive accumulation of these metals in the watershed soils. Copper loading is limited to 67 lb/acre/year (7.5 g/m^2•year) or 1,340 lb/acre (150 g/m^2•year) total, while zinc loading is limited to 125 lb/acre/year (14 g/m^2•year) and 2,500 lb/acre (280 g/m^2) total.

The cumulative metals and nutrient loading is determined through the soils monitoring requirement. The soils monitoring criteria include

- Soil organic matter, cation exchange capacity, pH
- TKN, ammonia-N, available-P
- Exchangeable Al, Ca, K, Na, and Mg (mg/kg and meq/100 g)
- Available Cu, Fe, Mn, and Zn

All parameters are composite-sampled and analyzed twice per year, while the other parameters are measured once every 3 years. There are no limitations for these parameters except for Cu and Zn, and the soil pH must be maintained between 5.0 and 5.5.

The final monitoring requirement in the permit is for the application site groundwater wells. These groundwater monitoring parameters include the following:

- Water level
- pH
- Nitrate-N, ammonia-N, total-P, sulfate, ortho-P
- Al, Cu, and Zn
- TDS

Groundwater grab sampling and analysis is required once every 3 months from 12 different monitoring wells. Parameters are reported in milligrams per liter.

Part I of the permit also lists 12 special conditions for permit compliance. These conditions apply to residuals application, monitoring, and reporting. These permit elements are summarized as follows:

1. No discharge to surface water is allowed.
2. Storage and handling methods shall prevent discharge to surface waters.
3. A site map is required to display application sites.
4. An O&M manual must be developed that explains the residuals application practices and procedures.
5. Application is not allowed during inclement weather (i.e., ice or snow covered ground, excessive moisture, frozen soils).
6. Buffer zones:
    - 200 ft (61 m) from recreation areas
    - 200 ft (61 m) from drinking water supplies
    - 100 ft (30 m) from property lines, surface water, public roads
    - 25 ft (8 m) from power poles
7. Quarterly reports must be submitted to DEQ for monitored parameters listed above.
8. Annual vegetation must be monitored.
9. Annual project summary report must be submitted to DEQ with all data collected during the year.
10. Rate of application as specified and must be reported
11. Soil pH and liming requirements must be established
12. PAN loading rate must not be exceeded

The monitoring and reporting requirements for the permit are further defined in Part II. The key elements of this section of the permits are listed as follows:

- Sampling and analysis must be conducted according to 40 CFR 136, Guidelines for Establishing Test Procedures for Analysis of Pollutants, under the CWA (USEPA 1995).
- Results must include sample and analysis date, analytical technique, and lab identification.
- Records must be retained for 3 years from sample date, and any additional monitoring must be reported to DEQ.
- DEQ may require water quality information as necessary to determine the effects on water quality or to ensure that pollution of water does not occur.
- Reporting criteria are provided for compliance and noncompliance to DEQ. Any bypasses or spills must be reported within 24 hours.

The management requirements outlined in the permit identify the procedures for the following issues:

- Any changes in the quality of residuals generated or any changes to application protocol must be reported. This could include changes to water plant treatment process or chemical additions.
- Operator requirements and licenses required are specified.
- Quality control over the application program is outlined in the O&M manual. Facilities must be maintained and sludge stored to prevent water runoff.
- Must minimize adverse impacts on state waters, prevent bypasses, notify if planned bypass is unavoidable, or notify if an unplanned bypass occurs.
- State maintains right to enter the plant and application areas for inspection at reasonable times.
- Permits can be transferred to another person or contractor as long as DEQ is notified.
- Public access to land application information is required.
- Permit modifications can be made for necessary changes with approval of DEQ.
- Permit can be terminated for unauthorized releases, failure to disclose documents requested, endangerment of human health, etc.
- Permit does not relieve the permittee from civil and criminal liability regarding land application of residuals.

INFORMATIONAL PACKAGE

Data collected from previous tasks such as the residuals characterization, facility assessment, regulatory discussions, and information generation should be summarized into a simple, educational, and informative document that could be distributed to potential end users. The informational package could include other utility case studies, treatment plant process information, treatment chemicals used, and possibly a small sample of the residuals. This informational package should be very useful when meeting with potential end users. It would also provide the necessary information for the end user to understand the basic water treatment process and how the residuals could potentially benefit them.

MARKETING RESIDUALS TO POTENTIAL END USERS

At this point, a utility should have a good idea for which beneficial use alternatives are the most promising. The next task is to locate specific end users such as farmers, manufacturers, and others that are interested in using residuals. The goal at this point is to establish a relationship with several potential end users. The utility should meet with prospective end users to discuss residuals as well as tour the users' facilities. The utility should provide an explanation of what residuals are and review the informational package with the user. A utility may also want to cite other case studies where residuals have been used for the same purpose. Discussions with end users will provide the utility with a better understanding of how residuals could benefit the user and how residuals would be introduced into the process. All potential end users visited by the utility should also be invited to tour the WTP where the residuals are generated. This will allow potential users to see exactly how residuals are formed and handled prior to disposal.

WATER TREATMENT PLANT TOUR

A group tour of the WTP with all the potential users should be conducted at this point. The objective of this tour is twofold. First, it allows the utility to demonstrate to the potential users that the water treatment process is performed, monitored, and managed by a professional staff. Also, it would be a good educational opportunity to explain to the potential users the water treatment process and how the residuals are generated.

The second objective would be to let each potential user know that there are other users interested in the residuals. This could provide a higher comfort level to the potential users, recognizing that their competitors are also interested.

SCREENING OF POTENTIAL USERS

At this point, a final list of potential users that are seriously interested should be developed. It would also be useful to assess what type of contract or agreement each user would prefer and how that compares with the utility's legal counsel's requirements. Also, preliminary user fees should be established to update the capital and operating costs.

REGULATORY APPROVAL

With one or more acceptable users that would be interested in the residuals, the utility's remaining task would be to obtain regulatory approval. If the regulators were extensively involved throughout the development of a beneficial use program, obtaining regulatory approval should be relatively straightforward. The utility should coordinate an appropriate sampling and monitoring program with the regulators, if required. Sampling and monitoring programs would be determined on a case-by-case basis as directed by the regulatory agency overseeing the project. Also, a site visit with the regulator who will prepare the permit to the WTP and to the potential end users would be very helpful.

CONTRACTUAL AGREEMENTS

Utilities should recognize that writing the actual permit by the regulatory agency could be a detailed and time-consuming process, even if the regulators were involved throughout the development of the beneficial use plan. While the regulators are preparing the permit, it would be appropriate to prepare draft agreements between the utility and the user. An agreement should, at a minimum, address the following issues:

- Acceptable physical residuals characteristics
- Residuals quantity
- Frequency of delivery and time of day
- Measure of payment such as a certified scale or other measuring method
- Suitable storage areas at the user's facility that prevent runoff into the environment
- Inspection of the trucks when they arrive at the WTP
- Method of loading the trucks at the WTP
- Driver's responsibilities and conduct at the water plant site
- Minimum condition of the vendor's transportation equipment
- Monthly reporting parameters
- Ownership and liability of the residuals once loaded onto the vendor's trucks

The potential users should fully explain the intended beneficial use plan for the residuals, including the location where the beneficial use will take place. Also, the potential users should provide information on any portion of the service that may be subcontracted and the qualifications of the subcontractors. Finally, the users should be willing to provide access to the regulatory agencies to inspect the intended facility where the beneficial use will take place as part of the approval process.

PROJECT DEVELOPMENT AND IMPLEMENTATION

At this stage in the project it must be determined who will be responsible for what activities and when they are to be accomplished. It is also very important that the project manager be able to coordinate the activities of all of the participants to ensure a successful project. Once the project is initiated, it will require operational and financial monitoring. Some of the specific activities that must be addressed include the following:

- Identification of all utility project members and their responsibilities
- Identification of activities to be accomplished by outside firms or agencies
- Budgeting and funding of all tasks
- Development of necessary construction projects
- Determination of deadlines for accomplishment of various tasks
- Supervision of all tasks
- Startup of new processes and equipment
- Scheduling of residuals utilization based on plant production and needs of residuals customers
- Monitoring of residuals quality and status of utilization project
- Financial evaluation of project on an ongoing basis

COMPLIANCE SAMPLING AND MONITORING

Compliance sampling and monitoring will be determined by the regulatory permit requirements and/or the desires of the utility. An evaluation must be made to compare the need for project information with the cost of sampling and monitoring. Because of the potential liability of the utility from any customers utilizing the residuals, it may be beneficial to conduct additional monitoring, especially in the early stages of the project. Some elements of the sampling and monitoring program that may be included—depending on the beneficial use market selected for residuals disposal—are residuals characterization, background soil analysis, background plant tissue analysis, or other tests suggested by the report that would limit liability due to potential problems that are not caused as a result of residuals use in a particular market.

Reporting the results from project monitoring may be required by the regulatory agency involved. It may also be desirable to transmit the results from some or all of the monitoring to the residuals end users. Publication of the monitoring information may also be useful to other utilities who are considering beneficial use programs.

IMPLEMENTATION

Finally, the residuals are beneficially used. Although getting to this point can be time consuming, the economic and environmental benefits can be well worth the struggle.

REFERENCES

Cornwell, D.A., R.N. Mutter, and C. Vandermeyden. 2000. *Commercial Application and Marketing of Water Plant Residuals*. Final Report. Denver, Colo.: AwwaRF and AWWA.

USEPA (U.S. Environmental Protection Agency). 1982. *Test Methods for Evaluating Solid Waste Physical/Chemical Methods*. 2nd ed. Publication SW-846. Front Royal, Va.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1983. 40 CFR Part 261. Identification and Listing of Hazardous Waste. April 1, 1983. 48 *Fed. Reg*. 14293. Washington, D.C.: USEPA.

USEPA (U.S. Environmental Protection Agency). 1995. 40 CFR Part 136. Guidelines for Establishing Test Procedures for Analysis of Pollutants. Aug. 28, 1995. *Fed. Reg*. 44669. Washington, D.C.: USEPA.

ABBREVIATIONS

A	ampere
AA	activated alumina
ACF	adsorption clarifier flush
AEA	Atomic Energy Act
ALARA	as low as reasonably achievable
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
AWWA	American Water Works Association
AwwaRF	Awwa Research Foundation
AWWSC	American Water Works Service Company
BOD	biochemical oxygen demand
BPJ	best professional judgment
Btu	British thermal unit
bu	bushel
BV	bed volume
°C	degrees Celsius
CALWET	California Waste Extraction Test
CCE	calcium carbonate equivalent
CDD	chlorinated dibenzo dioxin
CDF	chlorinated dibenzo furan
CDH	Colorado Department of Health
CWD	Cleveland Division of Water
CEBW	chemically enhanced backwash
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
cfm/ft^2	cubic feet per minute per square foot
CFR	Code of Federal Regulations
cfs	cubic feet per second
cfu	colony-forming unit
CIM	composite ion-exchange materials
CIP	clean-in-place
cm	centimeter
CMF	coagulation–microfiltration
cm/min	centimeters per minute
cm/month	centimeters per month
COD	chemical oxygen demand
CST	capillary suction time
cu	color unit
CUWCD	Central Utah Water Conservancy District
CWA	Clean Water Act

d	day
DAF	dissolved air flotation
DBCP	dibromochloropropane
DBP	disinfection by-product
DC	direct current
D/DBP	disinfectants/disinfection by-products
DEQ	Department of Environmental Quality
DER	Department of Environmental Resources
DHS	Department of Health Services
DOC	dissolved organic carbon
DOT	Department of Transportation
DT	dry ton
DT/d	dry tons per day
d/week	days per week
DWTP	drinking water treatment plant
EBCT	empty bed contact time
ECWA	Erie County Water Authority
ED	electrodialysis
EDR	electrodialysis reversal
EPA	Environmental Protection Agency
EQ	equalization
EU	European Union
°F	degrees Fahrenheit
FBRR	Filter Backwash Recycling Rule
FGD	flue gas desulfurization
fps	feet per second
ft^3/kg	cubic feet per kilogram
$ft\text{-}lb/ft^3$	foot-pound per cubic foot
ft/min	feet per minute
g	gram; gravity
GAC	granular activated carbon
gal	gallon
gal/acre·day	gallons per acre per day
gal/ft^3	gallons per cubic feet
gal/mil gal	gallons per million gallons
g/cm^3	grams per cubic centimeter
GCWW	Greater Cincinnati Water Works
GFH	Granular Ferric Hydroxide
g/kg	grams per kilogram
g/m^3	grams per cubic meter
gpd	gallons per day
gpm	gallons per minute
gpm/ft^2	gallons per minute per square foot

GWRS	groundwater replenishment system
GWUDI	groundwater under the direct influence of surface water
ha	hectare
HAA	haloacetic acid
HAA5	the sum of five haloacetic acids
HAA6	the sum of six haloacetic acids
hr	hour
hr/d	hours per day
hr/month	hours per month
HMT	Hazardous Materials Table
HMTA	Hazardous Materials Transportation Act
ICP	inductively-coupled plasma
IFA	immunofluorescence assay
in.	inch
in./month	inches per month
IPP	Industrial Pretreatment Program
IX	ion exchange
kcal	kilocalorie
kcal/mol	kilocalories per mole
kg	kilogram
kg/d	kilograms per day
kg/ft^3	kilograms per cubic foot
kg/ha	kilograms per hectare
kg/m^2	kilograms per square meter
$kg/m^2{\cdot}d$	kilograms per square meter per day
kg/m•hr	kilograms per meter per hour
$kg/m^2{\cdot}hr$	kilograms per square meter per hour
$kg/m^2{\cdot}year$	kilograms per square meter per year
kg/m^3	kilograms per cubic meter
kg/ML	kilograms per megaliter
kg/sec^2m^4	kilograms per square second per meter to the fourth power
kg/year	kilograms per year
kW•hr	kilowatt-hour
L	liter
LACSD	Los Angeles County Sanitation District
lb	pound
lb/acre	pounds per acre
lb/d	pounds per day
lb/ft^2	pounds per square foot
$lb/ft^2{\cdot}d$	pounds per square foot per day
$lb/ft^2{\cdot}hr$	pounds per square foot per hour
$lb/ft^2{\cdot}year$	pounds per square foot per year

lb/ft^3	pounds per cubic foot
L/m^2·d	liters per square meter per day
lb/m·hr	pounds per meter per hour
lb/mil gal	pounds per million gallons
lb/ton	pounds per ton
L/d	liters per day
LEC	Lawrence Energy Center
LRF	lime recovery facility
L/sec	liters per second
m	meter
m/hr	meters per hour
m^3/year	cubic meters per year
MCL	maximum contaminant level
meq	milliequivalent
MF	microfiltration
mg	milligram
MG	million gallons
mgd	million gallons per day
mg/kg	milligrams per kilogram
mg/L	milligrams per liter
mi	mile
min	minute
ML	megaliters
ML/d	megaliters per day
mL/min	milliliters per minute
mmol	millimole
mol	mole
MOU	memorandum of understanding
mrem/year	millirems per year
mS/cm	millisiemens per centimeter
MSWLF	municipal solid waste landfill
MW	molecular weight
MWC	Manchester Water Company
µg/L	micrograms per liter
µm	micrometer
µrem/hr	microrems per hour
N	newton
NA	not available; not analyzed; not applicable
ND	not detected
NF	nanofiltration
ng	nanogram
ng/g	nanograms per gram
ng/L	nanograms per liter
ng/m^3	nanograms per cubic meter

NJPDES	New Jersey Pollutant Discharge Elimination System
N/m^2	newtons per square meter
NOM	natural organic matter
NORM	naturally occurring radionuclide materials
NPDES	National Pollutant Discharge Elimination System
NS	not sampled
N•sec	newton-second
N•sec/m^2	newton-seconds per square meter
ntu	nephelometric turbidity unit

OCSD	Orange County Sanitation District
O&M	operations and maintenance

PAC	powdered activated carbon
PAN	plant-available nitrogen
particles/mL	particles per milliliter
PC	particle count
PCB	polychlorinated biphenyl
PCE	tetrachloroethylene
pCi/g	picocuries per gram
pCi/L	picocuries per liter
POTW	publicly owned treatment works
psi	pounds per square inch
psig	pounds per square inch gauge
Pt–Co	platinum–cobalt (color unit)
PVC	polyvinyl chloride
PWD	Philadelphia Water Department

RCRA	Resource Conservation and Recovery Act
RO	reverse osmosis
rpm	revolutions per minute

SAC	strong-acid cation
SARI	Santa Ana Regional Interceptor
SBA	strong-base anion
SD	standard deviation
SDS	simulated distribution system
SDWA	Safe Drinking Water Act
sec	second
sec^2/g	square seconds per gram
SFBW	spent filter backwash water
SI	Système International
SOC	synthetic organic compound
sp gr	specific gravity
SR	specific resistance
SS	suspended solids

TBLL	technically based local limit
TCDD	tetrachlorinated dibenzo dioxin
TCDF	tetrachlorinated dibenzo furan
TCE	trichloroethylene
TCLP	toxicity characteristic leaching procedure
TDS	total dissolved solids
THM	trihalomethane
THMFP	trihalomethane formation potential
TIE	toxic identification evaluation
TKN	total Kjeldahl nitrogen
TOC	total organic carbon
TOX	total organic halides
TS	total solids
TSCA	Toxic Substances Control Act
TSD	transport, storage, and disposal
TSS	total suspended solids
TTF	time to filter
TTHM	total trihalomethanes
UF	ultrafiltration
UIC	Underground Injection Control
U.S.	United States
USDOT	United States Department of Transportation
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
USNRC	United States Nuclear Regulatory Commission
VOC	volatile organic compound
VPA	Virginia Pollution Abatement
W	watt
WAC	weak-acid cation
WBA	weak-base anion
WEF	Water Environment Federation
WET	whole effluent toxicity
wt	weight
WTP	water treatment plant
WWTP	wastewater treatment plant
yd	yard
yd^3	cubic yard
ZAVC	zero air voids curve
ZHE	zero-headspace extraction

INDEX

Note: *f.* indicates figure; *t.* indicates table.

Awwa Research Foundation

6666 West Quincy Avenue
Denver, CO 80235-3098 USA
P 303.347.6100
www.awwarf.org
email: info@awwarf.org

Sponsors Research
Develops Knowledge
Promotes Collaboration

1P-1.2M-91093-10/06-NH

ISBN 1-58321-415-1

9 781583 214152