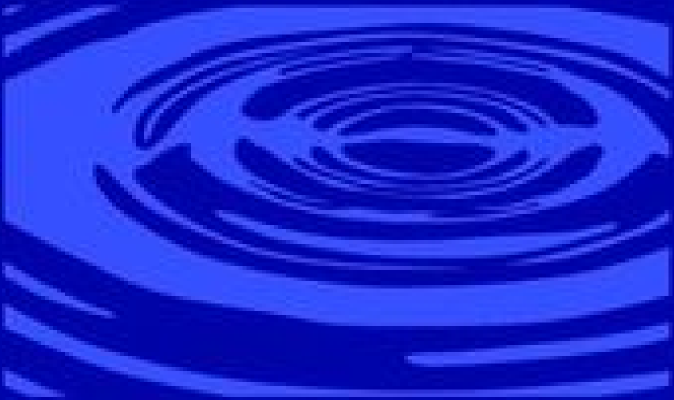
Handbook of Public Water Systems



Second Edition

HDR Engineering, Inc.

HANDBOOK OF PUBLIC WATER SYSTEMS

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CONTENTS

PREFACE

In 1986, we published the first edition of the *Handbook of Public Water Systems* under the Culp Wesner Culp corporate flag. HDR has since acquired Culp Wesner Culp. Now, 14 years later, we present the second edition. What has changed if...*there is no new thing under the sun?* Much has changed since that 1986 edition!

In 1986, *Giardia* was a worry; *Cryptosporidium* had yet to be recognized. High-pressure membranes (reverse osmosis) were used for demineralization, but low-pressure membranes were just coming on the scene and were too expensive for general application. In the intervening decade, advancements with both high- and low-pressure membranes have led to widespread application. Tube and lamella plate sedimentation and adsorption clarifiers were common in 1986, but the ballasted sedimentation concept was years away. Filtration design was more concerned with ''in-depth'' filtration, using relatively shallow filter depths (<36 inches) and filter rates of less than 5 gpm/sf, whereas many contemporary designs use 10- to 15-feet-deep filter beds and filtration rates of 10 gpm/sf or more.

Change has been driven by two factors: (1) a more competitive environment for water utilities, and (2) the need for better-quality water. Strong global economic forces are challenging every aspect of public service, including publicly owned and operated water utilities. Competition from private water companies is stimulating more efficient operation and management and improved technologies.

The desire for higher-quality water stems from the recognition of risk—and the need to minimize this risk—of adverse health effects from drinking water. The 1993 *Cryptosporidium* outbreak in Milwaukee caused near-panic in the public and the water industry. Pollutants such as radon and arsenic are identified as health risk problems but are expensive to remove using conventional technology.

Changes in water quality in distribution systems are of concern. Keeping water noncorrosive to distribution components and free of biological growths has become a more frequent challenge.

In response to drinking water health risk concerns, regulations are made more stringent. For example, the required finished water turbidity was 5 NTU in 1962. In 1982, the required turbidity was 1 NTU. The current (January 2000) standard is 0.5 NTU, 95 percent of the time. And many utilities have goals of 0.1 NTU. The future will require lower turbidities, restrictions on numbers of particles, and improved technologies. Even with improved technologies to treat water, greater emphasis will be placed on source water protection. Land use management, restrictions on contaminant sources, and improved nonpoint source runoff controls will likely be subjects of more and more regulations.

We can take pride in the safety of our drinking water; however, the future will require continually improved effort and technology in this area. Our industry has demonstrated the ability to respond to these challenges. In fact, change and improvements make this an exciting and memorable era in the drinking water industry.

The authors of this book are fortunate to have been involved in the development of several new and improved water treatment processes. They also have had the opportunity to incorporate these new methods into water treatment plant designs and to observe the results for full-scale plant operations. These experiences are heavily drawn upon in this book. The work of many other investigators using other new methods is also cited, in order to present the most accurate possible picture of currently available treatment methods.

This book is the result of the efforts of many individual members of the consulting engineering firm of HDR Engineering. Gordon Culp, a major contributor to the first edition, now with the firm of Smith Culp, also contributed. A list of the contributors to this handbook appears on page xiii. Their work is acknowledged with appreciation.

Henry H. Benjes, Jr.
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CHAPTER 1

Criteria and Standards for Improved Potable Water Quality

To be safe for human consumption, drinking water must be free from microorganisms capable of causing disease. It must not contain minerals and organic substances at concentrations that could produce adverse effects. Drinking water should be aesthetically acceptable; it should be free from apparent turbidity, color, and odor, and from any objectionable taste.[1]

THE HISTORY OF WATER TREATMENT

Ancient Practices

The quest for pure, abundant potable water is not a modern idea, as the beginnings of recorded history confirm. The Old Testament tells of the danger of ‘‘bitter’’ water and of a desperate search for life-sustaining, pure water. Sanskrit and Greek writings dating back 6,000 years describe early water treatment:

> Impure water should be purified by being boiled over a fire, or heated in the sun or by dipping a heated iron into it and then allowed to cool, or it may be purified by filtration through sand and coarse gravel.[2]

By 2000 B.C., the people of India were filtering water through charcoal and preserving it in copper pots. Paintings on Egyptian tombs dating to 1500 B.C. showed men filtering water using wick siphons. Egyptians at this time were also reported to have used the chemical alum to assist in settling particles and improve the clarity of water. Boiling water to render it potable and filtering water through sand and gravel have been prescribed for thousands of years. The health and medical benefits attributed to clean water have long been known. The ‘‘Father of Medicine,’’ Hippocrates (460–354 B.C.) wrote about the importance of water in maintaining public health and recommended that water be boiled and strained before drinking.

From these modest beginnings start the ever-evolving search for better, safer, and more plentiful drinking water supplies. But the transition from an individual household activity into a centralized, large-scale endeavor took many centuries. In 1685, a physician from Italy, Luc Antonio Porzio, published the first known illustration of sand filters, and by the mid-1700s, Frenchman Joseph Amy was granted a patent for a filtration system using sponges. Architect James Peacock was granted the first British patent in 1791 when he described a water filter using carefully placed graded layers of sand and gravel. Baker reports that the first town to have filtered water was Paisley, Scotland, in 1804—although the filtered water was transported to individual houses by carts. Three years later in Glasgow, filtered water was being piped to customers. By the early 1800s, centralized water treatment using sand filtration had been adopted by towns across Europe.

In 1799, Philadelphia, Pennsylvania, was the first U.S. city to build a public water system that distributed water through a system of pipes. Richmond, Virginia, on the James River, was the first town in the United States to build a centralized water treatment facility using sand and gravel filters in 1832, but it would take until the late 1800s for sand filtration to be widely adopted.

Filtration and Waterborne Disease

As advancements were being made in water filtration to improve primarily the aesthetics of drinking water, scientists were also beginning to understand the health significance of water. Dr. John Snow's landmark epidemiological studies linked, for the first time, contaminated water supplies as the causal agent in cholera outbreaks in London. In 1854, he concluded that a leaking sewer pipe had contaminated the water well located at No. 40 Broad Street, and when he removed the pump handle from the well, the outbreak subsided.

Dr. Snow also conducted detailed epidemiological studies on the water supplied by the Southwark and Vauxhall Company and the Lambeth Company. The former company obtained water from the Thames River in the middle of London, in an area polluted with sewage, whereas the latter obtained Thames River water upstream of London. Studies in an area served by both companies showed that the people receiving water from the Lambeth Company had a low incidence of cholera, whereas those served by the Southwark and Vauxhall Company had a high incidence. Because all other environmental factors were the same for both groups, Snow concluded that cholera was being spread by the water supply.

In the United States, typhoid was the disease of greatest public health concern. By the 1850s, typhoid outbreaks were common, causing thousands of deaths per year (see Fig. 1–1[3]). Although known to be highly contagious, the routes of exposure and infection were largely unknown. It wasn't until the late 1880s, when Louis Pasteur proposed his ''germ theory'' of disease, that scientists began to understand how microscopic organisms were responsible for sickness and disease.

Dr. Robert Koch provided the correlation between water filtration and protection against disease when, following an outbreak in 1892, he examined the incidence of cholera cases in the two German towns of Hamburg and Altona. Both of the contiguous cities drew water from the Elbe River. However, Altona filtered its water, since the water they withdrew was downstream of Hamburg and heavily contaminated. The results of Koch's study were conclusive. Despite Altona's having the more contaminated water source, the rate of cholera deaths in that downstream city was dramatically lower than in Hamburg. Water filtration, he concluded, was removing the bacteria that was causing the cholera outbreaks. The early works of Drs. Snow, Pasteur, and Koch are the foundation of modern water treatment and the protection of public health.[4]

With the link to waterborne disease (particularly cholera and typhoid) firmly established, improvements to public water supplies were soon to follow. To further study water filtration in the United States, the Massachusetts State Board of Health estab-

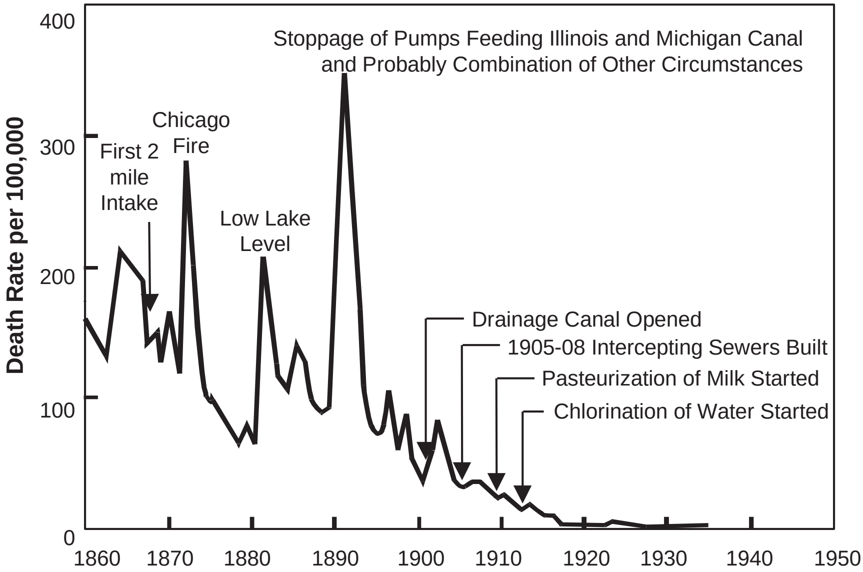


Fig. 1–1. Typhoid fever mortality in Chicago (1860–1950) (*Source:* References 3, 5)

lished an experiment station in Lawrence, Massachusetts, in 1887. While the Lawrence experiments were going on, the city was hit with a typhoid epidemic and a sand filter was installed in Lawrence. In 1893, Massachusetts Institute of Technology professor W. T. Sedgwick examined typhoid cases before and after the sand filter was installed and reported that the death rate from typhoid fever dropped 79 percent following the installation of the filter. Other filtration experiments were conducted at Louisville, Kentucky, in 1895 to 1897. By 1900, there were more than 3000 municipal water systems in the United States supplying water to customers. Philadelphia was one of the first large U.S. cities to adopt sand filtration in 1906, and by 1907 over 30 cities nationwide were doing so.

While filtration was found to be somewhat effective in *removing* disease-causing microbes (pathogens), dramatic improvements in the safety of water occurred due to the use of chlorine to disinfect or *kill* the pathogens. The first permanent chlorination plant was built in 1902 at Middelkerke, Belgium. Over the next few years, chlorination became standard practice in Great Britain. The first major city in the United States to chlorinate its water supply was Jersey City, New Jersey, in late 1908.[5] The dramatic effect of chlorination was a marked decrease of typhoid and other waterborne diseases. Early pioneers in water treatment included Abel Wolman, who in 1919 demonstrated that chlorine consumption varied widely from water to water and helped develop the concept of chlorine demand.

By the 1920s, use of sand filtration and chlorine disinfection in the United States defined ‘‘state-of-the-art’’ treatment and virtually eliminated major waterborne-disease outbreaks, such as typhoid and cholera. An example of how effective water treatment had become in reducing sickness is found in Wheeling, West Virginia. In 1917–18, the rate of typhoid fever was 150–200 cases per 100,000 population. Chlorination was added in 1918, and the rate of illness dropped to 25 per 100,000 population.[6]

Water Treatment Today

While there have been numerous technological advances since (such as new disinfectants, rapid filtration, etc.), this state-of-the-art treatment that was defined in the 1920s is, by and large, practiced today as conventional treatment. Adoption of conventional treatment throughout the United States has resulted in safe, dependable, and affordable drinking water in which the consuming public has fairly high confidence. Today in the United States, there are approximately 54,000 community water systems (CWSs) providing water to over 250 million people. America's high quality of public water supplies has traditionally been a source of local and national pride. Travelers drink water from the tap wherever they may be, with no question of its safety. Conformance to federally mandated drinking water quality standards has virtually eliminated the occurrence of waterborne diseases in this country; such diseases are not the serious problems that they are elsewhere in the world.

Despite this generally good record, water utilities using conventional treatment are not immune from waterborne-disease outbreaks (see Table 1–1). The most notable occurrence is the 1993 outbreak in Milwaukee, Wisconsin, where over 400,000 people were sickened with severe gastrointestinal upset due to cryptosporidiosis. Over 4,000 people were hospitalized, and, tragically, it is estimated that between 50 and 100 people died as a result of this illness.[7,8] The drinking water was contaminated with the protozoan parasite *Cryptosporidium parvum,* which has been found in the raw water of over 50 percent of the surface water sources in the United States. Of particular interest to water utility professionals is the fact that at no time during the Milwaukee outbreak did the utility violate any of the federal drinking water regulations in place at that time. But it remains a particular concern of all water utility professionals, public health officials, and treatment engineers that many community water systems are antiquated and do not meet present-day standards of quality and service. The public at large and numerous environmentally active groups are pushing for tighter controls and

TABLE 1–1. Significant Waterborne Disease Outbreaks in Community Water Systems

Year	State/Territory	Cause of Disease	No. of People Affected
1985	Massachusetts	*Giardia lamblia* (protozoan)	703 illnesses
1987	Georgia	*Cryptosporidium parvum* (protozoan)	13,000 illnesses
1987	Puerto Rico	*Shigella sonnei* (bacterium)	1,800 illnesses
1989	Missouri	*E. coli* 0157 (bacterium)	243 illnesses 4 deaths
1991	Puerto Rico	Unknown	9,847 illnesses
1993	Missouri	*Salmonella typhimurium* (bacterium)	650 illnesses 7 deaths
1993	Wisconsin	*Cryptosporidium parvum* (protozoan)	400,000 illnesses 50+ deaths
1998	Texas	*Cryptosporidium parvum* (protozoan)	1400 illnesses
1999	New York	*E. coli* 0157 (bacterium)	150 illnesses 1 death
2000	Ontario	*E. coli* 0157 (bacterium)	1000 illnesses 7 deaths

Source: Reference 1.

regulations concerning the treatment of water supplies to eliminate any possibility of the type of outbreak that occurred in Milwaukee.

TRENDS IN RAW-WATER QUALITY

Increased Water Withdrawals

Surface water sources are subject to ever-increasing withdrawals to supply a growing population, industry, and agriculture. Heavy withdrawals of water for consumptive use, such as irrigation, decrease stream flows available for downstream dilution of wastewaters. In the case of underground water supply sources, excessive withdrawals may have an adverse effect on the chemical quality of the supply, such as increasing the content of iron or manganese or of total dissolved solids.

Waste Discharges

Ever-increasing quantities of domestic, industrial, and agricultural wastewaters are being discharged directly or indirectly to water supply sources. Domestic sewage continues to add increasing quantities of bacteria, viruses, algal nutrients, substances with a high oxygen demand, suspended and dissolved solids, and taste- and odor-producing substances to water supply sources. Return irrigation waters in some locations are producing substantial increases in the total dissolved solids content of receiving streams and aquifers, and many of these substances, such as sodium sulfate, chloride, and others, are not removed by conventional treatment methods or other economically feasible means. Serious changes are also produced by industries that are discharging a wide variety of complex chemical pollutants to the waters of the nation.

Objectives in Water Treatment

The primary objective of water treatment for public supply is to take water from the best available source and to subject it to processing that will assure that it is always safe for human consumption and is aesthetically acceptable to the consumer. For water to be safe for human consumption, it must be free of pathogenic organisms or other biological forms that may be harmful to health, and it should not contain concentrations of chemicals that may be physiologically harmful. To provide safe water, the treatment plant must be properly designed and skillfully operated.

The general requirements of an aesthetically acceptable water are that it be cool, clear, colorless, odorless, and pleasant to the taste; also, it should not stain, form scale, or be corrosive. Treatment plants must be designed to produce water of uniformly good quality despite variations in raw-water quality and plant throughput. Because the consumer is interested in the quality of water at the tap rather than at the treatment plant, precautions must be taken to preserve water quality in the distribution system and to control water quality from tests of tap water samples as well as plant samples. Many of the advantages of a high-quality water supply are difficult to express in terms of exact economic return, but they are, without question, quite substantial. An excellent supply of high-quality water assures good public relations and favors industrial and community growth.

LEGISLATIVE AND REGULATORY HISTORY

Early Legislation

Legislation regarding the environment in the United States is common today and addresses virtually every aspect of our natural and our manmade environment. Early environmental legislation can be traced to 1872, when Congress enacted the Yellowstone Act to protect the unique landscape and habitat we know today as Yellowstone National Park. When President Nixon signed legislation that created the U.S. Environmental Protection Agency in 1970, the rate of new legislation increased dramatically. Figure 1–2 charts the growth in federal environmental legislation.

The first legislation passed by Congress indirectly addressing drinking water is considered to be the 1893 Interstate Quarantine Act. This law was in direct response to the scientific discoveries showing that such diseases as typhoid fever were highly contagious and could be transmitted by individuals from foreign countries to the United States or from state to state. Under this law, the surgeon general of the U.S. Public Health Service (USPHS) was given the authority to:

> . . . make and enforce such regulations as in his judgement are necessary to prevent the introduction, transmission, or spread of communicable disease from foreign countries into the states or possessions, or from one state or possession to any other state or possession.

Under the authority of this act, the USPHS developed the first drinking water–related regulation. The regulation prohibited the use of a common drinking cup by passengers on commercial transportation carriers traveling between states, such as ships and trains.[9,10]

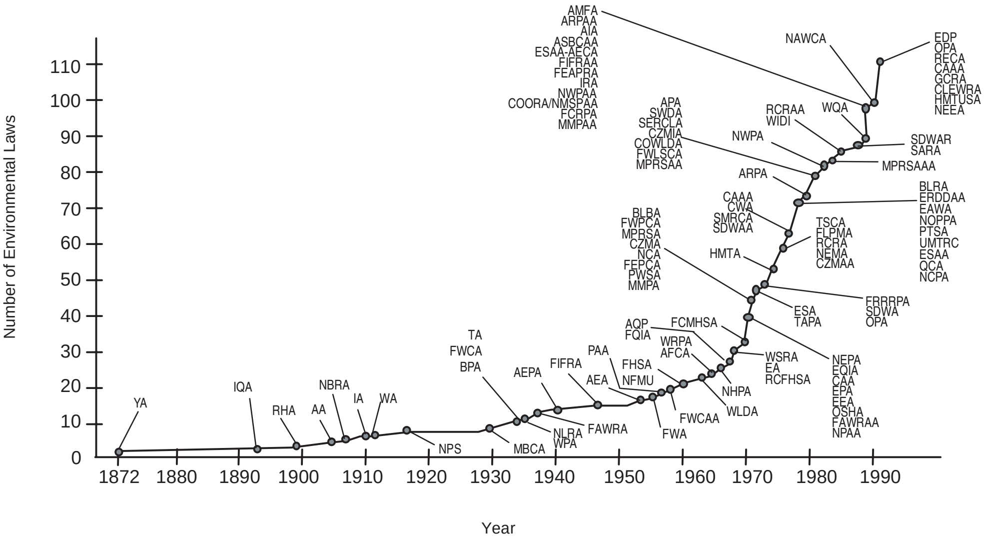


Fig. 1–2. Growth of federal environmental law in the United States (see the list of acronyms on pages 26–29 for the complete name of each law)

Early Regulations

From this modest beginning, drinking water standards in the United States began to develop. As water scientists and engineers developed a more thorough understanding of treatment technology and the link to waterborne disease, there was also an increased desire to protect the public from contaminated water. During the early 1900s, the USPHS developed new regulations that generally kept pace with the advancements in water treatment science. The first federal drinking water standard for bacteriological quality was developed in 1914 and applied to only interstate carriers. The Public Health Service revised and expanded these standards in 1925, and by 1942 the standards had become the basis for water quality regulations in the United States. The early regulations included bacteriological sampling in the distribution system and maximum permissible levels for lead, fluoride, arsenic, and selenium. Chromium was added to the list in 1946, which completed the early set of mostly nonenforceable guidelines for the country’s 19,000 water suppliers.

1962 Standards

By 1962, the list of mandatory regulations had expanded again, including both *mandatory* requirements as well as *recommended* requirements. Virtually all 50 states adopted these standards either as regulations or as guidance, even though they legally still applied to only interstate carriers.[9] The standards set mandatory limits for certain chemical constituents and recommended concentrations for others, including some radioactive elements. The bacteriological standards set limits for coliform organisms and prescribed methods for the collection and laboratory analysis of water samples, including the frequency thereof.

The minimum number of water samples per month to be collected for bacteriological examination varied according to population served by the system from 2 per month for 2,000 or fewer people, to 100 per month for 100,000 people, to 300 per month for 1,000,000 persons. The fact that the total number of bacteriological samples collected and analyzed was often less than the minimum required for significant results was one of the most common reasons for failure to meet the Drinking Water Standards.

In 1969, the USPHS’s Bureau of Water Hygiene surveyed water supply systems across the country to determine how the well the nation’s drinking water compared to the 1962 standards. This Community Water Supply Study (CWSS) surveyed 969 public water systems and found that over 40 percent of the systems did not meet the 1962 standards. Table 1–2 summarizes the 1962 mandatory requirements, and Table 1–3 summarizes the 1962 recommended standards.

1974 Safe Drinking Water Act

In the early 1970s, advances in analytical chemistry enabled scientists to begin looking for contaminants in drinking water in the then unbelievably sensitive “parts-per-million” range. And, in some cases, they didn’t like what they saw. While generally safe from microbial pathogens, some drinking water they examined contained trace levels of compounds not occurring naturally in water supplies. Some were manmade chemicals that had either leached or discharged into both groundwater and surface water sources and were not being removed by conventional treatment methods. A 1972

TABLE 1–2. Summary of Mandatory 1962 U.S. Public Health Service Drinking Water Standards

Constituent	Limit (mg/L except as shown)	Range of Concentrations Found in CWSS Survey* (mg/L except as shown)
Arsenic (As)	0.05	<0.03–0.10
Barium (Ba)	1.0	0–1.55
Boron (B)	5.0	0–3.28
Cadmium (Cd)	0.01	<0.2–3.94
Chromium (hexavalent)	0.05	0–0.079
Coliform organisms	1/100 ml	2,000/100 ml
Cyanide (CN^{-2})	0.10	0–0.008
Fluoride (F^{-})	Varies, †0.8 to 1.7	<0.2–4.40
Gross beta activity	1,000 $\mu\mu$c/L	154 $\mu\mu$c/L
Lead (Pb)	0.05	0–0.64
Selenium (Sc)	0.01	0–0.07
Silver (Ag)	0.05	0–0.03

* 1969 Community Water Supply Study by USPHS. 2,595 distribution samples.

† Dependent on annual average of maximum daily air temperatures.

TABLE 1–3. Summary of Recommended 1962 U.S. Public Health Service Drinking Water Standards

Constituent	Limit (mg/L except as shown)	Range of Concentrations Found in CWSS Survey* (mg/L except as shown)
Alkyl benzene sulfonate (ABS)	0.5	0–0.41
Arsenic (As)	0.01	0–3.28
Carbon chloroform extract (CCE)	0.200	0.0008–0.56
Chloride (Cl^{-})	250	<1.0–1,950
Color	15 units	49 units
Copper (Cu)	1.0	0–8.35
Cyanide (CN^{-})	0.01	0–0.008
Fluoride (F^{-})	Varies, 0.8 to 1.7	<0.2–4.40
Iron (Fe)	0.3	26.0
Manganese (Mn)	0.05	1.32
Nitrate (NO_3)	45	0.1–127.0
Phenols	0.001	
Radium-226	3 pCi/L	0–135.9 pCi/L
Strontium-90	10 pCi/L	0–1.0 pCi/L
Sulfate (SO_4)	250	<1–770
Zinc (Zn)	5	0–13.0
Total dissolved solids (TDS)	500	2,760
Odor, threshold number	3	
Turbidity		
Chlorination treatment only	5 NTU	53 NTU
Water treatment plants	1 NTU	

* 1969 Community Water Supply Study by USPHS, 2,595 distribution system samples.

report investigating the water quality of the Mississippi River revealed a total of 36 organic compounds found in the treated drinking water in New Orleans. This alarmed not only the water utilities but also the public, environmental activists, and Congress.[11]

Although health effects of these compounds were largely unknown, these "new" discoveries, in addition to the results of the 1969 CWSS Survey, led Congress to enact the original Safe Drinking Water Act (SDWA) of 1974. On December 16, 1974, President Ford signed into law the SDWA Public Law 93-523 (PL 93–523). This act gave the administrator of the fledgling U.S. Environmental Protection Agency (USEPA) the authority to control the quality of the drinking water in public water systems through the development of regulations, or by other methods. The act required a three-stage mechanism to establish comprehensive regulations (and standards) for drinking water quality:

- National Interim Primary Drinking Water Regulations (NIPDWRs) were to be promulgated to protect the public health. This would be accomplished using generally available technology and treatment techniques.
- The National Academy of Sciences (NAS)[12] was to conduct a study on the human health effects of exposure to contaminants in drinking waters. This study would "consider only what is required for protection of the public health, not what is technologically or economically feasible or reasonable."[12]
- Revised National Primary Drinking Water Regulations were to be promulgated, based on the NAS report, establishing maximum contaminant levels, to be set at levels sufficient to prevent the occurrence of any known or anticipated adverse health effects with an adequate margin of safety.

From 1975 to 1983, the USEPA promulgated interim regulations based on the 1962 Public Health Service Standards included in Tables 1–2 and 1–3. Some of the standards were revised by the EPA Advisory Committee on the Revision and Application of the Drinking Water Standards. The first set of interim regulations became effective on June 24, 1977, and contained maximum contaminant levels (MCLs) and monitoring requirements for microbiological contaminants, ten inorganic chemicals, six organic compounds, radionuclides, and turbidity.

Disinfection By-products

An additional interim regulation was adopted on November 29, 1979, regulating a new class of four contaminants: trihalomethanes (THMs). Researchers at USEPA and the Netherlands had discovered in 1974 that THMs were formed in drinking water as a result of chlorinating water that contained natural organic matter.[13] The four THM compounds include chloroform, bromoform, bromodichloromethane, and dibromochloromethane and are regulated together as Total THMs (TTHMs), or the sum of the concentration of each compound. At that time, chloroform was a suspected human carcinogen, and by 1976 the U.S. Food and Drug Administration (USFDA) had banned its use in all drugs. Thus, by 1979, the National Interim Primary Drinking Water Regulations were complete.[11] These are shown in Table 1–4.

In 1979, USEPA also set regulations for a set of non-enforceable standards. These National Secondary Drinking Water Regulations (NSDWRs) provide MCLs and guidance to public water suppliers regarding contaminants that may cause aesthetic

TABLE 1–4. 1979 National Interim Primary Drinking Water Regulations

Contaminant	MCL
Inorganic	
Arsenic	0.05 mg/L
Barium	1.0 mg/L
Cadmium	0.01 mg/L
Chromium	0.05 mg/L
Fluoride	
At ≤53.7°F (≤12.0°C)*	2.4 mg/L
At 53.8–58.3°F (12.1–14.6°C)	2.2 mg/L
At 58.4–63.8°F (14.7–17.6°C)	2.0 mg/L
At 63.9–70.6°F (17.7–21.4°C)	1.8 mg/L
At 70.7–79.2°F (21.5–26.2°C)	1.6 mg/L
At 79.3–90.5°F (26.3–32.5°C)	1.4 mg/L
Lead	0.05 mg/L
Mercury	0.002 mg/L
Nitrate (as N)	10.0 mg/L
Selenium	0.01 mg/L
Silver	0.05 mg/L
Organic	
Endrin	0.0002 mg/L
Lindane	0.004 mg/L
Toxaphene	0.005 mg/L
Methoxychlor	0.1 mg/L
2,4-D	0.1 mg/L
2,4,5-TP (Silvex)	0.01 mg/L
TTHM	0.1 mg/L
Radiological	
Alpha Emitters	
Radium-226	5 pCi/L
Radium-228	5 pCi/L
Gross alpha activity (excluding radon and uranium)	15 pCi/L
Beta and Photon Emitters†	
Tritium	20,000 pCi/L
Strontium-90	8 pCi/L
Turbidity (NTU)	1 NTU‡

Source: Reference 6.

* Average annual maximum daily air temperature.

† Based on a water intake of 2 L/d. If gross beta particle activity exceeds 50 pCi/L, other nuclides should be identified and quantified on the basis of a 2-L/d intake.

‡ One turbidity unit based on a monthly average. Up to 5 NTU may be allowed for the monthly average if it can be demonstrated that no interference occurs with disinfection or microbiological determination.

problems in drinking water. Although the secondary standards are not enforced at the federal level, some state regulatory primacy agencies have adopted them as additional *primary* standards, and they are therefore enforced at the state level. A complete listing of the 1979 secondary standards (as well as three additional secondary standards) is in Table 1–10.

1986 Safe Drinking Water Act Amendments

When originally passed in 1974, the SDWA required USEPA to:

- Set interim drinking water standards.
- Conduct (through the National Academy of Sciences [NAS]) an assessment of the risks of exposure to drinking water contaminants.
- Revise the interim standards to reflect the NAS findings.

The NAS study was completed in 1977 and became the basis for several amendments and reauthorizations to the act. The SDWA was amended in 1977, 1979, and 1980. However, USEPA was slow to promulgate the revised standards.

Frustrated by the lack of progress and concerned by continued reports of detecting organic and microbial contamination, Congress enacted sweeping changes in 1986. The SDWA amendments of 1986, placed EPA under stringent time schedules to promulgate regulations for a long list of contaminants (that may or may not have been likely found in drinking water). The major provisions of the 1986 amendments included:[12]

- Mandatory standards for 83 contaminants by June, 1989.
- Mandatory regulations of 25 contaminants every three years.
- National Interim Primary Drinking Water Regulations were declared National Primary Drinking Water Regulations (NPDWRs).
- Recommended maximum contaminant level (RMCL) goals were replaced by maximum contaminant level goals (MCLGs).
- Required designation of best available technology for each contaminant regulated.
- Specification of criteria for deciding when filtration of surface water supplies is required.
- Disinfection of all public water supplies with some exceptions for groundwater that meet, as yet, unspecific criteria.
- Monitoring for contaminants that are not regulated.
- A ban on lead solders, flux, and pipe in public water systems.
- New programs for wellhead protection and protection of sole-source aquifers.
- Streamlined and more powerful enforcement provisions.

As a result of the 1986 amendments (and encouraged by numerous lawsuits by the Bull Run Coalition), EPA did indeed move quickly to regulate the remaining contaminants on the list of 83 (23 of which were already promulgated by 1986). However, limited resources within the agency and several controversial proposals (e.g., radon) made it difficult for EPA to fully meet the congressional mandates. The statutory provision to regulate an additional 25 contaminants every three years (supposedly in perpetuity) strongly suggested that EPA could never get caught up. Figure 1–3 shows EPA's progress in regulating the number of individual contaminants required by the 1986 amendments.

1996 Safe Drinking Water Act Amendments

The implementation schedules in the 1986 amendments proved to be unrealistic for EPA to set using credible science. Thus, in 1991, when the congressional authorization for implementing the 1986 SDWA amendments expired, the act was not automatically reauthorized. For the next two years, little progress was made in determining the appropriate direction for the next version of the SDWA. However, in 1993, discussions began with renewed interest following the major outbreak of disease caused by *Cryptosporidium* in Milwaukee. Congress made substantial revisions to the SDWA in the form of the 1996 amendments of the SDWA.

The new amendments represented a major departure from the ‘‘regulatory treadmill’’ of the former act. Emphasis was now placed on regulating a smaller number of contaminants and focusing on those that had the highest risk to public health. Research funding was also increased to ensure that the best possible science would be incorporated into each rulemaking. Additional provisions included the requirement for EPA to report the cost of each regulation as it compares to the benefits of increased public health protection. The act also incorporated numerous provisions for public participation in drinking water issues, and it provided significant funding to create and capitalize a new State Revolving Fund to assist public water systems in financing drinking water improvements. Provisions of the 1996 amendments include:[10]

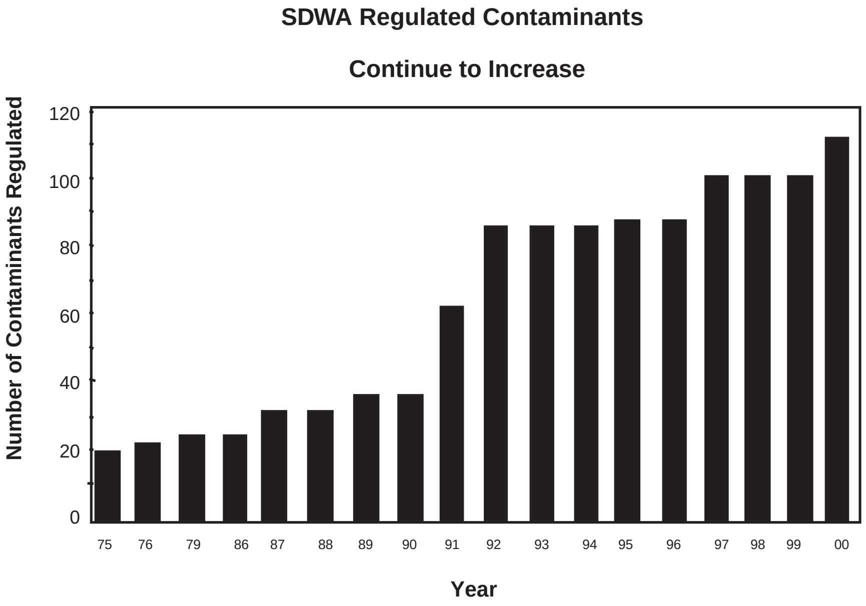


Fig. 1–3. Regulation development progress

- Repeal of the 1986 requirement that 25 contaminants be regulated every three years.
- Revision of the process for listing of contaminants for possible regulation.
- Revisions of the standard setting process to include consideration of cost, benefits, and competing health risks.
- New programs for source water assessment, local source water petitions, and source water protection grants.
- Mandatory regulation of filter backwash water recycle.
- Specified schedules for regulation of radon and arsenic.
- Revised requirements for unregulated contaminant monitoring and a national occurrence database.
- Provisions creating a state revolving loan fund (SRLF) for drinking water.
- New provisions regarding small system variances, treatment technology, and assistance centers.
- Development of operator certification guidelines by USEPA.

Under the 1996 amendments, many specific contaminant limits have been established; these are discussed in the following sections of this chapter.

APPROACH TO CONTAMINANT LIMITS

EPA is required under the 1996 amendments to establish Maximum Contaminant Level Goals (MCLGs) and National Primary Drinking Water Regulations (NPDWRs) for contaminants that have a known adverse effect on health, are likely to occur in public water systems at a frequency and level that are of health concern, or present an opportunity to health risk reduction. EPA may also establish National Secondary Drinking Water Regulations (NSDWRs) based on aesthetic rather than health concerns. NSWDRs are not enforceable standards.

MCLGs are nonenforceable, health-based goals set at levels at which no known adverse effect on health occurs and that allow for an adequate margin of safety. EPA must also establish a Maximum Contaminant Level (MCL) that is an enforceable standard. MCLs must be set as close to the MCLG as feasible, taking into account the best available treatment (BAT) and costs. Treatment methods for contaminants are discussed in Chapter 6, ‘‘Specific Contaminant Removal Methodologies.’’ In relation to contaminants that are potential carcinogens, three categories were established:[12]

> *Category I.* Contaminants for which sufficient evidence of carcinogenicity in humans and animals exists to warrant a carcinogenicity classification as ‘‘known probably human carcinogens via ingestion.’’
> *Category II.* Contaminants for which limited evidence of carcinogenicity in animals exists and that are regulated as ‘‘possible human carcinogens via ingestion.’’
> *Category III.* Substances for which insufficient or no evidence of carcinogenicity via ingestion exists.

MCLGs are set at zero for Category I contaminants. Because zero is not measurable or achievable using BAT, the MCLs are usually set at either the practical quantitation level or at a level achievable using BAT. The resulting MCL typically falls within the relative risk range of 1 in 10,000 to 1 in 1,000,000. An MCL for a Category II or III contaminant is often identical to the contaminant’s MCLG.

EPA may require the use of a treatment technique instead of establishing an MCL if it is not technically or economically feasible to monitor for the contaminant, such as for viruses and parasites.

CURRENT DRINKING WATER STANDARDS

Inorganics

Table 1–5 summarizes the MCLs, health effects, and monitoring requirements for inorganic contaminants. Refer to Chapter 2, ‘‘Inorganic and Radionuclide Contaminants,’’ for further discussion of health effects and international standards for these same contaminants.

The action levels for lead and copper shown in Table 1–5 are measured at the tap and must be met in 90 percent of the samples. All systems that exceed the lead or copper action limits must provide corrosion control and public education. See Chapter 21, ‘‘Distribution System Corrosion: Monitoring and Control,’’ for a discussion of corrosion control.

Organics

Table 1–6 summarizes the MCLs, health effects, and monitoring requirements for organic contaminants in the two categories of Synthetic Organic Compounds (SOCs) and Volatile Organic Compounds (VOCs). Chapter 3, ‘‘Organic Compounds in Drinking Water,’’ provides a more detailed discussion of these compounds. At the time of this writing, the MCLs for aldicarb and metabolites were pending reproposal.

Microbial Contaminants

Table 1–7 summarizes the MCLs, health effects, and monitoring requirements for microbial contaminants. Chapter 4, ‘‘Microbiology of Drinking Water,’’ presents a more detailed discussion of these contaminants.

EPA does allow variances to the coliform rule for nonpathogenic biofilms in the distribution system. Biofilms are discussed further in Chapter 22, ‘‘Water Quality Control in Distribution Systems.’’

Under the 1989 Surface Water Treatment Rule, filtered water turbidities must be maintained at less than or equal to 0.5 NTU in 95 percent of the samples collected each month. The 1998 Interim Enhanced Surface Water Treatment Rule (IESWTR) requires that surface water or groundwater systems under the influence of surface waters that serve 10,000 or more must produce a turbidity of less than or equal to 0.3 NTU in 95 percent of the measurements taken each month. The Long Term 1, Enhanced Surface Water Treatment Rule was proposed by EPA in April 2000 and applies the same turbidity requirement to smaller systems (serving less than 10,000) as the IESWTR (i.e., 0.3 NTU—95% of samples). This rule is expected to be promulgated in early 2001. If the water system meets these turbidity criteria, it is assumed to achieve the required two-log *Cryptosporidium* removal. Also, all systems are required to maintain a minimum disinfectant residual above 0.2 mg/L entering the distribution system.

TABLE 1–5. MCL, Health Effects, and Monitoring Requirements for Inorganic Chemicals

Name of Contaminant	Maximum Contaminant Level (MCL) (mg/L unless noted)	Health Effects of Contaminant	Monitoring Requirements
Antimony	0.006	Decreased longevity, blood effects	Once per year for surface waters. Once every three years for groundwater for IOCs, not including Asbestos, Lead and Copper, Nitrate, and Radionuclides
Arsenic[1]	0.05	Dermal and nervous system toxicity effects; bladder and kidney cancer	
Asbestos (fiber length > 10 μm)	7 MFL	Lung tumors/cancer risk	Asbestos. Once every nine years
Barium	2	Heart/cardiovascular system effects	
Beryllium	0.004	Bone/lung effects/cancer risk	
Cadmium	0.005	Kidney effects	
Chromium (total)	0.1	Kidney/nervous system/circulatory effects	
Copper	Treatment technology (Action Limit = 1.3)	Gastrointestinal effects	Copper: Same as lead
Cyanide	0.2	Spleen/brain/liver effects	
Fluoride	4	Skeletal effects	
Gross alpha Emitters	15 pCi/L	Cancer risk	Gross alpha and beta: Every four years—quarterly samples
Gross beta Particle and Photon Emitters	4.0 mrem	Cancer risk	
Lead	Treatment technology (Action Limit = 0.015)	Cancer risk/kidney/nervous system effects; highly toxic to infants	Lead: Follow-up monitoring is every six months after corrosion controls are initiated or optimized. Systems consistently meeting ALs can reduce monitoring to annually and then to every three years.
Mercury (inorganic)	0.002	Kidney/central nervous system disorder	
Nickel	0.1	Nervous system/Liver/heart effects	
Nitrate (as N)	10	Methemoglobinemia (blue baby syndrome)	Nitrate: Groundwater annually; surface water quarterly
Nitrite (as N)	1	Methemoglobinemia (blue baby syndrome)	Nitrite: One sample every three years
Radium 226 plus 228	Current 5 pCi/L (combined)	Cancer risk	Radium: Every four years—quarterly samples
Selenium	0.05	Nervous system/kidney effects	
Thallium	0.002	Kidney/liver/brain/intestine effects	

[1] On June 22, 2000, EPA proposed a lower MCL for arsenic, of 0.005 mg/L. Rule is expected to be final by June 22, 2001.

TABLE 1–6. MCL, Health Effects, and Monitoring Requirements for Organic Chemicals

Name of Contaminant	Maximum Contaminant Level (MCL) (mg/L unless noted)	Health Effects of Contaminant	Monitoring Requirements
Synthetic Organic Compounds			
2,3,7,8-TCDD (Dioxin)	0.00000003	Cancer risk	For SOCs: Original monitoring required four quarterly samples every three years. After one round of no detects; systems >3,300 reduce to two samples per year every three years. Systems ≤3,300 reduce to one sample every three years. Monitoring may be reduced or eliminated based upon results of vulnerability assessment.
2,4,5-TP (Silvex)	0.05	Liver/kidney effects	
2,4-D	0.07	Nervous system/liver/kidney effects	
Acrylamide	Treatment technology	Cancer risk/nervous system effects	
Alachlor	0.002	Cancer risk	
Aldicarb	0.003	Nervous system effects	
Aldicarb Sulfone	0.002	Nervous system effects	
Aldicarb Sulfoxide	0.004	Nervous system effects	
Atrazine	0.003	Cardio/reproductive system effects	
Carbofuran	0.04	Nervous/reproductive system effects	
Chlordane	0.002	Cancer risk	
Dalapon	0.2	Kidney/liver effects	
Di(2-ethylhexyl)adipate	0.4	Liver/reproductive effects	
1,2- Dibromochloropropane (DBCP)	0.0002	Cancer risk	
Di(2-ethylhexyl) Phthalate (DEHP)	0.006	Cancer risk	
Dinoseb	0.007	Thyroid/reproductive organ effects	
Diquat	0.02	Ocular/kidney/liver/gastrointestinal effects	
Endothall	0.1	Kidney/liver/gastrointestinal/reproductive effects	
Endrin	0.002	Kidney/liver/heart effects	
Epichlorohydrin	Treatment technology	Cancer risk	
Ethylene Dibromide (EDB)	0.00005	Cancer risk	
Gylphosate	0.7	Kidney/liver/reproductive effects	
Heptachlor	0.0004	Cancer risk	
Heptachlor Expoxide	0.0002	Cancer risk	
Hexachlorobenzene	0.001	Cancer risk	
Hexachlorocyclopentadiene (HEX)	0.05	Kidney/stomach effects	
Lindane	0.0002	Kidney/liver/nervous/immune/circulatory effects	
Methoxychlor	0.04	Kidney/liver/nervous/developmental effects	

Oxamyl (Vydate)	0.2	Kidney effects	
PAHs (Benzo(a)pyrene)	0.0002	Cancer risk	
PCBs (Polychlorinated Biphenyls)	0.0005	Cancer risk	
Pentachlorophenol	0.001	Cancer risk	
Picloram	0.5	Kidney/liver effects	
Simazine	0.004	Cancer risk	
Toxaphene	0.003	Cancer risk	
Volatile Organic Compounds			
1,1,1-Trichloroethane	0.2	Liver/nervous/circulatory system effects	For VOCs: Original monitoring required four quarterly samples during the first three years. Monitoring annually beginning in 1996, if no detects. Or, if all 21 VOCs were sampled after January 1988 and not detected, the state may authorize annual sampling in 1993. Monitor every three years after three years of no detects. Monitoring may be reduced based upon results of vulnerability assessment. USEPA issued guidelines for alternative monitoring (formerly Permanent Monitoring Relief) in August 1997. After completion of source water assessments in 2003, primary agencies will be allowed to develop alternative monitoring requirements.
1,1,2-Trichloroethane	0.005	Kidney/liver effects	
1,1-Dichloroethylene	0.007	Kidney/liver effects	
1,2,4-Trichlorobenzene	0.07	Internal organs/adrenal effects	
1,2-Dichloroethane	0.005	Cancer risk	
1,2-Dichloropropane	0.005	Cancer risk	
Benzene	0.005	Leukemia/cancer risk	
Carbon Tetrachloride	0.005	Cancer risk	
Chlorobenzene	0.1	Nervous system and liver effects	
Cis-1,2-Dichloroethylene	0.07	Liver/nervous/circulatory effects	
Dichloromethane	0.005	Cancer risk	
Ethylbenzene	0.7	Kidney/liver/nervous system effects/cancer risk	
Ortho-Dichlorobenzene	0.6	Kidney/liver/blood cell effects	
Para-Dichlorobenzene	0.075	Kidney/liver effects	
Styrene	0.1	Liver/nervous system effects	
Tetrachloroethylene (PCE)	0.005	Cancer risk	
Toluene	1	Kidney/liver/nervous/circulatory effects	
Trans-1,2-Dichloroethylene	0.1	Liver/nervous/circulatory effects	
Trichloroethylene (TCE)	0.005	Cancer risk	
Vinyl Chloride	0.002	Cancer risk	
Xylenes (total)	10	Kidney/liver/nervous system effects	

TABLE 1–7. MCL, Health Effects, and Monitoring Requirements for Microbiological Contaminants

Name of Contaminant	Maximum Contaminant Level (MCL)	Health Effects of Contaminant	Monitoring Requirements
Total Coliform rule (TCR) Total Coliforms Fecal Coliforms *E. coli*	Less than 40 samples/month; no more than 1 positive for total coliforms. If 40 samples or more/month, no more than 5% positive. MCLG = 0 for total coliform, fecal coliform and *E. coli.* Every sample containing total coliforms must be analyzed for fecal coliforms.	The presence of fecal coliforms or *E. coli* may indicate potential contamination that can cause diarrhea, cramps, nausea, headaches, or other symptoms.	For both surface waters and groundwaters, the total number and location of samples is based upon the population served and a system-specific sampling plan (see regulations).
Surface Water Treatment rule			
Turbidity	0.5 NTU in 95% of samples of filtered water, 5 NTU max	None. Interferes with disinfection.	Turbidity: Grab samples at least every four hours or continuous monitoring.
Giardia	Treatment technique (MCLG = 0)	Giardiasis.	
Enteric Viruses	Treatment technique (MCLG = 0)	Gastrointestinal and other viral infections.	Continuous chlorine residual required for systems $>3{,}300$.
Legionella	Treatment technique (MCLG = 0)	Legionnaires’ disease.	One to four grab samples per day are allowed for systems $<3{,}300$.
Heterotrophic Plate Count (HPC)	Treatment technique (MCLG = none)	Gastrointestinal Infections.	
Interim Enhanced Surface Water Treatment rule			Conventional and direct filtration systems must measure combined filter effluent turbidity at least every four hours and continuously monitor turbidity of each individual filter. PWSs that do not have four quarters acceptable TTHM and HAA5 data must conduct monitoring for four quarters (must be completed by February 2000). If annual average for TTHM > 0.064 mg/L or HAA5 > 0.048 mg/L and less than three years acceptable operational data exist, a PWS must monitor disinfectant residual, temperature, pH (chlorine only), and peak hourly flow on a daily basis for its Disinfection Profile (must begin by February 2000 and continue for one year).
Turbidity	0.3 NTU in 95% of samples of filtered water, 1 NTU max	None. Interferes with disinfection.	
Cryptosporidium	Treatment technique (MCLG = 0)	Cryptosporidiosis.	

Measurements of heterotrophic bacteria can be substituted for measurements of chlorine residual. A minimum of four-log removal/inactivation of virus and three-log removal/inactivation of *Giardia* is required.

Disinfectants and Disinfection By-products

Table 1–8 summarizes the requirements for MCLs, health effects, and monitoring requirements for disinfectants and disinfection by-products. For the first time, limits have been placed on the maximum residual disinfectant concentrations that can be present in the distribution system (MRDLs in Table 1–8). Chapter 3 provides further discussion of DBP formation. Total THMs include chloroform, bromoform, bromodichloromethane, and dibromochloromethane. HAA5 include chloroacetic acid, dichloracetic acid, trichloracetic acid, bromoacetic acid, and dibromacetic acid.

Special Requirements With the Stage 1 D/DBP Rule, a treatment technique is established for the removal of total organic carbon (TOC). By reducing the level of TOC, the formation of the disinfection by-products will also be reduced. The treatment technique is enhanced coagulation or softening and applies to all public water systems using conventional filtration regardless of size. The treatment technique can be met in one of two ways. The first way is meeting the removal percentage required (shown in Table 1–9) of the influent TOC based on the raw water TOC and alkalinity. If a system cannot meet the percentage removal requirements of the first step, then the other way is to apply to the primacy agency to determine the point of diminishing returns (PODR). PODR is the point on the TOC versus coagulant dose plot where the slope changes from greater to less than 0.3 mg/L of TOC removal for 10 mg/L coagulant dose. At this point of change, the alternative removal percentage for TOC can be set.

Exceptions There are no exceptions to the rule, but there are alternative compliance criteria. If the alternative compliance criteria are met, the system is deemed to have met the TOC removal requirements. These criteria are:

- The system’s source water TOC is less than 2.0 mg/L.
- The system’s treated water TOC is less than 2.0 mg/L.
- The system’s source water TOC is less than 4.0 mg/L, the source water alkalinity is greater than 60 mg/L (as $CaCO_3$), and the system’s DBP levels for TTHM and HAA5 are less than 40 μg/L and 30 μg/L, respectively.
- The system is using only chlorine as its disinfectant and the DBP levels for TTHM and HAA5 are less than 40 μg/L and 30 μg/L, respectively.
- The system’s source water specific ultraviolet absorbance (SUVA) prior to any treatment is less than 2.0 L/mg-m.
- The system’s treated SUVA is less than 2.0 L/mg-m.

Secondary Standards

Table 1–10 summarizes the current national secondary drinking water contaminant standards.

TABLE 1–8. MCL, Health Effects, and Monitoring Requirements for Disinfectants and Disinfection By-products

Name of Contaminant	Maximum Contaminant Level (MCL)— (mg/L unless noted)	Health Effects of Contaminant	Monitoring Requirements
Stage 1 D/DBPR			
Disinfectants			Chlorine/chloramines: Monitor at the same sample location as the TCR. Compliance based on running annual arithmetic average of monthly averages.
Chlorine	MRDL 4.0 (as Cl_2)	Hemolytic anemia in dialysis	Chlorine Dioxide and Chlorite: Daily sample at distribution system entry point.
Chloramines	MRDL 4.0 (as Cl_2)	Hemolytic anemia	Chlorine Dioxide and Chlorite: Daily sample at distribution system entry point.
Chlorine Dioxide	MRDL 0.8 (as ClO_2)	Cancer risk	
Disinfection By-products			
Total trihalomethanes (TTHMs)	0.080	Cancer risk	TTHM/HAA: 4 quarterly samples. Compliance based on running annual average.
Haloacetic Acids (HAA5)	0.060	Cancer risk/nervous system/liver effects	
Chlorite	1		
Bromate	0.010		Bromate: 1 sample per month (ozone systems only) and running annual average.
Total Organic Carbon (TOC)	Treatment Technique		TOC: Source and treated water TOC sampled once a month.

TABLE 1–9. Required Removal of TOC by Enhanced Coagulation and Softening

Source Water TOC (mg/L)	Source Water Alkalinity (mg/L as $CaCO_3$)		
	0–60	61–120	>120
>2.0–4.0	35%	25%	15%
4.0–8.0	45%	35%	25%
>8.0	50%	40%	30%

Note: Systems practicing enhanced (lime) softening must meet the TOC removal requirements of the last column on the right.

PENDING REGULATIONS

At the time of this writing, there are several new NPDWRs pending for arsenic, sulfates, radionuclides, groundwater, recycling of filter backwash water, Long-Term Enhanced Surface Water Treatment Rules, and Stage 2 of the Disinfectant/Disinfection By-Product (D/DBP) rule. The Long-Term Enhanced Surface Water Treatment Rule is expected to apply the turbidity requirements of the interim rule to systems serving less than 10,000 population. The Stage 2 D/DBP rule is expected to maintain the Stage 1 limits for THMs and HAA5s (0.08 mg/L and 0.06 mg/L, respectively), although the limits of each monitoring site will be regulated by a Locational Running Annual Average (LRAA). The Long-Term Enhanced Surface Water Treatment Rule is expected to require water systems with poor raw-water quality to supplement their conventional treatment with additional treatment from a “toolbox” of choices. Raw water containing significant levels of *Cryptosporidium* oocysts may need as much as 2½ logs of additional removal/inactivation credit and may dictate the use of ozone or membrane filtration. The pending regulations are summarized in Table 1–11. To keep up to date on these changing regulations, the reader may be included on a list to receive periodic updates by writing to: HDR Engineering, 2211 S. IH-35, Suite 300, Austin, TX 78741; or call (512) 912-5100.

FUTURE TRENDS

The current and pending rules discussed in this chapter are the base upon which future regulations will be built. The 1996 SDWA amendments require that EPA review and revise, as needed, each NPDWR at least every six years, with the first such deadline occurring in August 2002, and every six years thereafter. Currently, 92 contaminants are regulated, including:

- Turbidity
- 8 microbial indicators
- 4 radionuclides
- 19 inorganic contaminants
- 60 organic chemicals

In addition, there are secondary standards recommended for 15 contaminants and a candidate list of 50 chemicals and 10 microbial contaminants under consideration for

TABLE 1–10. National Secondary Drinking Water Regulations*

Contaminant	Effect(s)	SMCL (mg/L)
Aluminum	Colored water	0.05–0.2
Chloride	Salty taste	250
Color	Visible tint	15 color units
Copper	Metallic taste; blue-green stain	1.0
Corrosivity	Metallic taste; corrosion; fixture staining	Noncorrosive
Fluoride	Tooth discoloration	2
Foaming agents	Frothy; cloudy; bitter taste; odor	0.5
Iron	Rusty color; sediment; metallic taste; reddish or orange staining	0.3
Manganese	Black-to-brown color; black staining; bitter, metallic taste	0.05
Odor	‘‘Rotten egg,’’ musty, or chemical smell	3 TON (threshold odor number)
pH	Low pH; bitter metallic taste, corrosion; high pH; slippery feel, soda taste, deposits	6.5–8.5
Silver	Skin discoloration; graying of the white part of the eye	0.10
Sulfate	Salty taste	250
Total dissolved solids (TDS)	Hardness; deposits; colored water; staining; salty taste	500
Zinc	Metallic taste	5

*All NSDWRs established in 1979 except for aluminum—1991; fluoride—1986; silver—1991.

TABLE 1–11. Pending Regulations

Regulation	Statutory Deadlines Proposed Rule By: Final Rule By:	Summary
Radon	August 6, 1999 August 6, 2000*	The proposed regulation provides two options for the maximum level of radon that is allowable in community water supplies. The proposed MCL is 300 pCi/L, and the proposed alternative MCL is 4,000 pCi/L. The drinking water standard that would apply for a system depends on whether a state or CWS develops a multimedia mitigation (MMM) program. The proposed radon in drinking water rule applies to all CWSs that use groundwater or mixed ground and surface water.
Stage 2 Disinfectants/Disinfection By-products Rule	Final by May 2002	To be promulgated simultaneously with the LT2ESWTR, the Stage 2 D/DBPR is intended to address concerns about risk trade-offs between pathogens and DBPs. Under the framework approved in September 2000 by the Federal Advisory Committee Act (FACA) workgroup, Stage 2 MCLs for TTHMs and HAA5 will stay the same as Stage 1. A new Locational Running Annual Average (LRAA) established TTHM and HAA5 limits at all monitoring points. All systems must perform an Initial Distribution System Evaluation (IDSE) to target DBP hot spots.
Aldicarb		USEPA is reexamining risk assessment and occurrence data on aldicarb to determine if the administrative stay on the 1991 MCL of 7 μg/L should be lifted, revised, or otherwise acted upon.
Arsenic	January 1, 2000† January 1, 2001	The rule will revise the current maximum contaminant level (MCL) based on new data regarding the carcinogenic and noncarcinogenic health effects of arsenic. EPA has proposed lowering the current arsenic MCL of 50 μg/L down to 5 μg/L but has received public comments on MCL option of 3, 10, and 20 μg/L.
Groundwater Rule	Final by November, 2000‡	The rule will specify the appropriate use of disinfection while addressing other components of groundwater systems to ensure public health protection. The rule will stress a multibarrier approach, including best management practices and source water monitoring and contamination control. Potentially affected are all public systems using a groundwater source.
Long Term 1 Enhanced Surface Water Treatment (LT1ESWT/Filter Backwash [FB] Rule)	Final LT1ESWT by November 2000 Final FB Rule by August 2000§	This rule combines the LT1ESWT and FB rules to strengthen microbial controls for systems serving fewer than 10,000 people and prevent significant increase in microbial risk where small systems take steps to implement the Stage 1 Disinfectants and Disinfection By-products Rule. The LT1ESWTR will track the IESWTR for improved turbidity control, including individual filter monitoring and reporting. The rule will also address disinfection profiling and benchmarking. The FB rule will set a standard on recycling filter backwash within the treatment process of public water systems. The regulation will apply to all public water systems, regardless of size.

TABLE 1–11. (*Continued*)

	Statutory Deadlines	
Regulation	Proposed Rule By: Final Rule By:	Summary
Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)	Final by May, 2002	Developed simultaneously with the Stage 2 Disinfectants and Disinfection By-products Rule and will strengthen filtration provisions and microbial protection. Under the FACA framework approved in September 2000, all systems will be categorized into one of four "bin" classifications based on raw-water quality. Systems having poor raw water will be required to add additional log removal credits, which can be achieved via numerous nontreatment and treatment techniques.
Radionuclides (Radium, Uranium, Alpha, Beta, and Photon Emitters)	Final Rule for Radionuclides by November 2000‖ Final Rule for Uranium by 2000‖	Final Rule for Uranium by 2000. The current radionuclides regulation is being revised, and a standard will be set for uranium, using information from health effects studies, occurrence data, treatment technology, and analytical methods.
Sulfate	August 6, 2000‖ (Determination to regulate)	Sulfate is on the list of 83 contaminants required to be regulated under the 1986 SDWA amendments. USEPA is evaluating a sulfate study report and workshop summary, public comments, and other pertinent information to determine whether to regulate sulfate.
Methyl tertiary butyl ether (MTBE)	No schedule	Because of recent, widespread detection of MTBE in both ground and surface water supplies, EPA has added MTBE to its Contaminant Candidate List for possible future regulations. The gasoline additive allows gas to burn more efficiently. Refiners were required to add oxygenates such as MTBE to gasoline to comply with the Clean Air Act of 1990. By 1995, MTBE had become the second most widely produced organic compound in the U.S.[14] (See Chapter 3.)

Source: Reference 15.

[*] Radon proposal actually published November 2, 1999; (64 FR 59246). Final rule expected in early 2001.

†Arsenic proposal actually published June 22, 2000 (65 FR 38888); statutory deadline amended by Congress to June 22, 2001.

‡Groundwater proposal actually published May 10, 2000 (65 FR 30194); expect final rule in early 2001.

§LTIESWTR and FBR proposed published April 10, 2000 (65 FR 19046); expect final rules in early 2001.

‖EPA missed deadline.

future regulation (published in March 2, 1998, *Federal Register*). When regulating additional contaminants, EPA is to give priority to those contaminants that pose the greatest threat to public health and is to consider specific subgroups, such as infants, children, the elderly, pregnant women, individuals with a history of serious illness, or other subgroups that may be at greater-than-average risk due to exposure to contaminants in drinking water. Interim regulations can be promulgated without going through all of the steps normally required if there is an urgent threat to public health. As the analytical ability to detect small concentrations of added contaminants improves, water suppliers will be faced with an ever-increasing list of specific contaminant limits that must be monitored and met.

There will be a point at which regulations reach a practical limit on regulating the quality of water that can be produced by treatment and as measured at the exit to the treatment plant. As a result, regulatory emphasis may shift in the future to steps designed to:

- Provide greater protection of raw-water quality in surface water watersheds or groundwater wellheads. Most systems should already be implementing some type of watershed or wellhead protection program, but they can expect increased attention from customers as inventory data and protection boundaries become widely available. Increased attention may bring increased regulation.
- Limit water quality changes in the distribution system. Current results relative to coliform bacteria, lead, and copper, and disinfection by-products have established precedents for regulating water quality at the point of delivery to the customer, not just at the exit from the water plant. Future regulations may well add limitations on other constituents at the tap. For example, current limits on nitrates are at the treatment plant exit and do not take into account the potential increases in nitrite that may occur in the distribution system as the result of nitrification.
- Impose increasingly stringent requirements for operator certification. Current regulations mandate the establishment of a certified operator program for all federally regulated community and nontransient-noncommunity systems in the near future (current deadline: February 2001). Future regulations may establish more definitive regulation of operator qualification and certification.
- Require suppliers to demonstrate the adequacy of system capacity. States are now required to develop an implementation strategy for existing systems to acquire and maintain system capacity by October 2000. It is conceivable that this effort may be followed by added regulations designed to compensate for any perceived inadequacies of this initial effort.
- Establish requirements for the technical, managerial, and financial capabilities of the water supplier. In order to receive state revolving fund loans, a water supplier must already demonstrate that it has the technical, managerial, and financial ability to comply with SDWA requirements. Regulatory intervention in these areas may well increase in the future.

The prudent water supplier will adopt and regularly update a strategic plan for anticipating and dealing with a changing regulatory environment.

Additional Information

For an additional detailed information of the Legislative and Regulatory History of the Safe Drinking Water Act, please refer to the comprehensive discussion by Pontius, F. W., and Clark, S. W., Chapter 1, *Water Quality & Treatment*, 5th Ed., AWWA, 1999.

LIST OF ACRONYMS

1872
Yellowstone Act (YA)

1893
Interstate Quarantine Act (IQA)

1899
Rivers and Harbors Act (RHA)

1906
Antiquities Act (AA)

1908
National Bison Range Act (NBRA)

1910
Insecticide Act (IA)

1911
Weeks Act (Protects Watersheds) (WA)

1916
National Park Service Created (NPS)

1929
Migratory Bird Conservation Act (MBCA)

1934
Taylor Act—Grazing (TA)
Fish and Wildlife Coordination Act (FWCA)
Birds Protection Act (BPA)

1935
National Labor Relations Act (NLRA)
Wildlife Preservation Act (WPA)

1937
Federal Aid to Wildlife Restoration Act (FAWRA)

1940
American Eagle Protection Act (AEPA)

1947
Federal Insecticide, Fungicide and Rodenticide Act (FIFRA)

1954
Atomic Energy Act (AEA)

1956
Fish and Wildlife Act (FWA)

1957
Price-Anderson Act (PAA)

1958
Fish and Wildlife Coordination Act Amendments (FWCAA)

1960
Federal Hazardous Substances Act (FHSA)
National Forest Multiple Use Act (NFMUA)

1964
Wilderness Act (WLDA)

1965
Water Resources Planning Act (WRPA)
Anadromous Fish Conservation Act (AFCA)

1966
National Historic Prerservation Act (NHPA)

1967
Air Quality Act (AQA)
Freedom of Information Act (FOIA)

1968
Wild and Scenic Rivers Act (WSRA)
Estuarine Act (EA)
Radiation Control for Health and Safety Act (RCFHSA)

1969
Federal Coal Mine Health and Safety Act (FCMHSA)

1970
National Environmental Policy Act (NEPA)
Environmental Quality Improvement Act (EQIA)
Clean Air Act (CAA)
Creation of Environmental Protection Agency (EPA)
Environmental Education Act (EEA)
Occupational Safety and Health Act (OSHA)
Federal Aid to Wildlife Restoration Act Amendments (FAWRAA)

1972
Black Lung Benefits Act (BLBA)
Federal Water Pollution Control Act (FWPCA)
Marine Protection, Research and Sanctuaries Act (MPRSA)
Coastal Zone Management Act (CZMA)
Noise Control Act (NCA)
Federal Environmental Pesticide Control Act (FEPCA)
Ports and Waterways Safety Act (PWSA)
Marine Mammal Protection Act (MMPA)

1973
Endangered Species Act (ESA)
Trans-Alaska Pipeline Authorization Act (TAPA)

1974
Forest and Rangeland Renewable Resources Planning Act (FRRRPA)
Safe Drinking Water Act (SDWA)
Deepwater Port Act (DPA)

1975
Hazardous Materials Transportation Act (HMTA)

1976
Toxic Substances Control Act (TSCA)
Federal Land Policy and Management Act (FLPMA)
Resource Conservation Recovery Act (RCRA)

National Forest Management Act (NFMA)
Coastal Zone Management Act Amendments (CZMAA)

1977
Clean Air Act Amendments (CAAA)
Clean Water Act (CWA)
Surface Mining Control and Reclamation Act (SMCRA)
Soil and Water Resources Conservation Act (SDWAA)

1978
Black Lung Reform Act (BLRA)
Environmental Research, Development, Demonstration Authorization Act (ERDDAA)
Endangered American Wilderness Act (EAWA)
National Ocean Pollution Planning Act (NOPPA)
Port and Taker Safety Act (PTSA)
Uranium Mill Tailings Radiation Control Act (UMTRCA)
Endangered Species Act Amendments (ESAA)
Quiet Communities Act (QCA)
National Climate Program Act (NCPA)

1979
Archaeological Resources Protection Act (ARPA)

1980
Acid Precipitation Act (APA)
Solid Waste Disposal Act (SWDA)
Comprehensive Environmental Response, Compensation and Liability Act (CERCLA)
Coastal Zone Management Improvement Act (CZMIA)
Colorado Wilderness Act (COWLDA)
Fish and Wild Life Conservation Act (CWLCA)
Marine Protection Research and Sanctuaries Act Amendments (MPRSAA)

1982
Nuclear Waste Policy Act (NWPA)

1983
Marine Protection Research and Sanctuaries Act Amendments (MPRSAA)

1984
Resources Conservation and Recovery Act Amendments (RCRAA)
Wilderness Act Implementation in Various States (WLDI)

1986
Safe Drinking Water Act Amendments of 1986 (SDWAA-86)
Superfund Amendments and Reauthorization Act (SARA)

1987
Water Quality Act of 1987 (WQA)

1988
Alternative Motor Fuels Act of 1988 (AMFA)
Archaeological Resources Protection Act of 1979, Amendments (ARPAA)
Asbestos Information Act of 1988 (AIA)
Atlantic Striped Bass Conservation Act Authorization and Amendment (ASBCAAA)
Endangered Species Act Amendments of 1988—African Elephant Conservation Act (ESAA—AECA)

Federal Insecticide, Fungicide and Rodenticide Amendments of 1988 (FIFRAA)
Federal Ecosystems and Atmospheric Pollution Research Act of 1988 (FEAPRA)
Indoor Radon Abatement, Amendment (IRA)
Nuclear Waste Policy Act of 1982, Amendments (NWPAA)
Comprehensive Ocean Dumping Research Amendments and Authorization: National Marine Sanctuaries Program, Amendments and Authorization (CODRA/NMSPAA)
Federal Cave Resources Protection Act of 1988 (FCRPA)
Marine Mammals Protection Act Amendments of 1988 (MMPAA)

1989
North American Wetlands Conservation Act (NAWCA)

1990
Earth Day Proclamation (20th Anniversary) (EDP)
Oil Pollution Act of 1990 (OPA)
Radiation Exposure Compensation Act (RECA)
Clean Air Act Amendments (CAAA-90)
Global Change Research Act of 1990 (GCRA)
Great Lakes Fish and Wildlife Restoration Act of 1990 (GLFWRA)
Hazardous Materials Transportation and Uniform Safety Act of 1990 (HMTUSA)
National Environmental Education Act (NEEA)

GLOSSARY OF TERMS USED IN DRINKING WATER TREATMENT

Acute Health Effect An adverse health effect that occurs immediately (within a few hours up to a few days) as a result of consuming drinking water containing certain contaminants (e.g., pathogens).

Best Available Technology (BAT) The drinking water treatment technology deemed by the US EPA to be the most effective and/or appropriate technology to remove a specific contaminant from a raw-water supply.

Carcinogen A natural or manmade compound that may be found in water and is either known to cause or suspected to cause cancer in humans when either consumed and/or inhaled (e.g., chloroform is a consumed chemical, and radon gas is inhaled when released from drinking water).

Chronic Health Effect An adverse health effect that occurs after many years of exposure to a drinking water contaminant at levels above the maximum contaminant level.

Coliform Bacteria A family of many species of bacteria whose presence in drinking water may indicate contamination with sewerage and disease-causing microorganisms.

Community Water Systems (CWSs) Provide drinking water to the same people year-round. There are approximately 54,000 CWSs serving water to over 250 million people.

Cryptosporidiosis The gastrointestinal disease caused by the *Cryptosporidium* pathogen when ingested by humans. Symptoms of the illness include nausea, stomach cramps, diarrhea, and/or bloody diarrhea. There is no medicine to treat the illness, and healthy individuals will likely recover in 1 to 4 weeks. Persons with weakened or compromised immune systems (the very young, very old, cancer patients, AIDS patients, etc.) are likely to have more severe symptoms.

Cryptosporidium A protozoan organism (2–6 microns in size) found in most watersheds and surface waters where human or animal activity occurs. The organism is surrounded by an protective outer shell called an oocyst, can lie dormant in the environment for many years, and is highly resistant to disinfection.

Disinfection By-products (DBPs) The class of chemicals that are formed when disinfectants react with the organic compounds in water. Some of these compounds are carcinogens, and some are suspected of causing acute health effects.

Gastroenteritis A broad term used to describe illness associated with the stomach and intestines that may be caused by drinking water contaminated with pathogenic (disease-causing) microorganisms, such as viruses, bacteria, or protozoa. Symptoms may include nausea, vomiting, cramps, diarrhea, and fatigue.

Giardia lamblia A protozoan organism (5–12 microns in size), frequently found in watersheds and surface waters, that may cause gastroenteritis if consumed.

Inorganic Chemicals (IOCs) Broad category of naturally occurring mineral-based elements, such as metals, radionuclides, and nitrates that can be found in drinking water.

Maximum Contaminant Level (MCL) The highest level of a specific compound that is allowed in treated drinking water. MCLs are established by regulation and are set as close to the MCLG as possible using the best available technology (BAT) to remove the contaminant. MCLs are enforceable standards and, if violated by a public water supplier, may result in fines and or inprisonment.

Maximum Contaminant Level Goal (MCLG) The level of a specific compound in drinking water below which no adverse health effects are expect to occur. MCLGs are derived using either observed or predicted health endpoints and are nonenforceable public health goals.

Microbes (Microorganisms) Tiny living organisms that cannot be seen with the naked eye, and are capable of causing disease or illness.

National Primary Drinking Water Regulations (NPDWRs) Legally enforceable standards established by US EPA that apply to all public water systems. These standards protect the consumers of drinking water by limiting the levels of harmful contaminants allowed in the treated water.

National Secondary Drinking Water Regulations (NSDWRs) Non-enforceable federal guidelines regarding the aesthetic effects (i.e., non-health-based) of drinking water, such as taste, color, odor, etc.

Non-community Water Systems A public water system that serves customers on less than a year-round basis. There are two types of noncommunity systems, and they are generally very small systems (serving less than 500 people): Approximately 96,000 **transient systems** serve water to people on an infrequent, temporary basis, such as those at gas stations, restaurants, etc. Approximately 20,000 **nontransient systems** serve the same people for more than 6 months in a year, such as those at schools and factories.

Organic Chemicals Carbon-based, manmade chemicals, such as solvents and pesticides, which can get into a water supply through runoff from cropland or discharge from factories.

Pathogens Disease-causing organisms, such as some bacteria, viruses, and protozoa.

Public Water System A water supply and distribution system that serves at least 25 people or 15 service connections for at least 60 days per year. There are approximately 170,000 PWSs in the United States providing water to over 250 million people.

Treatment Technique A process required to be used in lieu of an MCL to reduce the level of a contaminant found in drinking water.

REFERENCES

1. U.S. EPA, *25 Years of the Safe Drinking Water Act: History and Trends,* 1999.
2. Baker, M. N., *The Quest for Pure Water,* American Water Works Association, New York, 1948.
3. Rosenau, M. J., *Preventative Medicine and Hygiene*, 5th ed., Appleton-Century, New York, 1935.
4. Klein, L., *Aspects of Water Pollution*. Academic Press, Inc., New York, 1957.
5. Dublin, L. L., et al., ‘‘Length of Life,’’ The Ronald Press, New York, 1949.
6. *Drinking Water and Health,* Safe Drinking Water Committee, National Academy of Sciences, Washington, DC, 1977.
7. MacKenzie, W. R., et al., ‘‘A Massive Outbreak in Milwaukee of *Cryptosporidium* Infection Transmitted Through Public Water Supply,’’ *N. Engl. J. Med.*, 331, 1994:161–167.
8. Morris, R. D., et al., ‘‘Temporal Variation in Drinking Water Turbidity and Diagnosed Gastroenteritis in Milwaukee.’’ *Am. J. Public Health,* 86(2), February 1996:237–239.
9. Oleckno, W. A., ‘‘The National Interim Primary Drinking Water Regulations. Part I—Historical Development.’’ *Jour. Envir. Health*, 44:5, May 1982.
10. Pontius, F. W., and Clark, S. W., Chapter 1, *Water Quality & Treatment,* 5th ed., AWWA, 1999.
11. *Water Quality and Treatment: A Handbook of Community Water Supplies,* 5th ed., American Water Works Association, 1999.
12. *Drinking Water and Health*, Vols. 1–5, Safe Drinking Water Committee, National Academy Press, Washington, DC, 1980, 1982, 1983.
13. Bellar, T. A., Lichtenberg, J. J., and Kroner, R. C., ‘‘The Occurrence of Organohalides in Chlorinated Drinking Water,’’ *JAWWA*, 66(12), December 1974:703.
14. Liang, S., et al., ‘‘Oxidation of MTBE,’’ *JAWWA* 91(6), June 1999.
15. *Opflow.* American Water Works Association, January 2000.

CHAPTER 2

Inorganic and Radionuclide Contaminants

This chapter presents information on four classes of contaminants:

- Inorganic constituents of health concern
- Inorganic contaminants causing aesthetic problems
- Residual disinfectants and inorganic disinfection by-products
- Radionuclides

Information on current U.S. water quality standards for these contaminants is found in Chapter 1.

INORGANIC CONTAMINANTS OF HEALTH CONCERN

Drinking water supplies are susceptible to contamination by a wide range of inorganic constituents that are known or suspected to cause adverse health effects. These contaminants enter water supplies from natural sources or from pollutant sources. Also, constituents such as lead and copper may be contributed from corrosion of piping and plumbing materials in the distribution system. The following sections present information on the source, occurrence, and health effects of 23 inorganic contaminants of concern.

Aluminum

Sources Aluminum is the most abundant metal found on the earth's crust—8.1 percent—and the third most abundant element. It is found in nature as alumino silicates, such as clay, kaolin, mica, and feldspar. Bauxite, an impure hydrated oxide ore, is a major source of industrial production.

Occurrence Aluminum occurs naturally in nearly all foods. Dietary intake varies from 1.53 to 100 mg/day, with an average of 20 mg/day. Aluminum is common in treated drinking water, especially water treated with alum; however, drinking provides a relatively minor proportion of dietary intake compared with other sources[1] (see Table 2–1).

TABLE 2–1. Aluminum Levels in Finished Drinking Water

	Levels (mg/L)	Median (mg/L)
Groundwater—no coagulant	0.014–0.290	0.031
Surface water—no coagulant	0.016–1.167	0.043
Surface water—alum coagulant	0.014–2.670	0.112
Surface water—iron coagulant	0.015–0.081	0.038

Source: Reference 2.

Health Effects Aluminum has low acute toxicity. At lower doses, aluminum administered to laboratory animals is a neurotoxin. Chronic exposure data are limited, but indicate that aluminum likely affects phosphorus absorption in a way that can create weakness, bone pain, and anorexia. Carcinogenicity, mutagenicity, and teratogenicity tests have all been negative.[3]

After considerable controversy and extensive research, it appears that aluminum in drinking water does not significantly increase the incidence of Alzheimer's disease. A recent study demonstrated that drinking-water consumption can be expected to contribute only 0.4 to 1.1 percent of the lifetime body burden of aluminum.[4]

In chronic dialysis patients, encephalopathic syndrome was associated with aluminum as an etiologic agent. Pending further study, the dialysis fluid should be used with softened water treated by reverse osmosis and deionization, in that order.

EPA included aluminum as one of the 83 contaminants to be regulated as part of the 1986 SDWA, but later removed aluminum from the list due to a lack of demonstrated health risk. A secondary maximum contaminant level of 50 to 200 μg/L was established to ensure removal of coagulated material from the finished water.

Antimony

Sources Antimony is a metallic chemical element; it is brittle, bluish white with metallic luster, and is not abundant in nature. The most common antimony ores are the sulfide stibnite and the trioxide valentinite. In industry, the most widely used antimony compound is antimony trioxide, which is used as a flame retardant and in the production of batteries, paint, and glass.

Occurrence Industrial dust and exhaust gases of cars and fuels are the main sources of antimony in urban air. According to the Toxics Release Inventory, over 12 million pounds of antimony and antimony compound were released to land and water between 1987 and 1993, with land release being the primary path. These releases were predominantly from copper and other nonferrous smelting and refining industries. The largest releases occurred in Arizona and Montana. The greatest releases to water occurred in Washington and Louisiana.[5]

Few data are available on the occurrence of antimony in drinking waters because of past difficulties in measuring its presence in low concentrations. Antimony is likely to be present in only very low concentrations.

Health Effects Required antimony in human nutrition has not been determined, but its toxicity is known. With similarity in symptoms to arsenic poisoning, doses of

100 mg have been demonstrated to be lethal. Acute exposures at elevated concentrations may cause nausea, vomiting, and diarrhea, whereas long-term exposure may cause decreased longevity, decreased growth, or altered blood levels of glucose and chlorestorol. There is inadequate evidence to define whether antimony has the potential to cause cancer from lifetime exposures in drinking water.

Arsenic

Sources Natural sources, such as the erosion of rocks, are thought to account for most of the arsenic in surface and ground waters. Arsenic is present in the earth's crust in concentrations averaging 2 mg/L. Erosion of arsenic-containing surface rocks probably accounts for a significant amount of the arsenic in water supplies. It is concentrated in shales, clays, phosphorites, coals, sedimentary iron ore, and manganese ores.[6] Industrial uses of arsenic include semiconductor manufacturing, petroleum refining, wood preservatives, animal feed additives, and herbicides.

Occurrence Arsenic is widely distributed across the United States in low concentrations in surface waters, as shown in Table 2–2. Except for local exceptions where arsenic concentrations could be traced to specific causes, there are only minor regional differences in the average values in surface waters or in the percentage of contaminated samples.

Arsenic occurrence is highest in groundwater systems in the western United States In 1985, EPA conducted a survey collecting 982 samples of groundwater. With a detection level of 5 μg/L, the survey found detectable concentration of arsenic in 7 percent of 982 samples. Data collected from the 64 samples ranged from 5 to 48 μg/L, with a mean of 13 μg/L and a median of 9 μg/L.[2]

Health Effects Arsenic can combine with other elements to form water-soluble arsenicals that are readily absorbed through the gastrointestinal tract, lungs, and skin. Pentavalent arsenic, both organic and inorganic, is absorbed more readily than the trivalent form. Arsenic primarily enters the liver, kidneys, intestinal wall, spleen, and lungs.

Human exposure to arsenic of a sufficient concentration to cause poisoning is usually a result of ingestion of contaminated food or drink. Inorganic arsenite produces rapid collapse, shock, and death. The major characteristics of acute poisoning are

TABLE 2–2. Regional Summary of Arsenic in U.S. Surface Waters

Region	Maximum μg/L	Minimum μg/L	Median μg/L	Proportion, <10 μg/L %	Proportion >10 μg/L %
New England and Northeast	60	<10	<10	80	20
Southeast	1,110	<10	<10	70	30
Central	140	<10	<10	75	25
Southwest	10	<10	<10	87	13
Northwest	30	<10	<10	86	14

Source: Durum, W. H., Hem, J. D., and Heidel, S. G. "Reconnaissance of Selected Minor Elements in Surface Waters of the United States," October 1970, U.S. Geological Survey Circular 643, Washington, DC, 1971.

substantial gastrointestinal damage and cardiac abnormalities. Symptoms may be evident within 8 minutes if arsenic is ingested in a drink, but at up to 10 hours if it is solid and taken with food. The acute fatal dose of arsenic trioxide for a human is in the range of 70 to 180 mg, or 0.76 to 1.95 mg/kg of body weight for a 70 kg person.

There is a wide variation in toxicity of various arsenical formulations. The arsenic compounds are listed below in descending order of toxicity:[7]

- Arsines (trivalent, inorganic, or organic)
- Arsenite (inorganic)
- Arsenoxides (trivalent with two bonds to oxygen)
- Arsenate (inorganic)
- Pentavalent arsenicals, such as arsenic acids
- Arsenium compounds (four organic groups with a positive charge on arsenic)
- Metallic arsenic

Epidemiologic investigations have reported an association between arsenic exposure in drinking water and cancer. Studies in laboratory animals have not demonstrated this effect following ingestion exposures. Consequently, there is no comparable model system for studying arsenic-induced carcinogenicity. Arsenic has been shown to be mutagenic in several bacterial test systems, and sodium arsenate and arsenite have shown teratogenic potential in several mammalian species. The EPA has classified arsenic as a human carcinogen (Group A).

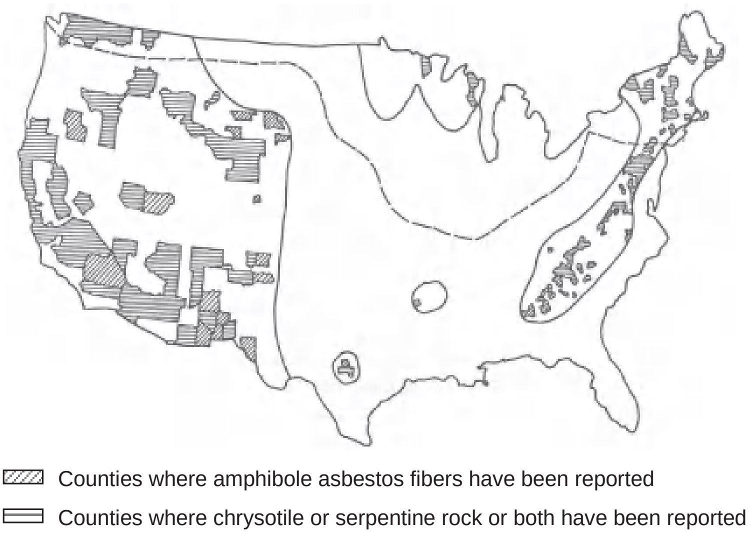
In addition to cancer, arsenic exposures have been reported to result in other adverse health effects. These include thickening of the skin; effects on the nervous system, such as tingling and loss of feeling in limbs; hearing impairment; effects on the heart and circulatory system, diabetes; developmental effects; and effects on the gastrointestinal system and liver. Many of these effects are observed at concentrations where cancer effects were observed in the epidemiology studies.[7]

Asbestos

Sources Asbestos fibers enter surface waters from both natural and anthropogenic sources. Large deposits of chrysotile and emphibole are found throughout North America (Fig. 2–1), and leaching from asbestos mineral deposits in contact with surface water constitutes the major natural source. Asbestos also occurs in water exposed to asbestos mining discharges, and in asbestos-cement pipe. Studies have also found that precipitation may result in surface water contamination through scavenging of airborne asbestos.[8]

Anthropogenic sources of asbestos fibers in water include mining operations and use of products containing asbestos. Chrysotile and amphibole are commercially mined to make over 3,000 products, including filters, auto brake and clutch linings, floor covering, paper products, textiles, gasket materials, and asbestos cement pipe.

Occurrence Asbestos levels are based on fiber counts rather than concentration by weight. WHO reports that expected values of asbestos in raw water will vary from less than 1 to as much as 10 million fibers per liter (MFL), with an average value of 1 MFL. EPA sampling of water distribution systems in 406 cities in 47 states reported the following asbestos levels: 29 percent were below detection limits, 53 percent were



▨ Counties where amphibole asbestos fibers have been reported

▤ Counties where chrysotile or serpentine rock or both have been reported

Solid lines surround those areas where fiber-bearing rocks might exist. The dashed line shows lowest limit of glacial activity. Rocks not native to the area can be found north of this line.

Fig. 2–1. Locations of asbestos sources in the United States (Reprinted from *Journal AWWA*, Vol. 66, No. 9 [September 1974], by permission. Copyright © 1974, American Water Works Association.)

less than 1 MFL, 8 percent were between 1 and 10 MFL, and 10 percent had over 10 MFL. In 1981, EPA sampled 100 systems and found 0.08 MFL in 12 systems, with values ranging from 0.385 to 1.071 MFL.[2]

Health Effects Inhaled asbestos is clearly carcinogenic. Human occupational and laboratory animal inhalation exposures are associated with lung cancer, pleural and peritoneal mesothelioma, and gastrointestinal tract cancers.[3] The physical dimensions of asbestos fibers rather than the type are more important in health effects, with the longer, thinner fibers more highly associated with cancers by inhalation. In drinking water, some inhalation may take place through showers and humidifiers.

With respect to ingested asbestos, health effect findings are inconsistent, with some studies suggesting elevated risk for gastric, kidney, and pancreatic cancers.[9] The EPA has classified ingested asbestos as a possible human carcinogen (Group C).

The American Water Works Association Research Foundation has stated[2]:

> No firm evidence shows that the proper use of asbestos-cement pipe poses a hazard to health by reason of ingestion of asbestos fibers. Calculations comparing the probably ingestion exposure in occupational groups to that likely to occur as a result of ingestion of potable water from asbestos-cement pipe systems suggest that the probability of risk to health from the use of such systems is small, approaching zero.

Barium

Sources In surface and ground waters, naturally occurring barium may be present in trace amounts through contact with barium-containing rocks. The most common ores are the sulfate (barite) and the carbonate (witherite). Barium is found in limestones, sandstones, and occasionally in soils. Barium is stable in dry air, but readily oxidized by humid air or water. The normally low solubility of barium sulfate increases in the presence of chloride and other anions.

Industrial release of barium occurs from oil and gas drilling muds, smelting of copper, coal-fired power plants, jet fuels, auto paints, and manufacture of vehicle parts.

Occurrence By its chemical nature, barium is unlikely to be found in raw water as a barium ion. In seawater, it is recorded at 6.2 μg/L. Barium is usually found in higher concentrations in groundwater than in surface supplies. Maximum expected level to be found in community water supplies is 1 mg/L, although the drinking water of many communities in Illinois, Kentucky, Pennsylvania, and New Mexico can contain barium at concentrations of 7 to 10 mg/L.

There are limited survey data on the occurrence of barium in drinking water. Most supplies contain less than 200 μg/L of barium. The average concentration of barium in U.S. drinking water is 28.6 μg/L (1977 data). The drinking water of many communities in Illinois, Kentucky, Pennsylvania, and New Mexico contains concentrations of barium that may be 10 times higher than the drinking water standard. The source of these supplies is usually well water. Currently, 60 ground water supplies and 1 surface water supply exceeds 1000 μg/L.[5]

Health Effects Barium is very toxic when its soluble salts are ingested. The human fatal dose of barium chloride is about 0.8 to 0.9 g, or about 550 to 600 mg of barium. The human digestive system is permeable to barium, which is transferred to the blood plasma. Acute barium poisoning exerts a strong, prolonged stimulant action on all muscles, including cardiac and smooth muscle of the gastrointestinal tract and bladder. In small chronic doses, barium may cause an increase in blood pressure and hypertension. Barium does not accumulate in bones, muscles, kidneys, or other tissues. EPA has not classified the carcinogenicity of barium (Group D) because of inadequate evidence.

Beryllium

Sources Beryllium is a member of the alkaline earth metals. It is found in many minerals, and is mostly produced by reducing beryllium fluoride with magnesium metal. Certain fossil fuels contain beryllium compounds, perhaps accounting for its presence in some community air samples. It is primarily used as a metal and alloy in nuclear reactors and aerospace applications, and as an alloy and oxide in electrical equipment.

Occurrence Beryllium is not common in drinking water due to the insolubility of its oxides and hydroxides at the normal pH range. It has been reported to occur in U.S. drinking water at 0.01 to 0.7 mg/L.[5] Groundwater surveyed between 1962 and 1967 had a 5.4 percent detection frequency, with a maximum concentration of

1.22 μg/L. Surface water in the same surveys had a 1.1 percent detection frequency, with a maximum concentration of 0.17 μg/L.[3]

Health Effects Inhalation of beryllium causes pulmonary ailments but has not been shown to cause lung cancers in humans. It can, however, cause osteosarcomas in laboratory animals irrespective of the mode of administration.[3a] Although beryllium has caused cancer in animals, it has so far not proved to be carcinogenic to humans.

Cadmium

Sources Cadmium is a silvery-white, soft metal that is in the same periodic group as zinc and mercury. Cadmium is found wherever zinc is located in nature, and is widely distributed in the earth's crust, although not in large quantities. Cadmium in the environment results principally from industrial sources, such as electroplating facilities, textile manufacturing, and chemical industries. Cadmium and zinc are also found in the soils and tailings around mines and smelters, which may result in localized, high concentrations in adjacent waters. Cadmium occurs as an impurity in zinc and may enter consumers' tap water as a result of galvanized pipe corrosion. Cadmium is used as a yellow pigment and in ceramics, photography, and insecticides, and as an alloy with copper, lead, silver, aluminum, and nickel.

Occurrence NIPDWR compliance monitoring as of November 1985 showed 25 public water supplies with levels of cadmium greater than 0.010 mg/L. Federal surveys conducted between 1969 and 1980 showed a mean concentration of 3 μg/L in 707 groundwater supplies, and 3.2 μg/L in 117 surface water supplies.[3]

Soft water of low pH may register higher values when plumbing systems contain cadmium.

Health Effects Cadmium acts as an emetic at ingested doses of 3 to 90 mg, becomes toxic at 10 to 326 mg, and is fatal at 300 to 3,500 mg. Chronic exposure results in renal dysfunction. Cadmium has been shown to induce sarcomas at injection sites in laboratory animals and to induce lung tumors in rats exposed to cadmium chloride via aerosol for carcinogenic effects.[3]

EPA has classified cadmium as a probable human carcinogen (Group B1), based on positive carcinogenicity testing; however, cadmium is being regulated based on its renal toxic effects because the carcinogenicity occurs via inhalation.

Chromium

Sources Chromium is found in the earth's crust from 10 to 200 ppm, occurring mostly as chrome iron ore (chromite). Though widely distributed in soils and plants, it is rare in natural waters.[5] Chrome plating along with chrome metallurgical and chemical operations may contaminate the atmosphere with chromium. Also, fossil fuel combustion, solid waste and sewage sludge incineration, and cement plant emission are potential sources. Other chromium salt usage is found in the leather industry; in the manufacture of paints, dyes, explosives, ceramics, and papers; and through use as a corrosion inhibitor in the textile, glass, and photographic industries.[2]

Chrome may find its way to water supplies through road dust carrying chrome-containing brake-lining material.

Occurrence Chromium can exist as Cr(II) through Cr(VI) in the environment, but Cr(III), the trivalent form, predominates in natural waters. Background levels in water average 1 μg/L, whereas municipal drinking water contains 0.1–35 μg/L. The higher values of chromium can be related to sources of anthropogenic pollution.

Between 1965 and 1980, EPA sampled 795 groundwater supplies and 142 surface supplies. They found that 10 percent of the groundwater had chromium concentrations above 5 μg/L (in positive samples, the average was 16 μg/L and the maximum 49 μg/L) and 17 percent of the surface waters had levels above 5 μg/L (in positive samples, the average was 10 μg/L and the maximum 25 μg/L).

A survey of 3,834 tap waters reported the concentrations of chromium to range from 0.4 to 8.0 μg/L. The reported chromium concentrations in this study may be a little higher than the actual values due to inadequate flushing of tap water before collection of samples. This indicates that the concentration of chromium in household tap water may increase due to plumbing materials.[5]

Health Effects The toxicity of chromium depends on its valence. Trivalent chromium is low in toxicity, and is the nutritional form of chromium. Cr(III) is so low in toxicity that a wide margin of safety exists between the amount normally ingested and the amount needed to induce undesirable effects. The National Academy of Sciences Committee on Chromium stated: ‘‘Compounds of chromium in the trivalent state have no established toxicity. When taken by mouth, they do not give rise to local or systematic effects and are poorly absorbed. No specific effects are known to result from inhalation. In contact with the skin, they combined with proteins in the superficial layers, but do not cause ulceration.’’[6]

Hexavalent chromium, Cr(VI), is significantly more toxic than Cr(III) and does not have any nutritional value. Normally, Cr(VI) compounds cause irritation and corrosion. Cr(VI) may be absorbed through ingestion, contact with the skin, and inhalation. Cr(VI) causes hemorrhage of the gastrointestinal tract, ulceration of the nasal septum, and cancer of the respiratory tract from inhalation. Chromium has not been shown to be carcinogenic through ingestion exposure. EPA classifies chromium as a human carcinogen (Group A), although standards are based upon noncancer toxic effects.

Copper

Sources Copper is commonly found in the earth’s crust as sulfides, oxides (cuprite, malachite, axurite, chalcopyrite, bornite), and rarely as metal. Low concentrations in water (generally below 0.20 mg/L) can result from rock weathering. Some industrial contamination also occurs, but the principal sources in water supplies are corrosion of brass and copper pipe, and the addition of copper salts during water treatment for algal control. Industrial sources include smelting and refining, copper wire mills, coal burning industries, electroplating, tanning, engraving, photography, insecticides, fungicides, and iron- and steel-producing industries.

Occurrence A 1967 survey of 380 drinking water supplies found copper concentrations ranging from 0.001 to 1.06 mg/L with a mean of 0.043 mg/L. Other surveys in the United States recorded a mean value of 0.134 mg/L with a maximum of 8 mg/L.[2]

A 1981 survey conducted by EPA measured copper from the consumer’s tap (30-second flushed, random daytime grab sample). Copper concentrations exceeded

1 mg/L in 3 percent of the 772 samples and 0.2 mg/L in 19 percent of samples. The national average was 0.221 mg/L, with a median of 0.04 mg/L.

Groundwater supplies have been tested at random. Copper levels were below 0.06 mg/L in 85 percent of 983 tap samples and below 0.46 mg/L in 98 percent of the samples. Values above 1 mg/L were found in 1 percent of the samples, with a maximum value of 2.37 mg/L.[2]

Health Effects Copper is a nutritional requirement, with insufficient copper leading to iron deficiency and reproductive abnormalities. The NAS lists a safe and adequate copper intake of 2 to 3 mg/day.[3]

Copper doses in excess of nutritional requirements are excreted; however, at high doses, copper can cause acute effects, such as gastrointestinal disturbances, damage to the liver and renal systems, and anemia. A dose of 5.3 mg/day was the lowest at which gastrointestinal tract irritation was seen.[3]

Poisoning from copper in water is normally avoidable because the taste threshold concentration of copper is at 1.0 to 2.0 mg/L, with levels of 5 to 8 mg/L making the water undrinkable. Poisoning occurs at higher concentrations. Individuals with Wilson's disease (disorder of copper metabolism) are at additional risk from the toxic effects of copper.

Exposure of mice via subcutaneous injection yielded tumors; however, oral exposure in several studies did not. Mutagenicity tests have been negative. EPA has not classified the carcinogenicity of copper (Group D) because of inadequate evidence.

Cyanide

Sources Cyanide occurs as an industrial pollutant and is not commonly found in drinking water at significant levels. Cyanides are used in plastics, steel, electroplating, and metallurgic industry, as well as in synthetic fibers and chemicals.

Occurrence The 1970 EPA Community Water Supply Survey of 969 systems found an average cyanide concentration of 0.09 μg/L and a maximum of 8 μg/L.

Health Effects Cyanide is readily absorbed from the lungs, gastrointestinal tract, and skin. It combines with cell cytochrome and prevents oxygen transport. With chronic exposure, cyanide can be detoxified in the liver; cyanide is converted to thiocyanate. The carcinogenicity of cyanide has not been evaluated. Potassium cyanide was negative when tested for mutagenicity in bacterial systems.[3] EPA has not classified the carcinogenicity of cyanide (Group D) because of inadequate carcinogenicity data in animals and humans.

Fluoride

Sources The most electronegative element is fluoride, which exists naturally in the fluoride form. It occurs principally in the earth's crust in the form of fluorite (CaF_2) and fluorapatite ($Ca_{10}(PO_4)_6F_2$), and is the 17th most abundant element.[6] Small amounts of fluoride are present in most soils, except those that have been leached. Fluorine and its compounds are used to produce uranium and over 100 commercial fluorochemicals, including high-temperature plastics.

Occurrence One study on natural water sources found the following fluoride concentrations:[6]

• Rivers	0.0 to 6.5 mg/L
• Lakes	up to 1,625 mg/L
• Groundwaters	0.0 to 35.1 mg/L
• Seawater	average value of 1.2 mg/L

EPA has reported estimated national occurrence of fluoride in public water supply systems,[2] as shown in Table 2–3.

Fluoride has been added to drinking water for more than 30 years to reduce the potential for dental caries. The EPA has endorsed controlled additions of fluoride to domestic waters, because small amounts (about 1 mg/L) have a beneficial effect in preventing tooth decay, particularly among children.

Health Effects Research on epidemiological studies of high concentration of fluoride in natural water can be summarized as follows:[2]

- Dental fluorosis (mottling of teeth enamel) appears in a very small percentage when fluoride in drinking water is in the range of 1 to 2 mg/L.
- Long-term intake of fluoride in concentrations higher than 4 mg/L may cause asymptomatic osteosclerosis in a small percentage of persons.
- Crippling fluorosis has been detected in individuals exposed to fluoride levels from 10 to 40 mg/L.
- Sharply reduced dental caries formation has been determined when the fluoride level is at least 0.8 mg/L.
- No carcinogenicity or other adverse effects have been detected.

Fluoride has not been shown, unequivocally, to be an essential element for human nutrition, except to the extent that it reduces the incidence of dental caries. Adequate and safe intakes of fluoride have been estimated as follows:

• Infants less than 6 months:	0.1 to 0.5 mg/d
• Infants from 6 to 12 months:	0.2 to 1.0 mg/d
• Children from 1 to 3 years:	0.1 to 1.0 mg/d
• Children from 4 to 6 years:	1.0 to 2.5 mg/d
• Children from 7 years to adulthood:	1.5 to 2.5 mg/d
• Adults:	1.5 to 4.0 mg/d

These levels are thought to be sufficient to protect against dental caries and osteoporosis.

Acute toxicity due to fluoride rarely occurs in humans. An acute dose acts swiftly on the gastrointestinal mucosa causing vomiting, abdominal pain, convulsions, and other effects. A lethal dose is unlikely to occur.

TABLE 2–3. National Occurrence of Fluoride in Public Water Supply

	Surface Water				Groundwater			
	Systems with Fluoride (mg/L)				Systems with Fluoride (mg/L)			
Population Served	<1.0	1.0–2.0	2.0–4.0	>4.0	<1.0	1.0–2.0	2.0–4.0	>4.0
Less than 500	3,670	117	5	3	31,931	2,281	833	220
500 to 2,500	2,980	265	6	1	8,964	341	165	40
2,500 to 10,000	1,967	174	3	2	2,828	219	44	14
More than 10,000	1,615	148	2	0	1,187	48	6	2

Hardness

Sources Hardness is defined as the total quantity of polyvalent cations present in water, expressed as an equivalent quantity of $CaCO_3$. The principal cations that cause hardness are calcium and magnesium, although iron, manganese, and a few others also constitute hardness. Generally accepted classifications of water hardness are as shown in Table 2–4.

Occurrence Figure 2–2 summarizes the regional occurrence of hardness.

Health Effects Some research has indicated that there may be some health benefits associated with the consumption of hard water. Many studies done since the 1960s have demonstrated a fairly consistent relationship between soft water and cardiovascular disease. These studies show that people living in soft water areas have somewhat higher cardiovascular disease rates than those living in hard water areas. Although several theories to explain this phenomenon have been advanced, no causal relationships have been established. Other studies have indicated that long-term consumption of demineralized water may result in the lowering of the bone calcium saturation level.[10,11] Again, the causative factors have not been clearly established. These studies suggest that hard water may be healthier than very soft water.

In waters of very high noncarbonate hardness, the softening process itself may increase the sodium content of the water enough to have significance for people with high blood pressure.

Lead

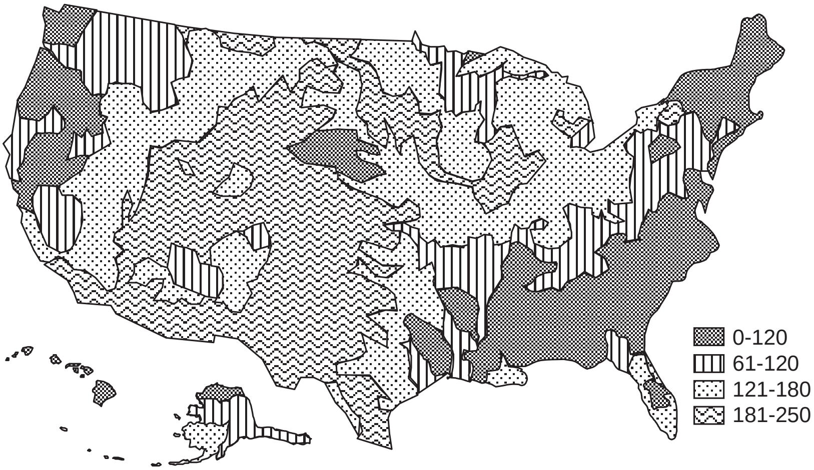
Sources In the environment, lead primarily occurs as sulfide (galena) in rocks, as well as in the forms of oxides and potassium feldspar. Lead carbonate is commonly seen in the oxidized zone of lead ores. The aqueous solubility of lead ranges from 0.5 mg/L in soft water to 0.003 mg/L in hard water.[12] Lead contributions to source waters may result from mining and processing activities, and, to a lesser extent, the use of leaded paints.

In drinking water, lead contamination results primarily from the corrosion of piping, fittings, and solder. Lead is more likely to be dissolved from distribution piping and plumbing by water that is low in hardness, bicarbonate, and pH, and high in dissolved oxygen and nitrate.

Occurrence Lead occurs in drinking water primarily from corrosion of lead pipes and solders, especially in areas of soft water. Federal surveys have found lead in 539 of 706 supplies using groundwater, with a mean concentration of 13 μg/L and a range

TABLE 2–4. Classification of Hardness in Water

Hardness Classification	Hardness, mg/L as $CaCO_3$
Soft	Less than 50
Moderately hard	50 to 150
Hard	150 to 300
Very hard	More than 300



Mean hardness as calcium carbonate at NASQAN stations during 1975 water year. Source: USGA Map.

Fig. 2–2. Regional occurrence of hardness

from 5 to 182 μg/L. The same surveys found lead in 100 of 119 supplies using surface water, with a mean concentration of 14 μg/L and a range of 5 to 32.5 μg/L.[3]

In other surveys, EPA found lead at levels above 5 μg/L in 75.5 percent of 1,200 groundwater supplies sampled, with a mean of 26 μg/L in ''positive'' supplies, with a range of 5 to 380 μg/L. In 273 surface water supplies tested, a similar percentage (76.2 percent) tested positive (above 5 μg/L), with a mean of 24 μg/L and a range of 5 to 164 μg/L.[2]

Health Effects Lead is a cumulative poison to humans. Acute lead poisoning is extremely rare. Health effects of lead are generally correlated with blood test levels. Infants and young children absorb ingested lead more readily than do older children and young adults. Lead exposure across a broad range of blood levels is associated with a continuum of pathophysiological effects, including interference with heme synthesis necessary for formation of red blood cells, anemia, kidney damage, impaired reproductive function, interference with vitamin D metabolism, impaired cognitive performance, delayed neurological and physical development, and elevations in blood pressure. EPA has classified lead as a probable human carcinogen (Group B2) because some lead compounds cause renal tumors in rats.[3]

Typical symptoms of advanced lead poisoning are: constipation, anemia, gastrointestinal disturbance, tenderness, and gradual paralysis in the muscles, specifically arms, with possible cases of lethargy and moroseness.

The subpopulation of special concern is children, where encephalopathy and death are registered at a starting level of 80 to 100 μg/dL (blood). In nonfatal cases, permanent, severe mental retardation with other neurologic symptoms is observed at levels as low at 40 to 60 μg/dL. Adverse health effects are noted in children with blood lead levels of 40 μg/dL or higher, with possible risk at levels as low at 15 to 30 μg/dL.[2]

Mercury

Sources Mercury, one of the least abundant elements in the earth's crust, has three oxidation states:[13]

- Elemental mercury
- Mercurous compounds
- Mercuric compounds

Mercury may be detected in at least 30 ores, and occurs in water primarily as an inorganic salt, and as organic mercury in sediments. A major natural source of mercury is the degassing of the earth's crust, which releases an estimated 25,000 to 150,000 tons of mercury per year.[5] Anthropogenic sources include burning of fossil fuels, metal refining operations, chloralkali plants, sewage discharges, and past uses of mercury-containing pesticides.

Occurrence Mercury detected in potable water is predominantly in the form of inorganic mercury. Maximum concentrations reported by EPA are:[2]

Groundwater = 30 percent above 0.5 μg/L from 106 supplies sampled

Surface water = 14 percent above 2.0 μg/L

Surface water = 32 percent surface water with 0.5 μg/L from 31 supplies sampled and 16 percent above 2 μg/L

Health Effects The comparative toxicity of different forms of mercury is related to their ability to be absorbed by humans. Inorganic mercury is poorly absorbed into the adult GI tract, does not readily penetrate cells, and is not as toxic as methyl mercury. However, the absorption of inorganic mercury can be much higher in infants and young children.[3] The principal target organ of inorganic mercury is the kidneys, with neurologic and renal disturbances.

Organic forms, such as methyl mercury, are readily absorbed and easily enter the central nervous system, causing death or mental and motor dysfunctions. The fact that certain microorganisms can convert inorganic and organic forms of mercury to the very toxic methyl or dimethyl form makes any form of mercury potentially dangerous in the environment.

Mercury intoxication can be acute or chronic. Acute intoxication usually is the result of self-inflicted or accidental exposure. Acute poisoning results in pharyngitis, gastroenteritis, vomiting, and bloody diarrhea initially, followed by such systemic effects as anuria with uremia, stomatitis, ulcerative-hemorrhage colitis, nephritis, hepatitis, and circulatory collapse. The inhalation of mercury vapor or dusts leads to the typical symptoms of mercury poison with lesions of the mucous membranes.

Chronic mercury poisoning results from exposure to small amounts of mercury over extended periods of time. Typically, chronic poisoning by inorganic mercurials is the result of accidents or environmental contamination. Workers who are continually exposed to inorganic mercury are susceptible to chronic mercurialism. Chronic alkyl mercury poisoning, known as Minamata disease, is insidious because the onset of mercurialism can take weeks or even years. This type of poisoning is characterized by major neurological symptoms that lead to permanent damage or death.[13]

Epidemiological studies can be used to determine the safe quantity of mercury ingestion. The lowest concentration associated with methylmercury toxic symptoms is 0.2 μg/g. This corresponds to prolonged continuous exposure of 0.3 mg/70 kg body weight/day. Using a margin of safety of 10, the maximum intake from all sources (air, water, and food) is 0.03 mg/person/day.

EPA has not classified the carcinogenicity of mercury (Group D) because of inadequate evidence.

Molybdenum

Sources Molybdenum does not occur as an element in nature; it is obtained from molybdenite (MoS_2) and minor commercial ores, such as wulfenite ($PbMoO_4$), or as a by-product of copper and tungsten mining operations. It is used in metallurgy in either a metal or salt form. Principal applications include nuclear energy, military use, electrical products, petroleum industry, glass and ceramics industries, and production of pigments.

Occurrence A 1970 survey of finished waters found molybdenum in 30 percent of the samples, with a mean of 85.9 μg/L and a range of 3 to 1,024 μg/L.[2] Other surveys produced mean values of 8.0 μg/L in 30 percent of positive samples, ranging in this group from 1.1 to 52.7 μg/L, with some high readings of 1 mg/L.[2]

Health Effects Molybdenum is considered an essential trace element in humans. There is no apparent bioaccumulation of this element in animal or human tissues. Molybdenum is readily absorbed by the gastrointestinal tract, with concentration in the liver, kidneys, and bones. Chronic exposure can result in weight loss, bone abnormalities, and male infertility.

EPA has not classified the carcinogenicity of molybdenum (Group D) because of inadequate evidence.

Nickel

Sources Nickel is found in many ores, such as sulfides, arsenides, antomonides, oxides, and silicates. The chief source is chalcopyrite, with other sources including pyrrhotite, pentlandite, ganierite, niccolite, and millerite. Nickel salts are soluble in water, as are many of its compounds. Nickel is extensively used in making stainless steel, Invar, Monel, Inconel, and corrosion-resistant alloys. It is also used in ceramics, special batteries, electronics, and space applications. Nickel is not commonly found in nature as a pure metal.

Occurrence Nickel is common in drinking water. EPA's Community Water Supply Survey detected nickel in 86 percent of groundwater supplies and 84 percent of surface supplies tested.[3] In finished water supplies, a mean concentration of nickel was calculated at 34.2 μg/L, with a range from 1 to 490 μg/L. In another survey, an average value was 4.8 μg/L, with a maximum of 75 μg/L.[2]

Health Effects Nickel is an essential element for animals, but nickel nutritional deficiency has not been recognized in humans. Interpolating animal studies to humans, a nutritional need for a human would be on the order of 50 μg/day. Nickel has low toxicity comparable to zinc, manganese, and chromium—it does not accumulate in the tissues. The potential health effects are connected to heart and liver damage.

Most ingested nickel is excreted; however, some absorption from the gastrointestinal tract does occur. Acute effects of overexposure include decreased weight gain, blood and enzyme changes, and changes in organ iron content. Nickel compounds are carcinogenic via inhalation and injection in laboratory animals. However, nickel has not been shown to be carcinogenic via oral exposure. EPA has classified nickel as a probable human carcinogen (Group B1) based upon inhalation exposure.

Nitrate and Nitrite

Sources In aerobic surface waters, all inorganic and organic forms of nitrogen eventually will be converted biologically to the nitrate form. The principal sources of nitrogen in surface waters include runoff from fertilized agricultural lands, feedlot runoff, municipal and industrial wastewater discharges, leachate from sanitary landfills, atmospheric fallout, and decaying vegetation. Nitrogen in the groundwater is derived from leachate from fertilized, irrigated agriculture and septic tank discharges.

Nitrite is a salt or ester of nitrous acid formed by the action of bacteria upon ammonia and organic nitrogen. No significant concentration is found in surface water due to the prompt oxidation to nitrates. Combined nitrogen may be found concentrated in wastewater, landfills, and agricultural and urban runoff.

Nitrate is used as a fertilizer, as a food preservative, and as an oxidizing agent in the chemical industry. Nitrite is used in industry as a food preservative (sodium and potassium salts), particularly in meat and cheese.

Occurrence In groundwater surveys, 56 percent of the 1,479 water systems sampled had nitrate/nitrogen levels above 0.3 mg/L, with a mean value of 1.8 mg/L, and 1.4 percent had levels above 10 mg/L. In 409 surface supplies sampled, 43 percent had levels above 0.3 mg/L, the mean value was 1.6 mg/L, and the maximum was 21 mg/L. Levels above 10 mg/L nitrate-nitrogen were detected in 5 surface supplies (1.2 percent).[2]

A survey conducted in South Dakota found that out of 1,000 wells, 4 percent had NO_3-N greater than 100 mg/L, 9 percent had greater than 50 mg/L, 17 percent had greater than 20 mg/L, and 27 percent had greater than 10 mg/L.[3] Nitrite does not typically occur in natural water at significant levels; its presence indicates likely wastewater contamination and/or lack of oxidizing conditions.

Health Effects Serious and occasionally fatal poisonings in infants have occurred following ingestion of waters containing nitrates in excess of 110 mg/L nitrate-nitrogen. Two health hazards are identified with nitrate-contaminated waters:

- Induction of methemoglobinemia (oxygen deprivation), especially in infants.
- Possible formation of carcinogenic nitrosamines.

Acute toxicity of nitrate is caused by its rapid reduction to nitrite in the stomach. The nitrite then converts hemoglobin (blood pigment that carries oxygen) to methemoglobin. Methemoglobin does not act as an oxygen carrier, and consequently anoxia and death may ensue. This phenomenon seems to affect only infants up to about 3 months. It is reasoned that older children and adults are not so susceptible because the ratio of fluid intake to body weight is significantly lower in them than it is in infants. The MCL of 10 mg/L NO_3-N was based on epidemiological studies that showed this value to be the limit above which methemoglobinemia could occur. At this level, there appears to be little margin of safety for some infants.

The carcinogenic effects of nitrate are unclear. Some epidemiological studies have correlated gastric cancer to waters containing nitrate. However, more study is needed to develop a scientific basis to support conclusions on the carcinogenic effect of nitrates. The nitrate may be converted to N-nitroso compounds. The steps are speculated to be as follows:

- Reduction of nitrate to nitrite.
- Reaction of nitrite with secondary amines or amides in food or water to form N-nitroso compounds.
- Carcinogenic reaction of N-nitroso compounds.

EPA has not classified the carcinogenicity of nitrate and nitrite (Group D) because of inadequate evidence.

Selenium

Sources Selenium appears in soil as ferric selenite, calcium selenite, and elemental selenium. In soils at concentrations from 0.03 to 0.8 ppm (United States), selenium is more water soluble in more alkaline soil. Sedimentary rocks, such as shale, normally contain more selenium than limestone or sandstone. Selenium is generally produced when recovering copper ores.

Selenium is used in its elemental form or in several salts for electronic and photocopy applications, glass manufacture, pigments, chemicals, pharmaceuticals, fungicides, electrical apparatus, and rubber industry.

Selenium occurs naturally in four oxidation states: selenide (−II), elemental selenium (0), selenite (+IV), and selenate (+VI). Environmental contamination probably is minimized because a majority of organic selenium is selenide, which decomposes to elemental selenium, which is not absorbed. The forms that are most frequently found in water are selenite and selenate. Selenate is taken up by plants and may reach toxic concentrations. Selenite salts are less soluble than the selenates, and are reduced to elemental selenium under acidic conditions. The selenium concentration in surface water is greatly influenced by pH, being high in acidic (pH 3.0) and in alkaline waters ($pH > 7.5$).

Occurrence EPA reported 150 groundwater and 6 surface supplies containing selenium with concentrations above 10 μg/L. A groundwater survey indicated 97 percent of 671 systems with selenium levels ranging from 1 to 65 μg/L with mean values of detectable selenium of 2.7 μg/L. Surface water surveys reported levels ranging from 1.0 to 10 μg/L (mean value 4.6 μg/L).[1] In other surveys, 42 of 329 groundwater samples had a selenium concentration greater than 5 μg/L, and 10 of 329 had a selenium concentration greater than 10 μg/L. In the same surveys, 2 of 115 surface supplies had a selenium concentration greater than 5 μg/L.[3]

Health Effects Selenium is an essential dietary element, with most intake coming from food. The levels found in food reflect local soil conditions.

Some selenium compounds are toxic to humans, with hydrogen selenide being one of the most irritating and toxic compounds. The poisoning symptoms are similar to those of arsenic poisoning. In industrial situations, human exposure is through the skin and lungs as a result of exposure to dust or fumes.

Chronic exposures to selenium, either by ingestion or through inhalation of dust and fumes, have resulted in depression, nervousness, occasional dermatitis, gastrointestinal disturbances, giddiness, and a garlic odor.[6] Epidemiological studies have shown an increase in the incidence of dental caries in children when small amounts of selenium were ingested as part of their diets.

Although selenium can be toxic to humans and animals, it is usually the result of accidental exposure. Selenium reacts in vivo with other elements, protecting against heavy metal toxicity from mercury, cadmium, silver, and thallium.

Naturally occurring selenium compounds have not been shown to be carcinogenic in animals. Selenium may inhibit tumor formation. EPA has not classified the carcinogenicity of selenium (Group D) because of inadequate evidence.[3]

Silver

Sources The average concentration of silver in soil is 0.1 ppm, whereas, in the earth's crust, it is 0.07 to 0.08 ppm. Chloride sulfide, phosphate, and arsenate are insoluble. Silver nitrate is highly soluble; silver sulfate is moderately soluble.

Occurrence One survey found silver in 309 out of 677 groundwaters with concentrations ranging from 0.1 to 9 μg/L, and 50 out of 109 surface waters with silver concentrations of 0.1 to 4 μg/L.[3]

A survey of potable water in the 100 largest cities in the United States found a mean silver concentration of 0.23 μg/L, with a low of 0 and maximum of 7.0 μg/L. Another survey of drinking water produced a mean of 2.2, minimum of 0.3, and a maximum of 5 μg/L, with a frequency of detection of 6.1 percent.[2]

Health Effects Silver is a nonessential element, providing no beneficial effects from its ingestion in trace amounts. Acute toxicity can result from large single doses, and can be fatal. Poisoning victims experience pulmonary edema after exhibiting anorexia and anemia.[13]

Chronic toxicity takes the form of an unsightly blue-gray discoloration of the skin, mucous membranes, and eyes, which is called argyrosis or argyria. Apparently, besides the cosmetic changes, there are no physiologic effects. Ingestion of trace amounts of silver or silver salts results in its accumulation in the body, particularly the skin and eyes. There is some evidence that changes to the kidneys, liver, and spleen can occur. Silver shows no evidence of carcinogenicity or mutagenicity.

Sodium

Sources Sodium is the sixth most abundant element on earth and is found in nature only in combined forms. Sodium compounds are used in paper, glass, soap, textile, petroleum, chemical, and metal industries. The most common compounds are:

- Common salt ($NaCl$)
- Soda ash (Na_2CO_3)
- Baking soda ($NaHCO_3$)
- Caustic soda ($NaOH$)

Sodium is a natural constituent of raw waters, but its concentration is increased by pollution sources, such as rock salt treatment of road surfaces in freezing temperature, precipitation runoff, soaps, and detergents.

Occurrence Sodium is a major constituent in drinking water. A survey of 2,100 finished waters found concentrations ranging from 0.4 to 1900 mg/L, with 42 percent having sodium greater than 20 mg/L and 5 percent having greater than 250 mg/L.[3] In another survey of 630 systems of finished waters, concentrations were found from less than 1 to 402 mg/L, with 42 percent sampled with levels over 20 mg/L, and 3 percent with levels higher than 200 mg/L.[2]

Health Effects Sodium is associated with high blood pressure and heart disease in the "at-risk" population, composed of persons genetically predisposed to essential hypertension. In addition, certain diseases are aggravated by a high salt intake, including congestive heart failure, cirrhosis, and renal disease. Harmful effects for the population as a whole have not been conclusively shown.

Sulfate

Sources Sulfates are found in natural waters in the final oxidized stage of sulfides, sulfites, and thiosulfates, or in the oxidized state of organic matter in the sulfur cycle. Sodium, potassium, and ammonium sulfate are highly soluble in water. The SO_2^- anion also occurs frequently in rainfall in or near metropolitan areas where sulfate is produced as a fossil fuel combustion by-product. Detergents add sulfate to sewage. Industrial pollution from tanneries, steel mills, sulfate pulp mills, and textile plants may contaminate raw water.

Occurrence A study of 969 supplies found sulfates in concentrations of 1 to 770 mg/L, with a median of 4.6 mg/L. Three percent had sulfate concentrations greater than 250 mg/L.

Health Effects High levels of sulfate cause diarrhea and dehydration. Sulfate salts are absorbed by the intestine, excreted in the urine up to a cathartic dose of 1 or 2 g (1,000 to 2,000 mg/L in a single liter). After a period of adjustment to unusually high doses with diarrhea and gastroenteritis occurring, particularly in infants, a tolerance of 400 mg/L (an advisable maximum in certain situations) and greater have been noted. In some cases, the local population apparently has not suffered from routine ingestion of well waters with concentrations reported as high as 2,000 to 3,000 mg/L.[2]

Sulfate is not known to be mutagenic, carcinogenic, or teratogenic in mammals.

Thallium

Sources Thallium is found in the rare minerals crooksite, lorandite, and hutchinsonite. It is also found in minute quantities in pyrites, from which it is recovered while producing sulfuric acid. Thallium is also obtained from the smelting of lead and zinc ores. Thallium has been used in photocells as an infrared detector (in thallium bromide-iodide crystals), in glass manufacturing, in the electronics industry, as an alloy, and in pharmaceutical products to treat skin infections. Thallium oxide, in the presence of water, forms hydroxide.

Occurrence Limited data are available on thallium occurrence. It is found at concentrations of 1 to 88 μg/L in rivers draining metal mining areas.

Health Effects Thallium is a suspected carcinogen to humans and affects the kidneys, the liver, and the brain.

Vanadium

Sources Vanadium is not common in drinking water, although it may occur locally near residue piles from milling and mining operations. Particularly useful in nuclear production, it is also used as an alloy (vanadium steel or ferrovanadium), in glass manufacturing and photography.

Occurrence A survey of raw waters had a 3.4 percent detection frequency with a maximum concentration of 300 μg/L and a mean of 40 μg/L. Finished waters surveyed demonstrated comparable results.[3] Another finished water survey of the 100 largest cities in the United States recorded a median value of less than 4.3 and a high concentration of 70 μg/L.

Health Effects Health effects may be summarized as follows:[2]

- Absorption in the human body is extremely low.
- Requirements in human nutrition have not been proven, but it has been suggested as a protective against atherosclerosis.
- Acute vanadium toxicity is primarily respiratory.
- There is no evidence of chronic oral toxicity.

Vanadium in drinking water may be significant when considering the total diet due to the small contribution of vanadium in food. If vanadium is proven as a beneficial nutritional trace element, the daily contribution in drinking water may be considered beneficial.

Zinc

Sources Zinc commonly occurs in source waters and may be added to finished water through corrosion of metal pipes. Many zinc salts are highly soluble in water; others are not (carbonate, oxide, sulfide). Zinc is typically detected only in traces in natural waters. Industrial pollution, particularly in zinc mining areas, may contribute to concentrations as high as 50 mg/L.

Occurrence Zinc was found in 76 percent of source waters surveyed. The range of concentrations found was 2 to 1,183 μg/L, with a mean of 64 μg/L. The same surveys found zinc in 77 percent of finished waters, with a range of 3 to 2,010 μg/L and a mean of 70 μg/L.[2,3]

Health Effects Adverse health effects associated with zinc result more from too low an intake rather than from an excessive intake. This is not a common problem in the United States. Zinc deficiency results in growth failure, loss of taste, and hypogonadism. The adult requirement for zinc is 15 mg/day. Drinking water contributes about 3 percent of this requirement. In excess, zinc has been reported to cause muscular weakness and pain, irritability, and nausea. The level of zinc associated with these effects was 40 mg/L over a long period.[3]

High concentrations of zinc (5 to 30 mg/L) is aesthetically objectionable in drinking water due to a milky appearance and a greasy film in boiling. An astringent taste reaction can occur in concentrations higher than 20 to 30 mg/L.[2]

INORGANIC CONTAMINANTS OF AESTHETIC CONCERN

If present in sufficiently high concentrations, certain inorganic constituents may degrade the aesthetic quality of drinking water, rendering it unacceptable to consumers. The primary impacts are usually related to color and off-tastes and odors. The following sections discuss the source, occurrence, and water quality impacts of iron, manganese, hardness, hydrogen sulfide, and total dissolved solids.

Iron and Manganese

Sources Iron and manganese often occur together in water supplies and pose similar water quality and aesthetic issues. For these reasons, the two constituents are discussed together.

Iron and manganese are natural constituents of the earth's crust, with iron being one of the most abundant elements. The lithosphere contains approximately 5 percent iron and 0.1 percent manganese; thus, iron is found more frequently and in greater concentrations than manganese. Iron occurs in the silicate minerals of igneous rocks, primarily as insoluble ferric oxide and also as slightly soluble ferrous carbonate (siderate). Manganese is found in metamorphic and sedimentary rocks, with only small amounts in igneous rocks.[14] Manganese is present as manganese dioxide and manganese carbonate (rhodochrosite).

Two types of iron are found in water supplies: inorganic and organically complexed. Inorganic iron is generally associated with groundwaters low in dissolved oxygen. Organically complexed iron may be found in both groundwaters and surface waters. The formation of organic complexes and chelates may increase the solubility of iron in some waters. At pH values encountered in natural waters, it is possible that organically bound iron will be insoluble but highly dispersed. Natural color found in water is frequently due to such highly stabilized colloidal dispersions of Fe(II).

Mn(IV) does not readily form complexes with organic or inorganic ligands in water.

Occurrence Soluble iron and manganese are commonly found in groundwater, resulting from the dissolution of these elements by the action of carbon dioxide on carbonate-bearing minerals. Iron in natural waters is found as ferric bicarbonate, ferric hydroxide, ferrous sulfate, or colloidal or organically bound iron. The concentrations of iron and manganese naturally found in groundwaters generally do not exceed 10 mg/L and 2 mg/L, respectively. Manganese is commonly found in groundwaters devoid of oxygen and high in carbon dioxide as Mn(II). Under anaerobic conditions, the manganese is reduced by biological activity and released into solution.[14]

Iron and manganese are rarely found in significant concentrations in free-flowing surface waters. Exceptions include streams where industrial wastes or acid mine drainage are present, as well as some spring-fed streams.

Free-flowing streams that do not naturally contain iron or manganese can yield troublesome concentrations when impounded. Vegetation and other debris in the hypolimnion of lakes or impoundments decompose to produce conditions of high carbon dioxide and zero dissolved oxygen. Iron and manganese present in the soils, vegetation, and sediments are dissolved under these conditions, and during such events as the spring or fall overturn, the iron and manganese are distributed throughout the impoundment water. When reduced iron and manganese ions rise to the surface during this period, they contact water saturated with oxygen and are oxidized and precipitated. Thus, waters near the surface of a reservoir are likely to contain the lowest concentrations of iron and manganese. Manganese concentrations in reservoir hypolimnions have been reported to range from 2.0 to 20.0 mg/L or more.[14]

Newly formed reservoirs are often particularly susceptible to elevated levels of iron and manganese. These problems may continue for several years and be especially intense during the summer months. In some rare cases, the concentrations will decrease over time, but generally the problems persist. On the other hand, some reservoirs may have no iron and manganese problems initially but will develop them over time.

Water Quality Aesthetic Effects Water containing iron and manganese is visually objectionable to consumers because precipitation of these metals turns water to a rusty yellow-brown or black appearance. This rusty-looking or black water causes stains on plumbing fixtures and laundry. When used for irrigation, it can stain buildings and concrete surfaces. The presence of iron and manganese in drinking water can also produce objectionable tastes and other aesthetic problems and may lead to the growth of microorganisms in a water distribution system. Generally, water containing less than 0.1 mg/L of iron and 0.05 mg/L of manganese is considered acceptable to the average customer; however, some industries may require lower levels of these metals.

Hardness

Sources and Occurrence The nature and occurrence of hardness was discussed in a previous section of this chapter.

Aesthetic Effects Hardness reacts with soap compounds to precipitate as a curd, which is a nuisance for such household uses as laundry and bathing. Synthetic detergents largely have replaced soaps in many applications. These compounds are not as reactive with hardness; consequently, hardness is less of a problem in the home for these applications. Excessive hardness in water is undesirable for other reasons since it causes internal scaling of pipes, water heaters, and plumbing fixtures.

Hydrogen Sulfide

Sources and Occurrence Hydrogen sulfide (H_2S) commonly occurs in groundwater supplies. Sulfides in water result from anaerobic reduction of sulfates and bisulfides. In natural water, under normal conditions, the sulfur species exists in five stable forms: HSO_4^-, SO_4^{2-}, S^0, H_2S, and HS^-. Other species, such as thiosulfate, polysulfide, and polythionate, also exist in natural water; however, they are not thermodynamically stable. The predominance of species is pH dependent. H_2S and HS^- are prevalent below pH 8, with the H_2S component increasing from 80 percent at pH 7 to 100 percent at pH 6. H_2S is the form responsible for tastes and odors.

Water Quality and Aesthetic Effects Hydrogen sulfide is a common source of taste and odor problems, particularly for groundwater supplies. Odors associated with this constituent range from a swampy, musty odor at low concentrations, to a characteristic rotten-egg smell at more elevated concentrations. The odor threshold of H_2S in water is less than 100 mg/L, and odors from water containing 0.1 to 0.5 mg/L or greater are considered offensive.

Hydrogen sulfide increases the corrosiveness of water to metal and concrete. Sulfides also promote the growth of filamentous sulfur bacteria, leading to a general degradation of water quality. The presence of hydrogen sulfide will turn silverware black, discolor lead-based paint, and stain plumbing fixtures.

Total Dissolved Solids

Sources and Occurrence Waters with high levels of salts, measured as total dissolved solids (TDS), are most commonly found in the Midwest and Southwest areas of the United States and in areas of saline intrusion. Many of the ions included in the TDS measurement have been previously discussed in this chapter.

Water Quality and Aesthetic Effects Waters with high TDS may cause taste and odor problems or have a laxative effect on transient consumers. The dissolved solids impart a distinctive taste at values above 750 mg/L, and specific ions, such as sulfate and chloride, cause taste problems above 300 and 250 mg/L, respectively.

RESIDUAL DISINFECTANTS AND INORGANIC DISINFECTION BY-PRODUCTS

Bromate

Formation and Occurrence Bromate may be formed by the ozonation of waters containing bromide ion. Chlorine dioxide will also oxidize bromide ion into bromate ion, if photolyzed.[15] Finally, bromate can be formed by reaction of bromine with sodium carbonate.

Limited data are available on the occurrence of bromate in U.S. waters, although some European surveys have been conducted. An EPA study projected bromate occurrence if all surface waters were preoxidized with ozone. According to this study, median bromate value would be 1 to 2 μg/L, with the 90th and 95th percentiles ranging from 5 to 20 μg/L.[16]

Health Effects Bromate has been shown to be mutagenic in several test systems.[17] Animal studies indicated an increased incidence of kidney tumors at doses as low as

60 mg/L, and thyroid tumors and peritoneal mesothelioma were detected at doses as low as 250 mg/L. EPA has classified bromate as a probable human carcinogen (Group B).

Bromine

Formation and Occurrence Bromine (Br_2) is rarely used in drinking water treatment, but has been applied on a limited basis for swimming pools, cooling towers, and industrial water applications. Bromine is a dark, brownish-red heavy liquid that emits a sharp, penetrating and suffocating odor at room temperature. Bromine may be fed to water in the form of liquid bromine or as bromine chloride gas.

In water at neutral pH, the predominant form of bromine is hypobromous acid (HOBr). Above pH 8.7 and at 25°C, the hypobromite ion (OBr^-) is the major form of bromine. At lower pH values (below 6), Br_2, Br_3, bromine chloride, and other halide complexes form. Bromine and bromine chloride react with basic nitrogen compounds to from bromamines.

Health Effects Most health effects data is on bromide salts because of their pharmaceutical use. Bromide occurs normally in blood at a range of 1.5 to 50 mg/L. Sedation occurs at a plasma concentration of about 960 mg/L, corresponding to a maintenance dose of 17 $mg/kg\text{-}day^{-1}$. Gastrointestinal disturbances can occur at high doses. No data have been developed on the mutagencity, carcinogenicity, or teratogenicity of bromine.[3]

Chloramines

Formation and Occurrence Chloramines are formed by the reaction of chlorine with inorganic ammonia compounds. The predominant form is usually monochloramine, although dichloramine, trichloramine, and nitrogen trichloride may also be present. Chloramines are effective at bacterial inactivation but do not react extensively with organic material to form halogenated by-products of concern. As a consequence, chloramines are widely used as a secondary disinfectant, and less frequently as the primary disinfectant.

Health Effects In humans, observed health effects associated with monochloramine in drinking water have been limited to hemodialysis patients. Chloramines in dialysis baths cause oxidation of hemoglobin to methemoglobin and denaturation of hemoglobin. Tests conducted on healthy human volunteers to evaluate the effects of monochloramine in drinking water at doses up to 24 mg/L (short term) and 5 mg/L (for 12 weeks) showed no effects.[3] Results on mutagenicity of chloramines are inconclusive. The NAS has determined that there are not sufficient data to determine limits for humans for either acute or chronic exposure.

Chlorine

Formation and Occurrence In the gaseous state, chlorine is greenish-yellow and toxic. Chlorine may also be fed in the hypochlorite form and can occur as an impurity in the generation of chlorine dioxide. In water, chlorine reacts to form hypochlorous and hydrochloric acids. The hypochlorous acid then dissociates into hydrogen and hypochlorite ions. The distribution of chlorine forms is pH dependent—hypochlorous acid is the predominant species between pH 2 and pH 7, and hypochlorite ion is the predominant at high pH values. In aqueous systems, chlorine reacts with ammonia to form chloramines. See Chapters 14 and 19 for more extensive discussions of chlorine chemistry.

Chlorine remains the most widely used disinfectant and oxidant in U.S. drinking water systems.

Health Effects The primary health concerns for chlorine are associated with the formation of halogenated organics, such as trihalomethanes and haloacetic acids. These are discussed in Chapter 3. For chlorine itself, no evidence has been reported of reproductive or developmental effects. No systematic effects were observed in rodents following oral exposure to chlorine in distilled water at levels up to 275 mg/L over a two-year period. Chlorinated water has been shown to be mutagenic to bacterial strains and mammalian cells; however, investigations with rodents to determine the potential carcinogenicity of chlorine have been negative.[3]

Chlorine Dioxide, Chlorite, and Chlorate

Formation and Occurrence Chlorine dioxide has been used on a limited basis in U.S. drinking water systems as either a disinfectant or as a preoxidant for taste and odor control or color removal. It is an explosive gas, but it is stable in water in the absence of light and elevated temperature. Because of its instability, it is normally generated at the point of use.

In drinking water, the predominant reaction products are chlorite, chlorate, and chloride. Approximately 50 to 70 percent of the chlorine dioxide will initially react to form chlorite. A fraction of the chlorite will, in turn, be reduced to chloride.

Pure chlorine dioxide per se does not react to form THMs, although some chlorine may be present as an impurity, leading to low levels of THM formation. Also, chlorine dioxide does not react with ammonia, but it will react with other amines. Since chlorine dioxide does not react with water as chlorine does, residual chlorine dioxide may be easily separable from water with mild aeration.

Health Effects

The health effects of chlorine dioxide, chlorite, and chlorate have been evaluated separately, as discussed below.

Chlorine Dioxide. Generally, health concerns for chlorine dioxide are the same as those described for chlorite (see below), because chlorine dioxide converts to chlorite in vivo. No studies of the carcinogenic effects of ingested chlorine dioxide are available, but concentrates of ClO_2-treated water did not increase the incidence of lung tumors in mice.[3]

Chlorite. Adverse effects of chlorite on the hematological systems of laboratory animals is well documented. In a variety of reproductive effect tests, chlorite was associated with a decrease in growth rate of rat pups, and delayed neurodevelopment was reported in rat pups exposed perinatally to chlorite. A two-generation reproductive and developmental study by the Chemical Manufacturers Association found neurodevelopmental effects at 6 mg/kg-day^{-1}.[18] Mutagenicity testing of chlorite with mice has been negative, and no tumorigenic activity has been observed in animals given an oral dose of chlorite.[3] From animal studies, a Suggested No Adverse Response Level (SNARL) of 0.38 mg/L has been derived with an uncertainty factor of 100.[2]

Chlorate. Oral studies with chlorate demonstrate the effects on hematological parameters and formation of methemoglobin, but at much higher doses than chlorite (between 157 and 256 mg/kg-day^{-1}). No carcinogenicity or reproductive effect tests have been conducted. Data were inadequate to develop a limitation for chlorate as part of the Stage 1 D/DBP rule.[3] Using concentrated solutions of chlorate in humans, a lethal dose of 71 mg/kg.[2]

Iodine

Formation and Occurrence Historically, iodine is better known for its use as a disinfectant for abrasions and other skin wounds than for disinfecting water systems. It has been used in piped water systems only in emergency situations, and then only in small, independent utilities.[19] An iodine residual of about 1.0 mg/L is required for effective disinfection.

Iodine reacts in water to form hypoiodous acid (HOI). In water at pH above 4.0, HOI undergoes disassociation to form the hypoiodite ion (OI^-). At pH values above 8.0, HOI is unstable and will not form OI^-, but decomposes to form iodate (IO_3^-) and iodide (I^-). Iodine is not likely to form with ammonia, and does not combine with many organic compounds.

Health Effects Iodine is an essential trace element, required for the synthesis of the thyroid hormone. The estimated adult requirement is 80 to 150 μg/day. Most intake is from food and table salt.[18] Concern has been expressed over the possible harmful effects of iodinated water on the thyroid function; however, field analyses of human users of water treated with iodine have failed to detect an adverse effect. Iodine is an irritant, with acute toxicity caused by irritation of the GI tract. A dose of 2 to 3 g may be fatal. No chronic data are available on iodine.[3] The NAS has calculated a Suggested No Adverse Response Level of 1.19 mg/L, considering a 20 percent intake from water with a 7-day exposure calculated at 16.5 mg/L.[2]

Ozone

Formation and Occurrence Ozone is one of the most potent oxidants used in water treatment. It is widely used in Europe (more than 1,000 systems) and is gaining widespread use in the United States (more than 100 systems) for both disinfection and chemical oxidation applications. A pale blue gas with an acrid odor, ozone is relatively unstable in air and must be produced on-site.

In water, ozone dissolves at acid pH values (below 7), and is 11.5 times more soluble than oxygen. It does not react with water and is present as the molecule O_3. As the pH is elevated, spontaneous decomposition of ozone occurs through a variety of steps to produce very reactive free radicals. Because of its instability, an ozone residual cannot be maintained for extended periods and essentially no ozone is present when the finished water reaches consumers.

Health Effects Because of its dissipation, there are no health concerns related to ozone itself. Instead, health effect issues are associated with ozonated by-products, such as bromate and formaldehyde.

Radionuclides

Radioactivity is the ability of a substance to emit positively or negatively charged particles, and sometimes electromagnetic radiation, by the disintegration of atomic nuclei. Three forms occur:

- Alpha particles, which are positively charged, have a mass of 4, and are doubly charged ions of helium
- Beta particles, which are negatively charged (electrons) and move at about the speed of light
- Gamma rays, which are electromagnetic radiation (photons) and travel at the speed of light

Elements heavier than lead, such as radon, radium, thorium, and uranium isotopes, decay by the release of alpha, alpha and gamma, or beta and gamma emissions. Radionuclides lighter than lead generally decay by beta and gamma emissions.[3] Radioactivity is normally reported in units of curies (Ci), rads, or rems. A commonly used unit is the picocurie (pCi), which equals 10^{-12} Ci.

Nearly all radionuclides of concern in drinking water supplies are naturally occurring, and are members of three radioactive series: the uranium series, the thorium series, and the actinium series. The specific elements of concern are radium (Ra), radon (Rn), and uranium (U). The isotopes in the uranium series that may pose a health risk due to their presence in drinking water are ^{238}U, ^{234}U, ^{226}Ra, and ^{222}Rn. Radium-228, which occurs in the thorium series, is also found in drinking water. Very few of the other isotopes in this series have been detected in drinking water.[20] Radium-228 is a beta emitter, whereas the others are all alpha emitters.

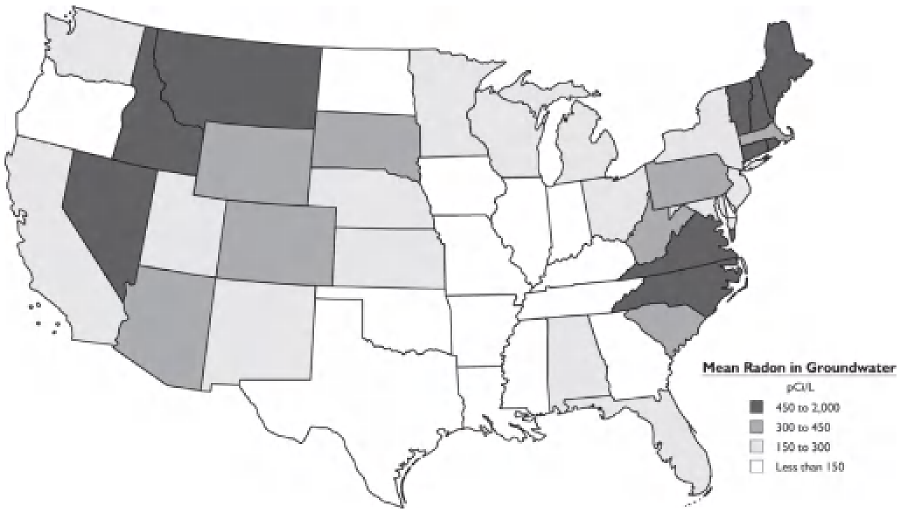
Radon

Occurrence Radon volatizes quickly from water when exposed to atmosphere; consequently, it is found only in groundwaters. Smaller water supplies tend to take their water from smaller aquifers, which generally have larger granular surface areas and, thus, and higher concentrations of radon. Consequently, radon concentrations tend to increase as system size decreases.[21]

The minimum detection level for routine analytical measurements for radon in water is in the range of 50 to 100 pCi/L. The average radon concentration in U.S. groundwaters is 200 to 600 pCi/L, and the highest recorded value is 750,000 pCi/L.[20]

The occurrence of radon in groundwater supplies varies widely on a geographical basis. Figure 2–3 illustrates the general pattern of radon occurrence in U.S. groundwaters.[23]

Health Effects Inhaled radon has been shown in several epidemiological studies to lead to lung cancer.[22] Radon gas decays into radioactive particles that become trapped in the lungs. As the particles break down, they release small energy bursts that can damage lung tissue and increase the chance of developing lung cancer over a lifetime. Radon in indoor air is the second-leading cause of lung cancer, resulting in 20,000 deaths per year. Only about 1–2 percent of radon in air comes from drinking water; however, breathing radon released to air from tap water increases the risk of lung cancer. Some radon remains in the water. Drinking this water presents a risk of



Source: USEPA NIRS Survey, 1985.

Note: State averaging of data may obscure local variations in radon levels.

Fig. 2–3. General patterns of radon occurrence in groundwater in the United States (*Source:* Reference 23)

developing internal organ cancers, primarily stomach cancer; however, this risk is smaller than the risk of lung cancer due to inhaled radon from drinking water. Based on an NAS report, EPA estimates that radon in drinking water causes about 168 deaths per year: 89 percent from lung cancer caused by breathing radon released to indoor air from water and 11 percent from stomach cancer caused by consuming water containing radon.[20]

Radium

Occurrence Radium is found primarily in groundwater. Based on review of compliance records for drinking water systems, one survey estimated that 500 of the approximately 60,000 public drinking water supplies have radium levels that exceed 5 pCi/L. The largest concentration that has been measured in a public drinking water system is approximately 100 pCi/L; the mean for ^{226}Ra is about 0.4 pCi/L and the mean for ^{228}Ra is about 0.7 pCi/L.[20]

Although the occurrences of ^{226}Ra and ^{228}Ra are independent (they occur in two different radioactive series), in general, their nationwide distributions with respect to concentration are somewhat similar. The highest levels are found in the Piedmont region of North Carolina; South Carolina and Georgia; the midwestern area of Iowa, northern Illinois, and southern Wisconsin; and the ‘‘four corners’’ region in the West.[20]

Health Effects Radium, biochemically similar to calcium and barium when ingested, concentrates in the bone. The health effects of ingested radium are well documented and are largely based on the studies of radium watch dial painters in the early part of this century who came into oral contact with radium as part of the work process.

Bone sarcomas and head carcinomas were diagnosed among this group, but no statistically significant number of leukemias was found.[20]

Uranium

Occurrence Natural uranium may be found in both surface and groundwater. The highest levels of uranium are found in the mountainous region of the western United States. In the 1960s and 1970s, the USGS conducted the National Uranium Resource Evaluation (NURE), collecting 89,000 surface water and groundwater samples. The largest concentration of uranium measured in this study was 600 pCi/L, although a measurement of 6,900 pCi/L has since been reported from a private well in Colorado. The Oak Ridge National Laboratory identified approximately 20,000 of the sites in the NURE study as existing or potential drinking water supplies. The mean concentration in these supplies was 1.8 pCi/L.[20] Other surveys report average concentrations of 2.4 and 3 pCi/L for U.S. groundwaters.[2,3]

Health Effects Ingested uranium goes primarily to the bone and kidney. The low radioactivity per gram of uranium makes it difficult to develop dose–response studies in animals. Limited evidence suggests the health effects data from radium may serve as a surrogate for uranium. Uranium is also known to have deleterious or chemotoxic effects on the kidney.[20]

REFERENCES

1. Reiber, S. H., and Kukull, W. A., *Aluminum, Drinking Water, and Alzheimer's Disease*. AWWA Research Foundation, Denver, CO, 1996.
2. DeZuane J., *Handbook of Drinking Water Quality*, John Wiley & Sons, New York, 1997.
3. Cohn, P. D., Cox, M., and Berger, P. S., ‘‘Health and Aesthetic Aspects of Water Quality,’’ *Water Quality and Treatment*, American Water Works Association, 5th Edition, McGraw-Hill, New York, p. 63, 1999.
4. Stauber, J. L., Florence, T. M., Davies, C. M., Adams, M. S., and Buchanan, S. J., ‘‘Bioavailability of Al in Alum-Treated Drinking Water,’’ *JAWWA* 91:11, 1999.
5. USEPA, ‘‘Contaminant Specific Fact Sheets,’’ Office of Ground Water and Drinking Water Web Page, January 2000.
6. *Drinking Water and Health*, Vol. 105, Safe Drinking Water Committee, National Academy Press, Washington, DC, 1980, 1982, 1983.
7. USEPA, ‘‘Arsenic in Drinking Water—Health Effects Research,’’ Office of Ground Water and Drinking Water Web Page, January 2000.
8. Hesse C. S., and Hellenbeck W. H., ‘‘Chrysotile Asbestos in Raw Water,’’ *Asbestos in Potable Water,* Office of Water Resources and Technology, January 1977.
9. Canto, K. P., ‘‘Drinking Water and Cancer,’’ *Cancer Causes Control* 8:292–308, 1997.
10. Sidorenko, G. I., et al., *Guidelines on Health Aspects of Water Desalination*, World Health Organization, 1981.
11. National Academy of Sciences Safe Drinking Water Committee, *Drinking Water and Health,* Vol. 3, National Academy Press, Washington, DC, 1980.
12. Faust S. D., and Alay O. M., *Chemistry of Water Treatment,* Ann Arbor Science, Woburn, MA, 1983.
13. USEPA, *National Interim Drinking Water Regulations*, Office of Water Supply, EPA-570/9–76–003.

14. Griffen, A. E., ‘‘Significance and Removal of Manganese in Water Supplies,’’ *JAWWA* 52: 1326, 1960.
15. Gordon, G., and Bubnis, B., ‘‘Bromate Ion Formation in Water When Chlorine Dioxide Is Photolyzed in the Presence of Bromide Ion,’’ Proceedings of the AWWA, Water Quality Technology Conference, New Orleans, November 1995.
16. USEPA, ‘‘National Primary Drinking Water Regulations; Disinfectants and Disinfection By-products; Proposed,’’ *Federal Register* 59:38668–38829, 1994.
17. USEPA, ‘‘National Primary Drinking Water Regulations; Disinfectants and Disinfection By-products Notice of Data Availability; Proposed Rule,’’ *Federal Register* 63, 1998:15673–15692.
18. National Academy of Sciences Safe Drinking Water Committee, *Drinking Water and Health,* Vol. 7, National Academy Press, Washington, DC, 1987.
19. National Academy of Sciences Safe Drinking Water Committee, *Drinking Water and Health,* Vol. 2, National Academy Press, Washington, DC, 1980.
20. Lappenbusch, W. L., and Cothern, C. R., ‘‘Regulatory Development of Interim and Revised Regulations for Radioactivity in Drinking Water—Past and Present Issues and Problems,’’ *Health Physics* 48:525–551, 1985.
21. Cothern, C. R., and Rebers, P. A., Radon, *Radium and Uranium in Drinking Water,* Lewis Publishers, 1990.
22. USEPA, Radon Fact Sheets, Office of Ground Water and Drinking Water Web Page, January 2000.
23. USEPA, ‘‘Nationwide Occurrence of Radon and Other Natural Radioactivity in Public Water Supplies,’’ National Air and Environmental Radiation Laboratory, EPA 520–5–85–008 Washington, DC, October 1985.

CHAPTER 3

Organic Compounds in Drinking Water

INTRODUCTION

It is estimated that over 5,000 new organic compounds are brought to market each year. The vast majority of these represent no health threat, nor do they threaten contamination of the nation’s water supplies. Nonetheless, this constant new production of compounds, coupled with an ever-improving analytical capability, means that the detection of heretofore unreported organic compounds in both ground and surface waters is a growth industry. It is essential that water industry professionals understand the nature and source of the organics threat, and the growing body of drinking water regulations governing these compounds. It is impossible for a single chapter, or even a single book, to adequately cover this topic. This chapter will focus on the currently regulated volatile and synthetic organic compounds, natural organic matter that serves as the precursor for DBPs, and the most important emerging organic chemical threat of the past decade, MTBE.

ORGANICS MONITORING

There are currently over 150 drinking water chemical contaminants that have been either proposed listed or finalized for regulation. Of this number, over 100 are organic compounds. Regulated contaminants have risen substantially over the past two decades and will continue to rise into the foreseeable future. Which utilities monitor for which organic compounds (or group of compounds) and with what frequency is usually determined based on the size of the utility, the nature of the water source, and the history of contamination. Because of the substantial expense of monitoring and the serious consequences of violating an MCL, proper collection techniques, chain of custody, and analysis is crucial. Table 3–1 presents the sampling methods and criteria for different classes of organic compounds.

VOLATILE ORGANIC COMPOUNDS

Volatile organic compounds (VOCs) are so called because of their distinctive common property, which is high volatility relative to most other organic substances. A practical definition of a VOC is an organic compound having a Henry’s constant (see Chapter 9, ‘‘Aeration’’) greater than 10. Table 3–2 lists the 16 VOCs of greatest concern in

TABLE 3–1. Sampling Criteria for Organic Compound Groups of Interest in Drinking Water

Parameter	Method	Amount	Container(s)	Preservative	Hold Time
THMs	EPA 502.2	120 mL	3 40-mL G vials	Cool 4°C, $Na_2S_2O_3$	14 days
VOCs (full suite)	EPA 502.2	120 mL	3 40-mL G vials	Cool 4°C, $Na_2S_2O_3$, HCl in field	14 days
EDB/DBCP	EPA 504.1	80 mL	2 40-mL G vials	Cool 4°C, $Na_2S_2O_3$	28 days
Pesticides	EPA 505	80 mL	2 40-mL G vials	Cool 4°C, $Na_2S_2O_3$	7, 14 days
Phthalate esters	EPA 506	2000 mL	2 1-L amber G	Cool 4°C, $Na_2S_2O_3$	14 days
N-P Pesticides	EPA 507	2000 mL	2 1-L amber G	Cool 4°C, $Na_2S_2O_3$	14, 14 days
Pesticides	EPA 508	2000 mL	2 1-L amber G	Cool 4°C, $Na_2S_2O_3$	7, 14 days
Herbicides	EPA 515.1	2000 mL	2 1-L amber G	Cool 4°C, $Na_2S_2O_3$	14, 28 days
Volatiles	EPA 524.2	120 mL	3 40-mL G vials	Cool 4°C, Ascorbic acid, HCl in field	14 days
Semivolatile Organics	EPA 525.2	2000 mL	2 1-L amber G	Cool 4°C, Sodium sulfite, HCl in field	7, 30 days
Carbamates	EPA 531.1	40 mL	1 40-mL G vial	Cool 4°C, $Na_2S_2O_3$, MCA in field	28 days
Glyphosate	EPA 547	125 mL	1 125-mL amber G	Cool 4°C, $Na_2S_2O_3$	14 days
Endothall	EPA 548.1	250 mL	2 125-mL amber G	Cool 4°C, $Na_2S_2O_3$	7, 1 days
Diquat	EPA 549.1	2000 mL	2 1-L amber P	Cool 4°C, $Na_2S_2O_3$	7, 21 days
PAHs (PNAs)	EPA 550	2000 mL	2 1-L amber G	Cool 4°C, $Na_2S_2O_3$, HCl	7, 40 days
D/DBP	EPA 551	80 mL	2 40-mL G vials	Cool 4°C, Ammonium chloride pH 5	14 days
Haloacetic acids	EPA 552.1	150 mL	2 125-mL amber G	Cool 4°C, Ammonium chloride pH 5	28 days
Halogenated volatiles	8010B *S*	100 g	1 4-oz G jar	Cool 4°C	14 days
Nonhalogenated volatiles	8015A *S*	100 g	1 4-oz jar	Cool 4°C	14 days
Aromatic volatiles	8020 A *S*	100 g	1 4-oz G jar	Cool 4°C	14 days
Halogenated and Aromatics	8021A *S*	100 g	1 4-oz G jar	Cool 4°C	14 days
Phenols	8040A *S*	100 g	1 8-oz G jar	Cool 4°C	14 days
Phthalate esters	8061 *S*	100 g	1 8-oz G jar	Cool 4°C	14 days
Pesticides and PCBs	8080A/8081	100 g	1 8-oz G jar	Cool 4°C	14, 40 days
PAHs (PNAs)	8100 *S*	1000 mL	1 1-L G amber	Cool 4°C	14 days
Organophosphorus pesticides	8141 *S*	100 g	1 8-oz G amber	Cool 4°C	14, 40 days
Chlorinated herbicides	8150B/8151	100 g	1 8-oz G jar	Cool 4°C	14 days
Volatile organics (GC/MS)	8260A *S*	100 g	1 8-oz G jar	Cool 4°C	14 days
Semivolatiles	8270B *S*	100 g	1 8-oz G jar	Cool 4°C	14, 40 days
TPH	418.1 *S*	100 g	1 8-oz G jar	Cool 4°C	14 days
Purgeable halocarbons	601/8010*W*	80 mL	2 40-mL G vials	Cool 4°C, 1:1, HCl	14 days

TABLE 3–1. (*Continued*)

Parameter	Method	Amount	Container(s)	Preservative	Hold Time
Purgeable aromatics	602/8020*W*	80 mL	2 40-mL G vials	Cool 4°C, 1:1, HCl	14 days
Phenols	604/8040*W*	1000 mL	1 1-L G amber	Cool 4°C	7, 40 days
Phthalate esters	606/8060*W*	1000 mL	1 1-L G amber	Cool 4°C	7, 40 days
Pesticides and PCBs	608/8080*W*	1000 mL	1 1-L G amber	Cool 4°C	7, 40 days
PAHs (PNAs)	610/8010*W*	1000 mL	1 1-L G amber	Cool 4°C	14 days
Organophosphorus	614/8141*W*	1000 mL	2 1-L G amber	Cool 4°C	7, 40 days
Chlorinated herbicides	615/8150*W*	1000 mL	1 1-L G amber	Cool 4°C	7, 30 days
Purgeable hydrocarbons	624/8260*W*	80 mL	2 40-mL, G vials	Cool 4°C, 1:1 HCl	14 days
Semivolatiles	625/8270*W*	1000 mL	1 1-L G amber	Cool 4°C	7, 40 days

Source: See Reference 1.

Notes: P = plastic; G = glass.

TABLE 3–2. Physical Properties of Common VOCs

Compound	Molecular Weight g/mol	Density g/mL	Boiling Point °C at 1 atm	Solubility mg/L	Vapor Pressure mm Hg	Henry's Constant** $\frac{(\text{atm-m}^3)\ 10^{-3}}{\text{mol}}$
Trichloroethylene	132	1.46	86.7	1,100	74	11.7
Tetrachloroethylene	166	1.62	121	140	18.6	28.7
cis-1,2-Dichloroethylene	97	1.29	60	3,500	206	8
trans-1,2-Dichloroethylene	97	1.26	48	6,300	271	5.2
1,1-Dichloroethylene	97	1.22	32	40	495	150
Vinyl chloride	62	0.92	−14	60	2,660	6,400
1,1,1-Trichloroethane	133	1.34	74.1	4,400	100	4.92
1,2-Dichloroethane	99	1.24	83.5	8,700	82	1.10
Carbon tetrachloride	154	1.59	76.7	800	91.3	30.2
Methylene chloride	85	1.33	40	19,400	438	3.19
Benzene	78	0.89	80	1,780	95	5.55
Chlorobenzene	113	1.11	132	448	15	3.93
1,2-Dichlorobenzene	147	1.31	180	100	1.0	1.94
1,3-Dichlorobenzene	147	1.29	173	123	2.0	1.94
1,4-Dichlorobenzene	147	1.25	147	79	1.0	1.94
1,2,4-Trichlorobenzene	182	1.45	219	30	0.29	1.42

Source: See Reference 2.

drinking water, along with their physical properties. Several of these VOCs are recognized as a threat to public health that in some instances must be dealt with by removing the chemical from the water supply by appropriate treatment. Table 1–6 lists the current MCL for all VOCs the EPA has determined to be of significant concern.

VOCs are seldom detected in concentrations greater than a few micrograms per liter in surface waters, because the compounds do not occur naturally and are relatively volatile. However, surface water subject to wastewater discharges may contain elevated concentrations of organic solvents during periods of ice cover when volatilization of these solvents is restricted.

The risk of groundwater contamination by VOCs is substantially more serious than that for surface water. VOCs can enter an aquifer and be transported great distances because they have little affinity for soils. Symons and colleagues estimated in 1975 that approximately 1 percent of the nation's groundwater was thought to be meaningfully contaminated by organic pollutants.[3] However, this estimate is only a rough approximation based on incomplete surveys. Moreover, detection of groundwater contamination is inherently characterized by time lags (sometimes measured in decades) because of the distance between the site of the contamination and the wellhead.

Groundwater provides a unique environment for VOCs because:

- It has limited contact with the atmosphere; hence, volatile compounds do not evaporate quickly.
- The surface environment below the active soil zone is relatively abiotic, allowing little biodegradation.
- The temperature of groundwater undergoes slow and limited fluctuations throughout the annual climatic cycle.
- The groundwater moves slowly, without turbulence, in a dark environment.

When VOCs are introduced into a groundwater system, they maintain a discrete flow pattern because of laminar flow conditions. Dispersion and diffusion alone are responsible for what limited dilution takes place. Thus, the characteristics of an aquifer tend to preserve, rather than dissipate, VOCs.

An adequate understanding of the extent of contamination within a given aquifer requires a thorough understanding of the aquifer geology and groundwater movement within that aquifer, as well as analyses from nearly every groundwater discharge within the aquifer. Contamination can be caused by a single discharge (such as a railroad accident) and remain undetected for several years. In cases of very slow groundwater movement, a contaminant may remain localized and impact only a small area. In any situation, predicting the likelihood of contamination at a particular point without a large database is extremely difficult. Often, the necessary data on aquifer contamination can be obtained only through the development of a comprehensive groundwater monitoring program. Excellent discussion of the design and installation of monitoring well networks is available from several sources.[4]

Several federal surveys, including the National Organics Reconnaissance Survey, the National Organics Monitoring Survey, and many state surveys, have identified VOCs in numerous groundwater-supplied potable water systems.[5] Although monitoring has been concentrated in areas of suspected problems, particularly New England and the mid-Atlantic states, it is now recognized that groundwater supplies in all areas of the country are suspect. A summary of selected monitoring data from the state studies indicating maximum contaminant levels is shown in Table 3–3.

TABLE 3–3. Summary of VOC Occurrence Data Taken from State Surveys

Chemical	No. States Tested*	No. Wells	% Positive	Max. μg/L
Trichloroethylene	8	2,894	28	35,000
Carbon tetrachloride	4	1,659	10	379
Tetrachloroethylene	5	1,652	14	50
1,2-Dichloroethane	2	1,212	7	400
1,1,1-Trichloroethane	3	1,611	23	401,300
1,1-Dichloroethane	9	785	18	11,330
Dichloroethylenes (3)	8	781	23	860
Methylene chloride	10	1,183	2	3,600
Vinyl chloride	9	1,033	7	380

Source: See Reference 5.

*Ratio of community wells to private wells unknown.

The monitoring data indicate that VOCs can occur at disturbingly high concentrations, often orders of magnitude higher than those found in raw or treated drinking water drawn from the most contaminated surface supplies. Generally, the concentrations of VOCs in groundwater have been several hundred micrograms per liter, with some instances of concentrations in the milligrams-per-liter range.

The monitoring data from both federal and state studies reveal interesting characteristics of affected groundwater supplies.[6]

- An affected groundwater supply typically contains several VOCs.
- Trichloroethylene (TCE), an industrial solvent and degreaser, has been detected most frequently and in the highest concentrations.
- Tetrachloroethylene (PCE) ranks second in occurrence.
- Within a specific well, at least one or two organic compounds at relatively high concentrations (100 to 500 μg/L) will likely predominate, with several other identifiable compounds present at lower concentrations (less than 50 μg/L).
- A given well field may include one well with a preponderance of one or two compounds at high concentrations, whereas in another well in the same area several different compounds may dominate.
- A groundwater system with all of its wells affected to the same extent is unlikely.

Sources of VOCs

Volatile organic chemicals are a widely employed in many types of industrial, commercial, agricultural, and household activities. Presently, VOCs are produced at a rate of over 20 billion pounds per year.[7] Both the multitude of uses and the magnitude of production of VOCs contribute to the introduction of these contaminants into the environment. It is generally perceived that most VOC contamination of groundwater is the result of improper surface or underground disposal of hazardous waste from industrial activities. Groundwater contamination may also occur as a result of activities not intended for waste disposal, such as accidental spills or leaking storage tanks.

Table 3–4 lists the major uses of the six VOCs most frequently detected in water supplies. This list illustrates the vast number of pathways by which VOCs can enter

TABLE 3–4. Use of Most Commonly Occurring VOCs

Substances	Uses
Trichloroethylene	Mainly as a degreasing solvent in metal industries. Common ingredient in household products, such as spot removers, rug cleaners, air fresheners, dry-cleaning fluids, refrigerants, and inhalation anesthetics.
Tetrachloroethylene	Mainly as a dry-cleaning solvent in commercial and coin-operated systems. Used as textile scouring solvent; dried vegetable fumigant; rug and upholstery cleaner; stain, spot, lipstick, and rust remover; printing ink ingredient; heat transfer media; chemical intermediate in the production of other organic compounds; and metal degreaser.
Carbon tetrachloride	Mainly in manufacture of fluorocarbons used as refrigerants, foam-blowing agents, and solvents. Used in fumigants, although use in grain fumigation is decreasing; minor uses in metal cleaning and manufacturer of paint and plastics. Banned for use in consumer goods in 1970 and as aerosol propellant in 1978.
1,1,1-Trichloroethane	Mainly in metal cleaning. Used for leather tanning, vapor depressant in aerosols and solvent for adhesives, septic tank degreasers, drain cleaners, inks, shoe polishes, cutting oils, and many other products.
1,2-Dichloroethane	Mainly as intermediate in manufacture of vinyl chloride monomers. Use as intermediate in manufacture of chlorinated solvents such as tetrachloroethylene, trichloroethylene, and 1,1,1-trichloroethane; as solvent for cleaning textiles, cleaning PVC-processing equipment, processing pharmaceutical equipment, extracting oil from soil seeds; and in manufacturing paints, coatings, and adhesives, fumigating stored grain products, and lead-scavenging additives.
Vinyl chloride	Mainly in the manufacture of plastics, polyvinyl chloride (PVC) resins, and polyvinyl chloride fabrication. Vinyl chloride and PVC used as raw materials in various industries such as rubber, glass, paper, and automotive; and in manufacture of electrical wire insulation and cables, pipe, industrial and household equipment, medical supplies, food-packaging materials, and building and construction products.

Source: See Reference 5.

the environment, and may prove useful in attempts to isolate specific causes of contamination.

Alternatives for Controlling VOCs

When a water supply is found to be contaminated with VOCs, various strategies are available to address the problem. These strategies may be classified as either management or treatment strategies. Selection of the proper strategy for a specific contamination problem may require an extensive evaluation of a wide range of factors. Among the most important criteria to be considered are:

- Public opinion
- Regulatory agency acceptance
- Cost

- Long-term effectiveness
- Ease of implementation
- Reliability

Most of the public water supply systems likely to be affected by VOC contamination are small, decentralized systems, serving fewer than 10,000 people. For these systems, the following additional considerations should be heavily weighed in evaluating control strategies:

- The existing level of treatment in small systems typically consists only of chlorination; thus, any treatment technique will represent a new and more complicated technology.
- Small water utilities have limited resources; thus, both capital and operating costs may be significant obstacles in controlling VOCs.
- Operators of small systems typically have other duties in addition to water plant operation and maintenance; therefore, simplicity in operation and maintenance, including monitoring, should be given special consideration.

Management Strategies

The long-term interest of both the water purveyor and the consumer would best be served by the availability of an uncontaminated source. The strategies available for accomplishing this goal are primarily management strategies, involving control of the water supply source to reduce or eliminate the presence of chemical compounds. These strategies can be categorized as the following:

Elimination of the Contaminant Source If the source of the VOCs can be identified and the size of the groundwater aquifer is small, elimination of the source may be an effective solution. This can be accomplished by purging the groundwater, treating the pumped water, and returning the water to replenish the aquifer. However, sources of organic contaminants are often difficult to locate because of the ease with which the chemicals migrate through the soil. Additionally, the size of the affected aquifer and the degree of infiltration may be such that many years would be required to purge the aquifer even after the source is identified and eliminated. For these reasons, eliminating the source of the compounds may be a management technique that is practical only for a limited number of affected utilities.

Containment of the Contaminant This technique involves the use of one or more purge wells to halt further migration of the VOCs into a well field, thus protecting the remaining wells from increased contamination. In order for this technique to be effective, the contaminant must be detected early, before extensive contamination of the aquifer has occurred. Also, a thorough understanding of groundwater movement is necessary to halt contaminant migration. One problem with this approach is the disposal of the contaminant water removed by the purge wells.

Locating a New Supply Source Locating a new supply source involves developing a new well in an unaffected aquifer, tapping a surface water source, or purchasing water from a neighboring community. This approach has been utilized by several communities. However, this method may not be practical because:

- An unaffected source of supply may not be available nearby, and the cost of developing a new source that is far removed from the service area may be prohibitive.
- Developing a new groundwater supply would not eliminate the possibility of the compound migrating to the new supply.

A neighboring community's supply may not be capable of providing enough additional water to replace a large supply. Consequently, this type of control may only be practical for very small systems, or for systems where only a portion of the supply has been affected.

Blending A fourth management strategy for systems with multiple wells is to blend water from several wells to reduce the concentration of the compound by dilution. Depending on the levels of the compounds in each well, blending water from the wells prior to pumping into the distribution system could reduce the concentration of a specific compound to acceptable levels. Use of this approach may be limited due to lack of system flexibility, insufficient dilution, or lack of consumer acceptance.

Treatment Strategies Other than some incidental evaporative losses, conventional water treatment consisting of coagulation, sedimentation, filtration, and chlorination has been found to be largely ineffective for reducing the concentration of VOCs. Table 3–2 lists properties of selected VOCs that are pertinent to removal and treatment techniques. Of greatest importance is the Henry's constant, which gives a relative indication of how a volatile compound will partition itself between liquid and gas phases. The high values for most VOCs indicate that the compounds are hydrophobic and their tendency is to readily partition into the gas phase. Thus, in most cases they are amenable to air stripping and other aeration processes. Because of their hydrophobicity, they can also be removed by sorption onto activated carbon; however, in most circumstances air-stripping is a far more economical approach than activated carbon sorption. Chapter 9 provides a detailed discussion of treatment, design, and cost considerations for the removal of the VOCs of greatest interest.

SYNTHETIC ORGANIC COMPOUNDS

Synthetic organic compound (SOC) is an obviously broad catchall term generally used to categorize nonvolatile organics of industrial origin. The major SOC groups of concern in drinking waters consist largely of agricultural chemicals, such as pesticides and herbicides. Organic pesticides gained prominence immediately before and during World War II. In the past half century, several hundreds of different pesticides and herbicides have been brought to market, while the total annual production of these chemicals runs into the billions of pounds. Overuse and inappropriate use of pesticides threaten some surface waters and are a serious concern for many rural groundwater utilities.

Another group of SOCs of concern are polychlorinated biphenyls (PCBs), most often sold under the trade name Arochlor. These represent a group of remarkably refractory compounds that were produced in great volume (prior to their ban in the 1980s) and commonly used as high temperature stabilizers for transformer oils. Resistant to both chemical and biological degradation, these compounds are environmentally persistent and continue to contaminate the sediments of many waterways in urban-industrial settings. Drinking-water MCLs have been established for a number of pesticides, herbicides, and PCBs. Table 1–5 presents MCLs, and suspected health effects, for all currently regulated SOCs.

Much of the concern about pesticides and herbicides relates to the ability of aquatic plants and animals (including humans) to store and concentrate these substances in their tissues, particularly fatty tissues. With some pesticides there can be a remarkable magnification effect, as the concentration of the pesticide increases with trophic level in the food chain. Ultimately, the levels achieved can be several thousand times greater than the ambient concentration of the pesticide in the water body. The chlorinated SOCs have been singled out for special consideration because of their resistance to biological degradation (environmental persistence). These compounds in particular have a remarkable longevity that in some cases span decades and make their inappropriate usage a true environmental hazard. Research to develop more effective and environmentally acceptable pesticides has produced many new classes and types of compounds. Table 3–5 presents a chart of the major classes of insecticides and herbicides, their structure and information relative to their toxicity and environmental persistence.

NATURAL ORGANIC MATTER

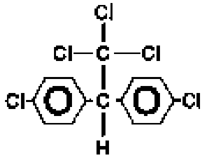
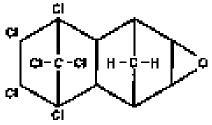
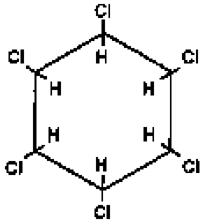
The Most Problematic Organics Are Natural

Diverse organic compounds generated by biological processes both in a water body (autochthonous material) and in the surrounding watershed (allochthonous material) are found in all surface and many ground waters. Collectively, these compounds, along with some organic compounds that enter the water as a result of human activities, are referred to as *natural organic matter* (NOM). The concentrations of NOM and of the subgroups of molecules that contribute to it are usually quantified in terms of the amount of carbon in the molecules. Typically, values are reported as the concentration of dissolved organic carbon (DOC) in a sample. The more inclusive term *total organic carbon* (TOC) includes particulate and colloidal material.

Natural waters can contain a considerable quantity of organic matter (see Fig. 3–1), a substantial portion of which may be particulate material—largely vegetative debris. The more important fraction is the dissolved organic carbon, which constitutes the reactive organic content, and is responsible for the majority of reactions of interest in water treatment processes. Natural waters generally contain between 2 and 10 mg/L of DOC.

Since the first application of gas-liquid chromatographic techniques and their subsequent coupling with mass spectrometry, an ever-growing number of the dissolved organic compounds have been identified. However, despite the more extensive application of these sophisticated analytical techniques, semiquantitative estimations carried

TABLE 3–5. Structure,* Toxicity,† and Persistence of the Major Pesticides and Herbicides

	Background	Environmental Persistence	Toxicity
CHLORINATED PESTICIDES ● DDT  ● Dieldrin  ● Lindane 	As a group, these are the most powerful and potentially harmful insecticides. The most notorious is DDT, which came into existence just prior to World War II. An extremely versatile insecticide, it is used mainly for mosquito control and was largely responsible for the control of malaria in the southern United States and elsewhere. Still used internationally, it has been banned in the United States for all applications since the 1960s. Two other very powerful chlorinated pesticides used extensively until the 1970s are Aldrin and Dieldrin (Endrin is the stereoisomer of Dieldrin); both have been banned for general usage. Chlorinated pesticides still in common usage include lindane, toxaphene, and heptachlor.	Months to years	Highly toxic to aquatic life and birds feeding on aquatic life. Very low acute toxicity to mammals.

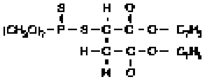
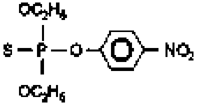
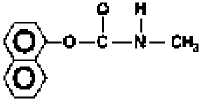
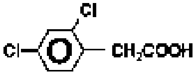
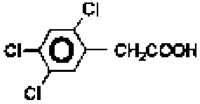
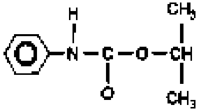
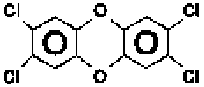
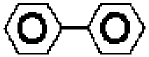
ORGANIC PHOSPHOROUS PESTICIDES ● Malathion  ● Parathion 	This class of compounds came out of chemical warfare research in Germany during World War II. Malathion is the most commonly used compound in this group and is highly toxic to a variety of insects, but displays low toxicity to mammals and is widely used in agriculture to control fruit flies. Other organic phosphorous compounds of significance include, parathion, systox, chlorothion, dicapthon, and metasystox.	Several months when dry. Hydrolyzes quickly when wet (persistence measured in days).	Minimal toxicity to aquatic life or mammals via ingestion. May be toxic to farm workers who come into direct contact with the pesticide or recently treated fruit (direct absorption across the skin).
CARBAMATE PESTICIDES ● Sevin 	The most recently developed group of pesticides, they are now in wide usage both as pesticides and herbicides. As a group they have a low toxicity to mammals and quickly decompose when exposed to sunlight and water. The most common pesticides are sevin and ferbam. Captan, which is also a member of this group, is commonly used as a fungicide.	Less than 24 hours	Very low to mammals and humans. Several carbamates are used as indoor insecticides and foggers.
CHLORINATED HERBICIDES ● 2,4-D (Dichlorophenoxyacetic acid)  ● 2,4,5-T (Trichlorophenoxyacetic acid) 	The two most widely used chlorinated herbicides are 2,4-D and 2,4,5-T. These herbicides are effective at killing broad-leaf plants, while not killing grasses. 2,4,5-T was used as a defoliant during the Vietnam War (1965–72) and was commonly referred to as Agent Orange because of the color of the drum in which the chemical was shipped. Until the 1980s, these herbicides were commonly used to kill aquatic vegetation in lakes, ponds, and reservoirs.	Months	The pure herbicide is nontoxic to mammals. A side product and contaminant of the 2,4,5-T is the chemical dioxin. Dioxin is thought to be extremely toxic to humans, although there is much conflicting epidemiological evidence on this point.

TABLE 3–5. (*Continued*)

	Background	Environmental Persistence	Toxicity
CARBAMATE HERBICIDES ● IPC 	As a group, the carbamate herbicides have found wide application in agriculture because of low toxicity to mammals and minimal environmental persistence. The carbamate pesticide that has achieved the greatest use is isopropyl N-phenylcarbamate (IPC). IPC is effective for the control of grasses without affecting broad-leaf crops.	Less than 48 hours	Minimal
BENZODIOXINS ● TCDD 	This class of compounds, generally referred to as ‘‘dioxins,’’ is a low-level contaminant of many compounds.	Thermally stable and poorly biodegradable	Considered to be the most toxic of the dioxin family. Long-term health effects for humans are in dispute.
POLYCHLORINATED BIPHENYLS ● Arcohlor 	Used as a coolant and insulation fluid in transformers and ballasts. Banned in 1976. Continues to contaminate sediments and waterways.	Measured in decades	While considered a threat to humans because of its ability to bio-concentrate in fish and birds (levels of 7.5 ppm have been found in ducks), toxicity relative to humans is unclear.

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*Reference 8.

†Reference 9.

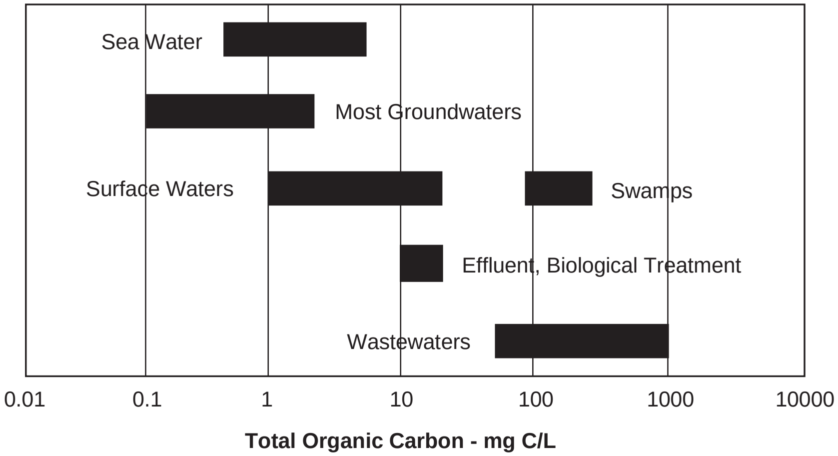


Fig. 3–1. Ranges of TOC for various natural waters

out with numerous types of water samples indicate that the most sophisticated methods usually allow the identification of only between 5 and 15 percent of the dissolved organic compounds, all of which are generally identified at the nanogram and microgram/L concentration levels. The bulk of the nonchromatographic DOC remains largely uncharacterized.

Because NOM includes literally thousands of distinct chemical species, evaluation of its properties based on a compilation of individual compounds is not realistic. Rather, researchers have attempted to characterize NOM by grouping the NOM molecules into a limited set of categories (fractions). As a practical matter, the assignment of molecules to particular categories is always operational, although the categorization is often described in terms of fundamental or conceptual characteristics. An implicit expectation of NOM fractionation is that fractions isolated from independent water sources by the same set of procedures will have similar composition and properties, though the concentration of a given NOM fraction in different sources may differ.

Numerous compounds can be present in the refractory, nonchromatographable fraction. This poorly defined fraction is often referred to as ‘‘humic or fulvic acids,’’ the distinction being an arbitrary molecular-weight cutoff of 50,000 (humic substance $>$50,000 daltons). Several models have been proposed for the structure of these humic and fulvic acids. They generally describe the presence of highly condensed polyhydroxyaromatic cores.[10]

Like snowflakes, NOM molecules are all unique while also sharing many common properties. NOM characterization has been and remains a priority for the water treatment industry, in part because such characterization holds the key to understanding, predicting, and perhaps controlling NOM reactivity under water treatment conditions.

One common approach for characterizing NOM divides the mixture into hydrophilic and hydrophobic fractions. The hydrophilic fraction includes carboxylic acids, carbohydrates, amino acids and amino sugars, and proteins, while the hydrophobic fraction includes so-called humic species. All these groups of compounds are likely to be present in all natural waters, though their absolute and relative concentrations are expected to vary from site to site. Despite the site specificity of NOM and some variability of its properties over time (often related to seasonal cycles of biological activity), humic species typically dominate the NOM on a mass basis, contributing from ~50 percent to >90 percent of the DOC in most natural waters.[11]

Most dissolved humic substances are thought to have molecular weights of a few hundred to a few thousand atomic mass units.[12] Humic molecules contain aromatic, carbonyl, carboxyl, methoxyl, and aliphatic units, with the phenolic and carboxylic functional groups providing most of the protonation and metal complexation sites. As opposed to synthetic polymers and many biological polymers (e.g., proteins), humic molecules are not composed of unique, highly reproducible monomeric building blocks.[13] Rather, a group of similar building blocks is probably present in many humic molecules, but the sequence and frequency of occurrence of the building blocks, and the exact structure of the regions between adjacent building blocks, is probably different in every humic molecule.

Previous investigations of NOM from a wide variety of sources have led to some generalizations about the characteristics of NOM molecules in different environments. For instance, environments in which water is exposed to mineral surfaces that complex and adsorb NOM contain low concentrations of dissolved NOM, especially humic substances. NOM in lakes and reservoirs of moderate to high trophic status is often dominated by material generated in the water body (autochthonous material), whereas low-order rivers and streams usually carry more NOM that is generated exterior to the water body (allochthonous NOM). Allochthonous NOM has large C/N ratios (near 100:1), is highly colored, and has significant aromatic carbon content, whereas autochthonous NOM has lower C/N ratios (near 10:1), is almost colorless, and has low aromatic carbon content.[14]

NOM Impact on Water Treatment

For the drinking water industry, the major goal of NOM characterization is to understand and predict the reactivity of NOM or its fractions in specific treatment processes. Any water treatment plant is likely to have specific compliance issues and/or operational problems that require attention and for which certain types of NOM characterization are useful. For example, if biological activity in the distribution system is a major issue, then attention should focus on chemical classes contributing to the biodegradable compounds in solution (e.g., proteins, carbohydrates, amino acids) and less attention can be paid to humic species (unless they have been altered by ozonation). Alternatively, if the concentration of disinfection by-products is the major concern, the humic part of NOM should receive special attention, and losses of carbohydrates and proteins during the NOM characterization will not affect the results dramatically.

Table 3–6 provides a general evaluation of water quality issues associated with NOM, and the fractions of the NOM that are most likely to be relevant for each issue. Table 3–7 provides general information relating different NOM fractions to the formation of some important disinfection by-products.

DISINFECTION BY-PRODUCTS

What Are They and Why Are They Important?

In the past three decades, the use of chlorine as both a drinking water disinfectant and as an industrial chemical has moved from savior to suspect. Its incorporation in chem-

TABLE 3–6. Major Chemical Classes of NOM Compounds and Associated Water Quality Problems

	Associated Compliance Problems					
Chemical Class of Compounds	Disinfection By-products, Chlorination	Disinfection By-products, Ozonation	Biological Activity	Color	Transport of Heavy Metals	Taste and Odor
Humic species	Major role	Major role	Little impact*	Major role	Major role	Secondary importance
Carbohydrates	Not known, probably insignificant	Probably not significant	Major role	None	Insignificant	Insignificant
Amino acids	Important	May be significant*	Major role	Major	Secondary importance	Insignificant
Proteins	Important	Important*	Major role	Major	May be significant	Insignificant
Carboxylic acids	Important	Generated by ozonation	Secondary importance	None	Insignificant	Insignificant
Other						Primarily geosmin and 2-methyl-isoborneol (2-MIB)

Source: See Reference 15.

*Ozonated NOM and ozonation by-products form a major group of BDOC.

TABLE 3–7. Association Between Specific Types of Disinfection By-products and Major Chemical Classes of NOM

Class of Disinfection By-products	Class of NOM Compounds				
	Humic Species	Carbohydrates	Amino Acids	Proteins	Carboxylic Acids
Trihalomethanes (THM)	Primary source	Not known, probably minor	Minor*	Important*	Secondary source
Haloacetic acids (HAA)	Primary source	Not known, probably minor	Not known, probably insignificant	Not known, may be significant	Secondary source
Chlorophenols	Primary source	Insignificant	Insignificant	Insignificant	Insignificant
Haloketones	Primary source	Not known, may be significant	Not known, probably insignificant	Not known, may be significant	Insignificant
Chloral hydrate	Primary source	Not known, probably insignificant	Not known, probably insignificant	Not known, probably insignificant	Insignificant
Haloacetonitriles	Important	Not known, probably insignificant	Important	Important	Insignificant
Trihalonitromethane	Important	Not known, probably insignificant	Important	Important	Insignificant
Cyanogen halide	Important	Not known, probably insignificant	Important	Important	Insignificant
Aldehydes	Primary source	Not known, may be significant	Not known, probably insignificant	Important	Not known, probably minor
Contribution to the pool of DBP precursors	Predominant (80–90%)	Little known, probably $<5\%$	Important (5–10%)	Important (5–10%)	Probably 5–10%

Source: See Reference 16.

*Some amino acids produce THMs, but their concentration is much smaller than that of humic species.

icals, packaging materials and structural products gives them desirable properties, including thermal stability and resistance to degradation. Many plastics, plasticizers, solvents, pesticides, and dielectric fluids are not possible without chlorine. As a drinking water disinfectant, chlorine has literally saved tens of millions of lives and is arguably the single most important public health measure ever instituted. Although its history is frequently overlooked, it is axiomatic that chlorine is the most important of the multiple pathogen barriers used in water treatment.

One of the hallmarks of chlorinated organic compounds is their persistence—resistance to biologic or environmental degradation. Concerns about chlorinated compounds began to surface, first in the 1960s with respect to chlorinated pesticides, then in the 1970s with respect to the negative persistence of other chlorinated substances, such as polychlorinated biphenyls (PCBs) and photochemically active solvents like trichloroethylene (TCE). TCE was first associated with atmospheric smog reactions but later became one of the most pervasive contaminants of the nation's groundwater. Like most other chlorinated compounds, the properties that initially made it a desirable product were the same properties that made it an environmental liability, and much of the concern was focused on chlorine.

In 1974, a Dutch chemist, Johannes Rook, published results that implicated the use of chlorine as the cause of chloro and bromo trihalomethanes (THMs) found in treated drinking waters. These early disinfection by-products were quickly joined by other halogenated organic, nonhalogenated organic, and inorganic halogen oxide compounds. None of these substances represented a new threat; their detection simply came about as the result of improved analytical methods and greater scrutiny directed at the contents of chlorinated drinking waters. Their presence in drinking water underscores the importance of understanding the disinfection process, and the need to strike a balance between potentially harmful disinfection products and the necessary protection from biological disease agents.

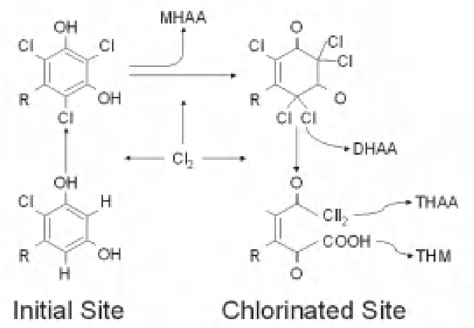
The family of trihalomethanes (THMs) and haloacetic acids (HAAs) are the most common forms of DBP. They are formed by the reaction of free chlorine with certain naturally occurring organic compounds. The generalized formula for the reaction is:

$$\begin{aligned}&\text{Free chlorine and/or bromine} + \text{Organic precursors} \rightarrow\\&\quad \text{Trihalomethanes} + \text{Haloacetic Acids} + \text{By-product compounds}\end{aligned}$$

The naturally occurring organic precursors (discussed in the section on NOM) generally are humic substances, such as humic and fulvic acids. Industrial wastes are generally not a factor in production of DBPs. The actual chemical mechanisms by which the various DBPs are formed is not thoroughly understood, but a decade of research by many groups has produced several conceptual models. Figure 3–2 presents the model of Korshin and Benjamin, displaying the suspected interactions of HAAs and THMs in the formation process.[17] Note that this model predicts that HAAs are formed first and that THMs are a by-product of the HAA formation.

Disinfectants/Disinfection By-products Rule (D/DBP)

On November 29, 1979, regulations were adopted that established a maximum contaminant level (MCL) of 0.1 mg/L for the total trihalomethane (TTHM) concentration, defined as the sum of the concentration of trichloromethane or chloroform ($CHCl_3$), tribromomethane or bromoform ($CHBr_3$), bromodichloromethane ($CHBrCl_2$), and di-



- MHAA, DHAA and THAA represents mono-, di- and trihalomethanes
- THM represents trihalomethanes

Fig. 3–2. Preferred model of THM and HAA formation following chlorination of NOM (*Source:* Reference 17)

bromochloromethane ($CHBrCl_2$). This was the first piece of regulation directed specifically at DBPs, and it covered only the trihalomethanes and affected only the larger utilities. Almost two decades of debate on the existence and importance of other disinfection by-products would follow.

In 1998, Stage I of the Disinfectant and Disinfectant By-Product Rule (D/DBP rule) was promulgated. The D/DBP rule is the first rule to be promulgated under the 1996 amendments to the Safe Drinking Water Act. Unlike the previous MCL for TTHM, all water systems regardless of source or size will have to comply with D/DBP limits. Moreover, it is likely that Stage I will be followed by a Stage II rule sometime in 2005. This future rule will address health-related information gathered in the interim and is envisioned as a means of structuring the rule-making process to accommodate our expanding knowledge base on drinking water organics. Tables 1–8 and 1–9 of this book provide an overview of Stage I relative to MCLs, monitoring and compliance issues for both DBPS, and the use of specific disinfectants. Stage I will go into effect for large systems (serving over 10,000 people) in December of 2001, with small systems following in December of 2003.

Nonregulated DBPs

Because the health implications of DBPs are better understood, the current DBP regulatory emphasis is on halogenated organic groups like THMs and HAAs. It is important to note, however, that all oxidants and disinfectants can produce DBPs, and that as our analytic capability improves new DBPs will continue to be identified. The occurrence of different DBPs should be viewed in light of the Enhanced Surface Water Treatment Rule, which essentially limits primary and secondary disinfection practices to these strategies:

- chlorine-chlorine
- chlorine-chloramine

- ozone-chloramine
- chlorine dioxide-chlorine
- chlorine dioxide-chloramine

All these disinfection strategies can produce a wide variety of DBPs, including the ozone-chloramine strategy. In fact, any strong oxidant, halogen or not, will react with NOM to produce oxidation by-products. Ozone by itself has been observed to increase the concentration of nonhalogenated organics, such as aldehydes and carboxylic acids. When used in conjunction with chlorine, ozone will increase the formation of DBPs, such as chloropicrin and the haloketones.[18] One of the principal treatment concerns relative to ozonation is that a portion of the refractory NOM compounds are oxidized to smaller, more biologically degradable compounds, hence increasing the assimilable organic carbon content. In some circumstances, this may require additional treatment to achieve a biologically stable water. Even the use of preoxidants for taste and odor control (i.e., potassium permanganate and hydrogen peroxide) can impact the DBP distribution formed by subsequent disinfection strategies.

Table 3–8 presents a shortlist of disinfectant and DBP-related compounds that are currently receiving focused attention from the USEPA relative to health effects, analytical methods, occurrence, and treatment methods. It is anticipated that Stage II of the D/DBP Rule (scheduled for promulgation in 2005) will address many of these compounds.

DBP Control Strategies

Developing a DBP control strategy requires careful planning. The current USEPA approach strongly favors those strategies involving removal of DBP precursors prior to disinfectant addition. This may involve optimizing existing processes or adding new processes to remove NOM.

In response to the original THM MCL (1978), many utilities instituted DBP control measures based on alternative disinfectants, such as ozone and chloramines. Although this approach is useful for control of THMs and, to a lesser extent, HAAs, it may not be adequate for control of other DBPs, such as chloropicrin and the haloketones. Consequently, utilities that have already installed these approaches for THM control will need to review the DBP issue once again. A limited number of utilities have installed processes, such as aeration, that remove THM after their formation. These utilities will also need to reexamine their existing approach for control of the other DPBs. The recently promulgated D/DBP rule will be a major driving force behind the design and operation of drinking water treatment facilities into the foreseeable future.

Three general strategies (or a combination thereof) are available for reducing DBPs in finished drinking water supplies:

- Remove the DBPs after they are formed.
- Use a disinfectant-oxidant other than chlorine that does not produce undesirable DBPs.
- Remove the natural organics (precursors) before disinfection-oxidation.

TABLE 3–8. USEPA's Shortlist of DBPs, Under Consideration for Future or Additional Regulations

Organic Halogenation By-products	Disinfectant Residuals and By-products
Total trihalomethanes	Free chlorine
Chloroform	Hypochlorous acid
Bromodichloromethane	Hypochlorite ion
Dibromochloromethane	Combined chlorine (chloramines)
Total haloacetic acids	Monochloramine
Monochloroacetic acid	Dichloramine
Dichloroacetic acid	Trichloramine
Trichloroacetic acid	Chlorine dioxide
Monobromoacetic acid	Chlorate
Dibromoacetic acid	Chlorite
Total haloacetonitriles	Ozone
Trichloroacetonitrile	
Dichloroacetonitrile	Inorganic By-products
Bromochloroacetonitrile	Bromate
Dibromoacetonitrile	Iodate
Total haloketone	Hydrogen peroxide
1,1-Dichloropropanone	Chlorate
1,1,1-Trichloropropanone	
Total chlorophenols	Organic Oxidation By-products
2-Chlorophenol	Total aldehydes
2,4-Dichlorophenol	Formaldehyde
2,4,6-Trichlorophenol	Acetaldehyde
Chloropicrin	Hexanol
Chloral hydrate	Heptanal
Cyanogen chloride	Total carboxylic acids
N-organochloramines	Hexanoic acid
	Heptanoic acid
	Assimilable organic carbon

Of these, the first two may be faulted for not treating the problem but dealing only with symptoms. The third strategy gets to the root of the problem itself—natural organics or precursors found in all surface water supplies and, to a lesser degree, in many groundwater supplies.

DBP Removal After DBPs are formed, several treatment technologies are available for removing them: oxidation, adsorption, and aeration. However, each of these technologies has disadvantages. Oxidation, even with ozone, is relatively ineffective for removing THMs and can also form other non-THM DBPs. Adsorption, using GAC columns, is effective for removing many DPBs, but requires long empty bed contact times (EBCTs), which makes for large columns with substantial GAC content. Moreover, the GAC sorption capacity for come DBPs is quite limited, requiring frequent and costly regeneration. (The use of activated carbons is discussed in Chapter 17.) Precursor materials are not easily stripped and will still be present after aeration to form DPBs after a disinfectant-oxidant is added to the water. In addition, aeration cannot remove any nonvolatile DBPs (such as HAAs) formed during treatment. In general, DBPs are difficult, and therefore costly, to remove from drinking water once they have been formed. Hence, the current emphasis on preventing their formation.

Alternative Disinfectants-Oxidants Switching to alternative disinfectants-oxidants may be feasible provided the following criteria are met:

- DPBs are not produced at undesirable levels.
- Microbial inactivation is at least as effective as disinfection with chlorine.
- A stable disinfecting residual is provided in the distribution system.

From an economic standpoint, the ideal alternative disinfectants-oxidants should be no more expensive than chlorine. Unfortunately, on a cost basis, free chlorine is by far the most effective disinfectant. Moreover, no single alternative disinfectant-oxidant (e.g., ozone, chlorine dioxide, chloramines, and UV radiation) can satisfy all of the above requirements. Hence, to replace free chlorine, a combination of disinfectant-oxidants is usually needed.

For example, a utility might decide to use ozone, which is much more effective than chlorine for raw water microbial inactivation and produces no significant halogenated organics. However, it is known that ozone creates a variety of other DBPs, for which little is known relative to health effects, and because ozone leaves no long-lasting residual, chloramines are commonly used as a secondary disinfectant.

Although such a strategy is adequate for reducing THM and total halogenated organic by-product levels, the combined use of these disinfectants will produce other DBPs. Studies have shown that the use of ozone with chloramines can increase levels of chloropicrin, cyanogen chloride, and total aldehydes. Consequently, utilities using this strategy to reduce THMs may have to reevaluate treatment as new DBP regulations are developed.

Removal of NOM Removal of natural organics, or precursor materials, prior to disinfection represents an optimal approach for controlling DBPs. Because precursor materials are constituents of the dissolved organic carbon (DOC) in raw water, optimizing treatment to remove DOC before adding disinfectant-oxidant provides the best strategy for reducing DBPs. Treatment technologies to remove NOM include conventional treatment, oxidation, adsorption, and membrane processes.

Application of these technologies for organics removal is discussed in this book’s respective chapters on coagulation/flocculation, activated carbon, and membranes.

MTBE

Methyl tertiary butyl ether (MTBE) poses a historically unprecedented challenge to the management of the nation’s water supplies. It is a chemical produced in massive quantities and distributed throughout almost all areas of the United States. It has unique properties that make it a water quality threat, as well as exceptional resistance to conventional treatment, including aeration and activated carbon adsorption. Of little concern prior to the late 1990s, MTBE literally burst onto the regulatory scene with reports of serious water supply contamination in almost all states. Ironically, the widespread use of MTBE, and consequent problems, is the result of environmental initiatives designed to address air quality issues.

MTBE is the most common oxygenated fuel additive used in reformulated gasoline. In 1996 alone, over 18 billion pounds were produced, making it the second most widely produced organic chemical in the United States. Initially, MTBE was formulated into gasoline as an anti-knock additive. However, its widespread use dates to 1995, when the USEPA mandated the seasonal use of reformulated gasoline in polluted metropolitan areas to reduce vehicle emissions. The state of California mandated the year-round use of reformulated gasoline in 1996. Other states have since followed suit.

Reformulated gasoline must contain at least 2 percent oxygen by weight, which is the equivalent of 11 percent MTBE by volume (in some metropolitan areas the MTBE content reaches as high as 18 percent by volume). Although other oxygenates are available, MTBE has been favored because of cost, ease of production, and fuel blending characteristics.

MTBE is a significant water quality threat because it is substantially more soluble than most organic solvents. It has been reported in ground water supplies at concentrations as high as 650 μg/L, and can reach several mg/L in a water column at equilibrium.[19] It has relatively low volatility (Henry's constant of 0.001 atm), and hence it does not quickly evaporate into the air when spilled. And, because it does not have a strong sorptive affinity for the organics in the soil matrix, it can readily percolate through the soil to the groundwater. It is most commonly associated with release from leaking underground storage tanks, product pipelines, and above-ground storage tanks. Surface water contamination has been attributed to exhaust emissions from recreational watercraft and to poor fuel-handling practices. At present, the full extent of MTBE contamination is not known, because only limited monitoring data are available. Most striking are the accumulating reports on MTBE groundwater contamination that suggest it can be readily identified at concentrations in the 2–10 μg/L range in most urban settings.

Health Effects

No maximum contaminant level has yet been established for MTBE in drinking water. However, the USEPA has issued a draft lifetime health advisory of 20–200 μg/L—a range that reflects the prevailing uncertainty about MTBE's carcinogenicity.[20] (A health advisory describes the concentration of a contaminant at which adverse health effects would not be expected to occur for a specific duration of a specific exposure—it includes a margin of safety to protect sensitive populations.) Sixteen states have established a standard health advisory guideline, or action limit, between 35 and 230 μg/L for MTBE in drinking water. The state of California has established an interim action level of 35 μg/L, above which will trigger remediation efforts or other appropriate action.

Taste and Odor Concerns

In addition to health effects, MTBE also has aesthetic implications, most notably, taste and odor. Thresholds vary widely, but recent studies suggest a taste and odor threshold in the 25–60 μg/L range.[21] In 1999, the state of California enacted a secondary drinking water standard (aesthetic based) of 5 μg/L for MTBE.

Treatment Technologies

A substantial portion of the threat presented by MTBE relates to the difficulty in removing it from contaminated water supplies. In contrast to most chlorinated VOCs, air-stripping and granular activated carbon (GAC) are not likely to be cost-effective for MTBE. Air stripping can remove MTBE, but generally requires air-to-water ratios in excess of 150:1, yet can achieve removal efficiencies of only 97 percent. (Most VOCs can be 99 percent removed at air-to-water ratios of less than 50:1.) Likewise, the capacity of GAC to adsorb MTBE is low, about 0.0016 g/g carbon.[22] This is an order of magnitude lower than the adsorption characteristic for most organic solvents, and hence would require excessive amounts of GAC and exceptionally large GAC contactors.

The most promising technology for MTBE removal appears to be advanced oxidation. It has been shown that MTBE can be oxidized through direct contact with ozone, or indirect reaction with radical oxidant species (hydroxyl radical). The reaction with molecular ozone appears to be quite slow, whereas oxidation by the radical oxidants is extremely rapid.[23] Several oxidation technologies are in development specifically for MTBE treatment. The most promising of these appears to be an UV–hydrogen peroxide process and an ozone–hydrogen peroxide process (Peroxone). Tests at the Metropolitan Water District have shown that advanced oxidation removed an average of 80 percent of the contaminant at influent MTBE levels in the range of 25–100 μg/L.[24]

REFERENCES

1. APHA, AWWA, WEF, *Standard Methods for the Examination of Water and Wastewater,* 19th ed., Washington, DC, 1995.
2. Love, O. T., et. al., ‘‘Treatment of Volatile Organic Carbon in Drinking Water,’’ EPA-600/8-83-019, 1983.
3. Symons, J. et. al., ‘‘National Organics Reconnaissance Survey for Halogenated Organics,’’ *JAWWA* 67:11, p. 634, 1975.
4. Kim, N., and Stone, D.W., ‘‘Organic Chemicals in Drinking Water,’’ New York State Health Department, Albany, NY, 1980.
5. USEPA, Office of Drinking Water, National Organics Monitoring Survey (NOMS), Cincinnati, OH, 1978.
6. Hubbs, S. A., *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* AWWA Research Foundation, 1983.
7. Kavanaugh, M., and Trussell, R. R., ‘‘Design of Aeration Towers to Strip Volatile Contaminants,’’ *JAWWA* 72:12, 1980.
8. Roberts, P. V., ‘‘Nature of Organics Contaminants in Groundwater and Approaches to Treatment,’’ Proceedings AWWA National Conference, St. Louis, MO, 1981.
9. Fawell, J. K., and Hunt, S., *Environmental Toxicology: Organic Pollutants,* John Wiley & Sons, New York, 1988.
10. Ram, N. M., et. al., *Organic Carcinogens in Drinking Water: Detection, Treatment, and Rich Assessment,* John Wiley & Sons, New York, 1986.
11. Mallevialle, J., ‘‘Why Is Natural Organic Matter Problematic? Workshop Proceedings— Natural Organic Matter in Drinking Water, September 1993,’’ AWWA Research Foundation, Denver, CO 1994.
12. Thurman, E. M., *Organic Geochemistry of Natural Waters,* Niphoff and Junk Publishers, Dordrecht, 1985.
13. McIntyre, C., et. al., ‘‘Electrospray Mass Spectrometry of Groundwater Organic Acids,’’ *J. Mass Spectrometry* 32:328, 1997.

14. Christman, R. F., et. al., *Oxidation Degradation of Humic Substances from Freshwater Environments, Humic Substances,* John Wiley & Sons, New York, 1989.
15. Aiken, G. R., et al., *Evidence for Diffusion of Aquatic Fulvic Acids for Sediments, Organic Substances and Sediments in Water, Humics and Soils,* Lewis Publishers, Boca Raton, FL, 1991.
16. Korshin, G., et. al., *Use of UV Spectroscopy to Study Chlorination of Natural Organic Matter, in Water Disinfection and Natural Organic Matter* (Minear and Amy, eds.), American Chemical Society, 1996.
17. Larsen, R. A., and Weber, E. J., *Reaction Mechanism in Environmental Organic Chemistry,* Lewis Publishers, Boca Raton, FL, 1994.
18. Korshin, G. V., et. al., ''Monitoring the Properties of Natural Organic Matter Through UV Spectroscopy: Evaluation of a Consistent Theory,'' *Water Research* 31:7, 1997.
19. Weinberg, H. S., and Glaze, W. H., ''An Overview of Ozonation Disinfection By-Products,'' in *Disinfection By-Products in Drinking Water* (Minear and Amy, eds.), Lewis Publishers, Boca Raton, FL, 1996.
20. Rodriguez, R., ''MTBE in Groundwater and the Impact on the City of Santa Monica's Drinking Water Supplies,'' presented at MTBE Workshop, Association of California Water Agencies, Ontario, California, 1997.
21. Dale, M. S., et al., ''MTBE: Occurrence and Fate in Source Water Supplies,'' Proceedings of American Chemical Society Conference, Div. of the Environment, San Francisco, 1997.
22. Lowry, J. D., et. al., ''Adsorptive Removal of Gasoline from Household Supplies,'' *Oil in Fresh Water,* Perganson Press, New York, 1987.
23. Buxton, G. V., et al., ''Critical Review of Rate Constants for Hydroxl Radicals,'' *Journal of Physics and Chemistry,* Ref. Data, 17:2, p. 13, 1988.
24. Liang S., et. al., ''Oxidation of MTBE by Ozone and Peroxide Process,'' *JAWWA* 91:6, p. 104, 1999.

CHAPTER 4

Microbiology of Drinking Water

INTRODUCTION

All natural waters support biological communities, which in turn contain microorganisms. In almost all instances, a portion of the microorganism population will have derived from mammalian species, and a small subgroup of these will represent potential health concern to humans.

The microbiology of drinking water is complex, and our understanding of it is far from complete. Recent advances in our ability to isolate and identify microbes point to an even broader range of human health threats than had been previously realized. This chapter provides an overview of the state of knowledge on pathogens in raw and treated water. It discusses the history of waterborne disease outbreaks in the United States, reviews issues of groundwater contamination, and examines the potential for further microbial regulations as relating to the question of emerging pathogens.

PATHOGENIC ORGANISMS

Although the epidemiological relation between water and disease had been suggested as early as 1854, it was not until the establishment of the germ theory of disease by Louis Pasteur in the mid-1880s that water as a carrier of disease-producing organisms could be understood. In 1859, London experienced the ‘‘Broad Street Well’’ cholera epidemic, and Dr. John Snow conducted his famous epidemiological study, as discussed in Chapter 1, ‘‘Criteria and Standards for Improved Potable Water Quality.’’ Now, more than 100 years later, the list of potential waterborne microbial diseases is considerably larger, and includes bacterial, viral, and protozoan parasitic microorganisms.

Bacteria

Bacteria are single-celled organisms ranging in size from 0.1 to 10 μm. The physical structure of the bacterial cell can be characterized by shape, components, size, and the manner in which they grow. Most bacteria can be grouped by shape into four general categories: spheroid, rod, curved rod or spiral, and filamentous. *Cocci,* or spherically shaped bacteria, are approximately 1 to 3 μm in diameter. *Bacilli,* or rod-shaped bacteria, are variable in size and range from 0.3 to 1.5 μm in width (or diameter) and from 1.0 to 10.0 μm in length. *Vibrios,* or curved rod–shaped bacteria, typically vary in size from 0.6 to 1.0 μm in width (or diameter) and from 2 to 6 μm in length. *Spirilla*, or spiral-shaped bacteria, can be found in lengths up to 50 μm, whereas filamentous bacteria can occur in length in excess of 100 μm. Table 4–1 summarizes what is known relative to specific pathogenic bacteria, the disease caused by the organisms and the host or transmitting agent for the organisms.

Viruses

Viruses are microorganisms composed of a strand of genetic material—deoxyribonucleic acid (DNA) or ribonucleic acid (RNA)—located within a protective protein coat. Viruses are obligate parasites, unable to carry out any form of metabolism and are completely dependent upon host cells for replication. Viruses are typically 0.01 to 0.1 μm in size and are very species specific with respect to infection, choosing to attack only one type of host. Although the principal modes of transmission for several types of viruses, such as hepatitis B virus and poliovirus, are through food, personal contact, or exchange of body fluids, viruses can also be transmitted through potable water. Some viruses, such as the retroviruses (including the HIV group), appear to be too fragile for water transmission to be a significant danger to public health.[1] Viruses are not classified by species, but rather are grouped by structural characteristic. Table 4–2 lists the viral growth of interest in drinking water, the associated diseases, and disease pathology.

Protozoa

Protozoa are single-cell eucaryotic microorganisms without cell walls that utilize bacteria and other organisms for food. They are typically much larger than bacteria, ranging in size from 2 to 15 μm. Most protozoa are free-living in nature and can be encountered in water. However, several species are parasitic and live on or in host organisms. Host organisms can vary from primitive organisms, such as algae, to highly complex organisms, such as human beings. Several species of protozoa known to utilize human beings as hosts are shown in Table 4–3.

The protozoans of most concern in drinking water are *Giardia lamblia* and *Cryptosporidium parvum*. They are typically found in water as resistant spores, cysts, and oocysts. These forms protect them from environmental stresses and make them far more resistant to chlorination than are viruses and most bacteria. However, effective filtration and pretreatment can reduce their density by at least 99 percent. Spores/cysts/oocysts of these organisms are typically found in surface water or groundwaters directly influenced by surface waters. The serious consequences of distributing water contaminated with these organisms is discussed in the next section.

WATERBORNE DISEASE OUTBREAKS

The number of disease outbreaks identified as waterborne in the United States dropped steadily following the introduction of potable water chlorination. Figure 4–1 shows the dramatic decline in waterborne disease outbreaks from 1920 through 1960 as the practice of chlorination became widespread. Of course, improvements in other treatment procedures (flocculation, filtration, etc.) have contributed to the decrease in waterborne disease. However, disinfection is, and will continue to be, one of the most important treatment process for the prevention of waterborne diseases.

TABLE 4-1. Waterborne Bacterial Diseases

Causative Agent	Disease	Symptoms	Transmitting Agent
Salmonela typhosa	Typhoid Fever	Incubation period 7–14 days. Headache, nausea, loss of appetite, constipation or diarrhea, insomnia, sore throat, bronchitis, abdominal pain, nose bleeding, shivering, and increasing fever. Rose spots on trunk.	Feces and urine of typhoid carrier or patient
S. paratyphii *S. schottinulleri* *S hirschfeldii C.*	Paratyphoid fever	General infection characterized by continued fever, diarrhea disturbances, and sometimes rose spots on trunk. Incubation period 1–10 days.	Feces and urine of carrier or patient
Shigella flexneri *Sh. dysenteriue* *Sh. sonnei* *Sh. paradisinteriae*	Bacillary dysentery	Acute onset with diarrhea, fever, tenesmus, and stool frequently containing mucus and blood. Incubation period 1–7 days.	Bowel discharges of carriers and infected persons
Vibrio comma *V. chlorea*	Cholera	Diarrhea, vomiting, rice water stools, thirst, pain, and coma. Incubation period a few hours to 5 days.	Bowel discharges, vomitus, carriers
Pasteruella tularensis	Tularemia	Sudden onset with pains and fever; prostration. Incubation period 1–10 days.	Rodent, rabbit, horsefly, wood tick, dog, fox, hog
Brucella melitensis	Brucellosis (undulant fever)	Irregular fever, sweating, chills, and pain in muscles.	Tissues, blood, mild, urine, infected animal
Pseudomonas pseudomallei	Melioidosis	Acute diarrhea, vomiting, high fever, delirium, and mania.	Rats, guinea pigs, cats, rabbits, dogs, horses
Laptospira icterohaemorragiae (spirochete)	Leptospirosis (Well’s disease)	Fevers, rigors, headaches, nausea, muscular pains, vomiting, thirst, prostration, and jaundice may occur.	Urine and feces of rats, swine, dogs, cats, mice, foxes, sheep
Enteropathogenic *E. coli*	Gastroenteritis	Water diarrhea, nausea, prostration, and dehydration.	Feces of carrier
Legionella pnermophila	Legionnaires’ disease	Pneumonia, fever, cough, lethargy, and severe gastrointestinal symptoms. Pontiac fever is the second clinical syndrome associated with *Legionella* infections.	River and lakes, cooling towers, evaporative condensers, and potable water distribution systems

Sources: References 2 and 3. From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

TABLE 4-2. Waterborne Human Enteric Viruses and Known Associated Diseases

Group	Subgroup	No. of Types or Subtypes	Disease Entities Associated with These Viruses	Pathological Changes in Patients	Organs Where Virus Multiplies
Enterovirus	Poliovirus	3	Muscular paralysis	Destruction of motor neurons	Intestinal mucosa, spinal cord, brain stem
			Aseptic meningitis	Inflammation of meninges from virus	Meninges
			Febrile episode	Viremia and viral multiplication	Intestinal mucosa and lymph
	Echo virus	34	Aseptic meningitis	Same as above	Same as above
			Muscular paralysis	Same as above	Same as above
			Guillain-Barre's Syndrome[1]	Destruction of motor neurons	Spinal cord
			Exanthem	Dilation and rupture of blood vessels	Skin
			Respiratory diseases	Viral invasion of parenchymiatous of respiratory tracts and secondary inflamatory responses	
			Diarrhea	Not well known	
			Epidemic myalgia	Viral invasion of cells with secondary inflammatory responses	
			Pericarditis and myocarditis	Same as above	Pericardial and myocardial tissue
			Hepatitis		Liver parenchyma
	Coxsackie virus A	>24	Herpangina[2]	Virus invasion of mucosa with secondary inflammation	Mouth
			Acute lymphatic pharyngitis	Same as above	Lymph nodes and pharynx
			Aseptic meningitis	Same as above	Same as above
			Muscular paralysis	Same as above	Same as above
			Hand-foot-mouth disease[3]	Viral invasion of skin cells of hands-feet-mouth	Skin of hands-feet, and much of mouth
			Respiratory disease	Same as above	Same as above
			Infantile diarrhea	Viral invasion of cells of mucosa	Intestinal mucosa
			Hepatitis	Viral incasion of liver cells	Parenchyma cells of liver
			Pericarditis and myocarditis	Same as above	Same as above

	B	6	Pleurodynia[4]	Viral invasion of muscle cells	Intercostal muscles
			Aseptic meningitis	Same as above	Same as above
			Muscular paralysis	Same as above	Same as above
			Meningoencephaitis	Viral invasion of cells	Meninges and brains
			Pericarditis, endocarditis, myocarditis	Same as above	Same as above
			Respiratory disease	Same as above	Same as above
			Hepatitis or rash	Same as above	Same as above
			Spontaneous abortion	Viral invasion of vascular cells	Placenta
			Insulin-dependent diabetes	Viral invasion of insulin-producing cells	Langerhan's cells of pancreases
			Congenital heart anomalies	Viral invasion of muscle cells	Developing heart
Reo virus		6	Not well known	Not well known	
Adenovirus		31	Respiratory diseases	Same as above	Same as above
			Acute conjunctivitis	Viral invasion of cells and secondary inflammatory responses	Conjunctival cells and blood vessels
			Acute appendicitis	Viral invasion of mucosa cells	Appendia and lymph nodes
			Intussusception	Viral invasion of lymph nodes	Intestinal lymph nodes
			Subacute thyroiditis	Viral invasion of parenchyma cells	Thyroid
			Sarcoma in hamsters	Sarcoma in hamsters	Muscle cells
Hepatitis		>2	Infectious hepatitis	Invasion of parenchyma cells	Liver
			Serum hepatitis	Invasion of parenchyma cells	Liver
			Down's Syndrome	Invasion of cells	Frontal lobe of brain, muscle, bones

Source: Adapted from *Journal AWWA*, Vol. 66, No. 5 (May 1974) by permission. Copyright © 1974, American Water Works Association. Reference 4.

[1] Ascending type of muscular paralysis.

[2] Febrie episode with sores in mouth.

[3] Rash and blister on hand-foot-mouth with fever.

[4] Pleuritis type of pain with fever.

TABLE 4–3. Human Parasitic Protozoans

Protozoan	Host(s)	Disease	Transmission	Occurrence
Acenathamoeba castellani	Fresh water, sewage, humans	Amoebic meningoencephalitis	Gains entry through abrasions, ulcers, and as secondary invader (hiding other infections)	
Balantidium coli	Pigs, humans	Balantidiasis (dysentery)	Contaminated water	Micronesia has the only known site of an outbreak
Cryptosporidium	Animals, humans	Cryptosporidiosis	Person-to-person or animal-to-person contact, ingestion of fecally contaminated water or food, or contact with fecally contaminated environmental surfaces	Canada, England, and the United States
Entamoeba	Humans	Amoebic dysentery	Contaminated water	Last in United States outbreak, 1953
Giardia lamblia	Animals, humans	Giardiasis (gastroenteritis)	Contaminated water	Mexico, United States, USSR
Nagleria fowleri	Soil water, decaying vegetation	Primary amoebic meningoencephalitis	Nasal inhalation with subsequent penetration of nasopharynx, exposure from swimming in freshwater lakes	

Source: Reference 5. From Montgomery, J., *Water Treatment Principles and Design.* Copyright © 1985 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

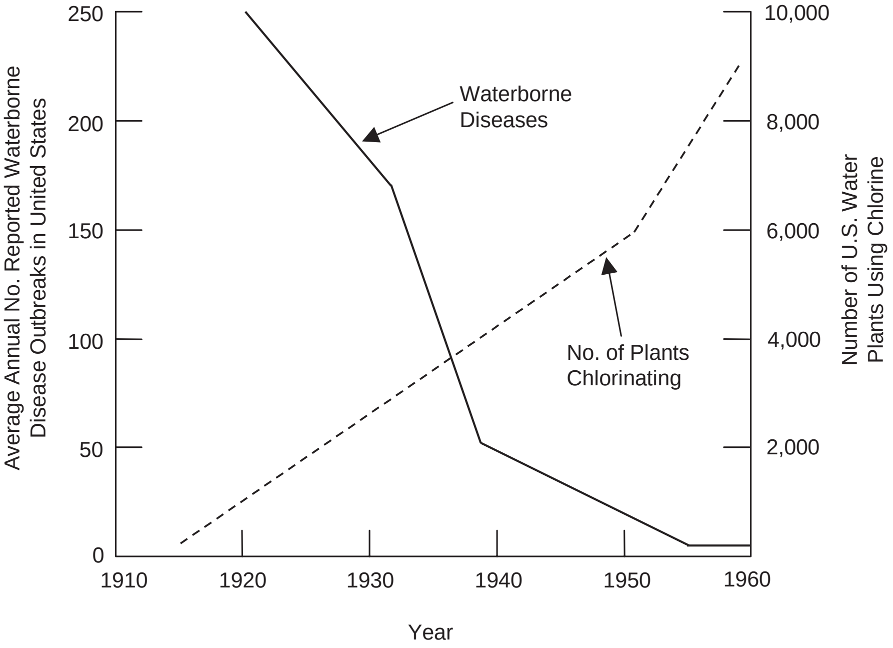


Fig. 4–1. Decline in waterborne diseases (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Within the past 40 years, several pathogenic agents never before associated with the documented waterborne outbreaks have appeared in the United States. *Escherichia coli* (*E. coli*) and *Giardia lamblia* were first identified to be the etiological agents responsible for waterborne outbreaks in the 1960s. The first recorded human *Cryptosporidium* infection occurred in the mid-1970s. Also during that time was the first recorded outbreak of pneumonia caused by *Legionella pneumophilia.*[6,7] Recently, there have been numerous waterborne outbreaks that have been caused by *E. coli, Giardia lamblia, Cryptosporidium,* and *Legionella pneumophila.*

Despite many improvements in water treatment, waterborne disease continues to occur at high levels. Figure 4–2 shows the etiology of waterborne diseases from 1971 to 1994. Approximately 50 percent of the reported outbreaks occurred in non–community systems and were caused by unknown etiology (acute gastrointestinal illness).[8] Protozoan parasites were the most frequently identified etiologic agents and accounted for approximately 20 percent of the total number of outbreaks between 1971 and 1994. *Giardia* was the most frequently identified of the protozoa that cause disease outbreaks. However, the number of outbreaks attributed to *Cryptosporidium* has risen dramatically within the last 10 years.

E. coli Outbreaks

Between 1961 and 1970, an agent never before associated with documented waterborne outbreaks in the United States appeared: enteropathogenic *E. coli.* In the past, various

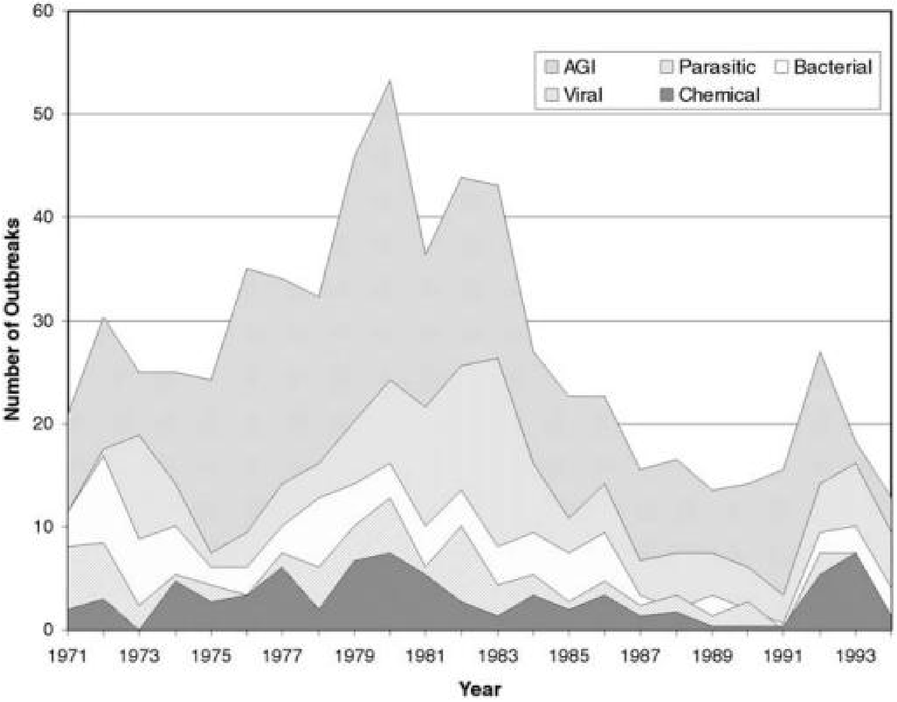


Fig. 4-2. Outbreaks of waterborne disease (Reprinted from *Journal AWWA*, Vol. 88, No. 3 (March 1996), by permission. Copyright © 1996, American Water Works Association)

serotypes of *E. coli* have been implicated as the etiological agent responsible for disease in newborn infants, usually the result of cross-contamination in nurseries. More recently, there have been several well-documented outbreaks of *E. coli* (serotypes 0 1111:B4 and 0 124:B27) associated with adult waterborne disease outbreaks.[9,10] In 1975, the etiologic agent of a large outbreak at Crater Lake National Park was *E. coli* serotype 06:1116.[11]

Giardia lamblia Outbreaks

Giardia lamblia is a flagellated protozoan that is responsible for Giardiasis, a disease that ranges from being mildly to extremely debilitating. Of all known etiological agents that were responsible for waterborne disease outbreaks, *Giardia lamblia* is one of the most commonly identified pathogens. The life cycle of *Giardia* includes a cyst stage when the organism remains dormant and is extremely resilient. The cyst can survive extreme environmental conditions until it is ingested by a warm-blooded animal, and its life cycle continues with excystation. The cysts are relatively large (8–14 μm) and can be removed effectively by filtration using diatomaceous earth, granular media, or membranes.

Giardiasis can be acquired by ingesting viable cysts from food or water or by direct contact with fecal material. In addition to humans, wild and domestic animals have been implicated as hosts. Between 1972 and 1984, 50 waterborne outbreaks of Giardiasis occurred, with about 20,000 reported cases.[7] Currently, there is no simple and reliable method for assaying *Giardia* cysts in water samples. Microscopic methods for detection and enumeration are tedious and require examiner skill and patience. *Giardia* cysts are relatively resistant to chlorine, especially at higher pHs and low temperatures.

***Cryptosporidium* Outbreaks**

Cryptosporidium is a protozoan similar to *Giardia*—it forms resilient oocysts as part of its life cycle. The oocysts are smaller than *Giardia* cysts, typically about 4–6 μm in diameter. These oocysts can survive under adverse conditions until ingested by a warm-blooded animal and then continue with excystation.

Due to the recent increase in the number of outbreaks of *Cryptosporidiosis*, substantial research has focused on *Cryptosporidium*. Medical interest has increased because of its occurrence as a life-threatening infection to individuals with compromised immune systems, such as the very young, the very old, cancer patients, and those inflicted with acquired immune deficiency syndrome (AIDS). The primary symptom of *Cryptosporidiosis* is acute diarrhea. Other symptoms include vomiting, abdominal pain, and low-grade fever. Pets, farm animals, contaminated drinking water, and person-to-person contact have been identified as sources of infection.

Cryptosporidium oocysts have been detected in 65 to 97 percent of the surface water supplies that have been tested recently within the United States.[11] Between 1984 and 1992, all reported *Cryptosporidium* outbreaks in the United States occurred in communities whose water utilities met the state and federal standard for acceptable drinking water quality that were in effect when the outbreak occurred. As discussed in Chapter 1, in 1993 the largest documented waterborne disease outbreak in U.S. history occurred in Milwaukee, and was caused by *Cryptosporidium*. The outbreak was associated with a deterioration in raw-water quality and a simultaneous decrease in effectiveness of the coagulation–filtration process. In turn, these factors led to an increase in the turbidity of treated water and inadequate removal of *Cryptosporidium* oocysts.

Legionnaires' Disease Outbreaks

An outbreak of pneumonia occurred in 1976 at the annual convention of the Pennsylvania American Legion. A total of 221 people were affected by the outbreak; 35 of those afflicted died. The cause of the pneumonia was not determined immediately, despite an intense investigation by the Centers for Disease Control. Six months after the incident, microbiologists were able to isolate a bacterium from an autopsy lung tissue of one of the Legionnaires. The bacterium (*Legionella pneumophila*) responsible for the outbreak was isolated and found to be distinct from other known bacteria and was named for the Legionnaires.[12] Following the discovery of this organism, other *Legionella*-like organisms were discovered. Altogether, 26 species of *Legionella* have been identified, and seven are etiologic agents for Legionnaires' disease.[1]

Legionnaires' disease does not appear to be capable of being transferred from person to person. Epidemiological studies have shown that the disease enters the body through the respiratory system. *Legionella* can be inhaled in water particles less than

5 μm in size from such facilities as cooling towers, hospital hot water systems, and recreational whirlpools.[12]

Pneumonia is the major manifestation of Legionnaires' disease. Typically, patients feel lethargic, have fever and cough, and become disoriented. Unlike those with other forms of pneumonia, patients with Legionnaires' disease often have severe gastrointestinal symptoms. The disease rarely afflicts healthy individuals; rather, it tends to afflict people with preexisting illnesses, especially those with chronic lung disease.

Regulatory Requirements

The ability to detect bacteria, coupled with the introduction of chlorine as a disinfectant, led to the first quantitative microbial quality standard for potable water. In 1914, the U.S. Public Health Service adopted the first maximum permissible limits for bacterial plate counts and *Bacillus coli*. Since 1914, numerous standards that addressed pathogenic contaminants have been promulgated.

MICROBIAL INDICATORS OF WATER QUALITY

A variety of methods are available to characterize the microbial content of potable water. However, due to monetary and technological constraints, indicator organisms have been used historically to ascertain the microbial content of drinking water. Because most pathogens that are transmitted via potable water originate in human and/or animal feces, one of the characteristics of an appropriate indicator organism is its presence in the fecal contamination of warm-blooded animals. Other important attributes of an ideal indicator organism are:

- Suitable to all waters
- Present in sewage and polluted waters
- Populations should correlate with the degree of microbial contamination
- Occur in greater populations than pathogens
- Absent from unpolluted waters
- Easily and quickly detected by simple laboratory tests
- Consistent biochemical and identifying characteristics
- Harmless to humans and animals

Currently, no organism or groups of organisms meet all of these criteria. However, coliform bacteria fulfill most of these criteria and have historically been the most commonly used indicator organism. This group of organisms include all aerobic and facultative anaerobic, gram-negative, non-spore-forming, rod-shaped bacteria. One of the primary characteristics of coliform bacteria is their ability to ferment lactose with gas formation within 48 hours at 35°C. Within this group, *E. coli* is the most widely known member because it is commonly used for bacteriological experiments.

Although the use of coliform bacteria as indicator organisms has many advantages, there are several disadvantages of its use:

- Under certain conditions, regrowth of coliform bacteria can occur in water that results in a false positive.
- *Aeromonas* can biochemically mimic coliform bacteria.
- When coliforms are in the presence of high populations of other plate-count bacteria, false results can be obtained.
- Several pathogens have been found to survive longer than coliform in natural waters and/or through various treatment processes. For example, coliforms were detected in only 55 percent of the protozoan outbreaks in 1993 and 1994, as opposed to 92 percent for bacterial or unknown etiology. This suggests that the use of coliforms as indicators of protozoan contamination is not reliable.

CURRENT REGULATIONS

With the passage of the Safe Drinking Water Act (SDWA) in 1974, the U.S. Environmental Protection Agency was granted the authority to set standards for the drinking water delivered by public water supplies. The SDWA contained a series of steps and timetables for developing regulations. Since 1974, the SDWA has been amended to include more restrictive criteria for pathogenic organisms, such as the Surface Water Treatment Rule (SWTR), the Total Coliform Rule (TCR), and the Interim Enhanced Surface Water Treatment Rule (IESWTR).

Surface Water Treatment Rule

The Surface Water Treatment Rule of amendments of 1996 reestablished goals of microbial integrity shown in Table 4–4 for *Giardia* cysts and virus removal in surface water supplies. To meet these goals, the SWTR established treatment requirements for filtration and disinfection to meet certain removal levels of *Giardia* cysts and viruses.

TABLE 4-4. Log Removal/Inactivation Through Filtration and Disinfection Required Under the SWTR*

Process	*Giardia* Cysts	Virus
Total log removal/inactivation required	***3.0***	***4.0***
Conventional sedimentation/filtration credit	2.5	2.0
Disinfection inactivation required	0.5	2.0
Direct filtration credit	2.0	1.0
Disinfection inactivation required	1.0	3.0
Slow sand filtration credit	2.0	2.0
Disinfection inactivation required	1.0	2.0
Diatomaceous earth credit	2.0	1.0
Disinfection inactivation required	1.0	3.0
No filtration	0.0	0.0
Disinfection inactivation required	3.0	4.0

Source: Reference 13.

*One log removal/inactivation = 90%; two days = 99%.

These goals can be met using various treatment processes, primarily filtration and disinfection.

Total Coliform Rule

To maintain biological stability in the distribution system, the Total Coliform Rule requires that treated water contain a residual disinfectant of 0.2 mg/L when entering the distribution system. A measurable disinfectant residual must be maintained in the distribution system, or the utility must show through monitoring that the heterotrophic plate count remains less than 500 per 100 mL sample.

FUTURE REGULATIONS

Regulations pertaining to drinking water standards are bound to change in the future. This change is driven by several factors:

- The objective is to produce a drinking water free from microbial contaminants, thus requiring effective methods for the removal of microbes.
- Disinfectants produce undesirable by-products because they are strong oxidants and react readily with organic and inorganic compounds in the water to produce these by-products. The formation of by-products needs to be reduced to a minimum.
- Organisms such as *Giardia, Cryptosporidium,* and other emerging pathogens may be more resistant to traditional methods of disinfection and water treatment.
- Safety concerns associated with handling and dosing hazardous chemicals are requiring additional facilities and procedures to meet public safety concerns, and are affecting the overall cost of water treatment facilities.
- Recent advances in disinfection practices and new disinfectants are producing highly effective inactivation to specific organisms at modest costs.
- A need exists to maintain a disinfectant residual in the distribution system.

As of this writing, there is an intense regulatory effort to limit exposure to, and enhance the removal of *Cryptosporidium* in water supplies. The Interim Enhanced Surface Water Treatment Rule (IESWTR) of 1996 established an MCL (goal) of zero *Cryptosporidium* oocysts in treated water. However, this and all future regulations will be limited in effectiveness until a viable method of analyzing for *Cryptosporidium* is developed. At present, the most promising approach is USEPA Method 1622. However, even this improved technique is exceedingly time and labor intensive. It requires filtering of large sample volumes, elution and pelletizing steps, separation of oocysts using density gradients, staining of the oocysts with fluorescent immuno-antibodies, and then physical enumeration under the microscope. After all this, the actual oocyst recovery is only about 35 percent.[14]

New analytical techniques for both *Cryptosporidium* and *Giardia* are under development. However, until a reliable and easily implementable technique is established, it will be impossible to assess or even set an acceptable level of risk for these pathogens in a drinking water system.

Cryptosporidium and *Giardia* in Groundwater

Because groundwater suppliers have a natural barrier to surface contaminants, they have long been regarded as relatively sure sources of drinking water. Only recently has this been shown to be suspect. In a study conducted by Hancock,[15] groundwater suppliers in the vicinity of livestock waste agricultural application sites were evaluated.

Data generated from the detection of *Cryptosporidium* oocysts and *Giardia* cysts in 463 groundwater samples were compiled. Samples were taken from 199 sites in 23 of the 48 contiguous states. The groundwater sources were vertical wells, springs, infiltration galleries, and horizontal wells. Samples were analyzed using an immunofluorescence technique. Inclusion of data from all sources showed that 12 percent of the sites were positive for *Cryptosporidium* and *Giardia* or both. *Cryptosporidium* oocysts were found in 5 percent of the vertical wells, 20 percent of the springs, 50 percent of the infiltration galleries, and 45 percent of the horizontal wells. *Giardia* cysts were found in 1 percent of the vertical wells, 14 percent of the springs, 25 percent of the infiltration galleries, and 36 percent of the horizontal wells. These data suggest that springs are at a higher risk for contamination than vertical wells. Overall, the results strongly suggest that groundwater systems in agricultural areas should be carefully monitored until contamination levels and acceptable risk factors are determined.

Emerging Pathogens

Although much attention has been directed at chemical compounds in drinking water over the past two decades (i.e., pesticides, heavy metals, trihalomethanes), recent waterborne disease outbreaks have returned the health focus to the microbial quality of the distributed water. Concerns mount about both food- and waterborne outbreaks of previously unknown "bugs." Even some familiar bacteria have become sinister, as microbial regrowth in biofilms appear to add to the invulnerability of certain bacterial groups. Although no action has been taken as of this printing, the USEPA is considering expanding the list of microbial contaminants. The AWWA Microbiological Contaminants Research Committee (1999) has received the available evidence on potential pathogens and has identified four species of bacteria as of particular concern.

Table 4–5 presents these bacteria and their health effects, occurrence, and persistence in order of their concern to the water industry. The *M. avium* complex was ranked the highest priority because of its common occurrence in water; its health effects, particularly in immunocompromised individuals; its ability to regrow in distribution system biofilms; and its resistance to disinfection. *H. pylori* was ranked of moderate concern, primarily because of its common occurrence in human populations and its health effects. Ranked of low concern for the water industry were pathogenic *E. coli* and *Campylobacter*, primarily because they are controlled by conventional treatment and disinfection.

Methods to detect all of these bacteria need to be improved, and a better understanding of the occurrence and control of these agents in water may change the concern about their health risks. The AWWA Microbiological Contaminants Research Committee also reviewed the pathogenicity of unregulated viruses and protozoa (1999). Although several viruses were identified as potential emerging pathogens, they were deemed to be of far lesser concern than the bacteria identified above.

TABLE 4–5. Emerging Bacterial Pathogens of Concern in Drinking Water

Bacterium	Priority for Water Industry	Drinking Water Treatment	Health Effects	Occurrence in Water	Environmental Persistence	Documented Waterborne Outbreaks	Size, μm
Mycobacterium avium complex	High	Chlorine-resistant	Pneumonia, gastrointestinal illness	Common	Regrows	Yes	0.3–0.6 × 1–10
Helicobacter pylori	Moderate	Sensitive to disinfection	Ulcers, possible stomach cancer	Unknown	Unknown	No	0.5–1 × 2.5–4
Pathogenic *Escherichia coli*	Low	Sensitive to disinfection	Diarrhea, kidney failure	Common	Low; one to two weeks	Yes	1.0–1.5 × 2–6
Campylobacter jejuni	Low	Sensitive to disinfection	Diarrhea, relapsing fever	Common	Low	Yes, but commonly foodborne	0.3 × 1–5

REFERENCES

1. AWWA, *Water Quality and Treatment.* F. W. Pontius (ed.), McGraw-Hill, New York, 1990.
2. Salvato, J. A., Jr., *Environmental Engineering and Sanitation,* 2nd ed., John Wiley & Sons, New York, 1972.
3. Geldreich, E. E., in *Water Pollution Microbiology* (R. Mitchell, ed.), John Wiley & Sons, New York, 1972.
4. Taylor, F. B., ‘‘Viruses—What Is Their Significance in Water Supplies?’’ *JAWWA,* Vol. 66, p. 306, 1974.
5. Montgomery J. M., *Water Treatment Principles and Design,* John Wiley & Sons, New York, 1985.
6. Centers for Disease Control, ‘‘Assessing the Public Threat Associated with Waterborne Cryptosporodiosis Report of a Workshop,’’ *JAWWA,* Vol. 80, No. 2, p. 88, 1989.
7. Craun, G. F., and Jakubowski, W., ‘‘Status of Waterborne Giardisis Outbreaks and Monitoring Methods,’’ *JAWWA,* Water Related Health Issue Symp., Atlanta, GA, November 1986.
8. Craun, G. F., ‘‘Outbreaks of Waterborne Disease in the U.S.: 1971–1978,’’ *JAWWA,* Vol. 73, No. 7, p. 360, 1981.
9. Craun, G. F., ‘‘Statistics of Waterborne Outbreaks in the US (1920–80),’’ in *Waterborne Diseases in the United States* (G. F. Craun, editor), CRC Press Inc., Boca Raton, Fla., 1986.
10. Craun, G. F., ‘‘Surface Water Supplies and Health,’’ *JAWWA,* Vol. 80, p. 40, 1998.
11. Kramer, M. H., et al., ‘‘Water Disease 1993 and 1994,’’ *JAWWA,* Vol. 88, p. 66, 1996.
12. Witherell, L. E., Duncan, R. W., Stone, K. M., Stratton, L. J. Orciari, L., Kapel, S., and Jillson, D. A. ‘‘Investigation of Legionella Pneumophila in Drinking Water,’’ *JAWWA,* Vol. 80, No. 2, p. 87, 1988.
13. USEPA, *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Works Systems Using Surface Water Sources*. Malcolm Pirnie, Inc., and HDR Engineering, Inc., for USEPA Science and Technology Branch. Contract No. 68-01-6989. 1990.
14. Clancy, J. L., et al., ‘‘USEPA Method 1622,’’ *JAWWA,* Vol. 91, No. 9, p. 60, 1999.
15. Hancock, C. M., et al., ‘‘Correlation of *Cryptosporidium* and *Giardia* Occurrence in Groundwaters with Surface Water Indicators,’’ *JAWWA,* Annual Conference Proceedings, Dallas, TX, June 1998.

SUPPLEMENTARY READING

Riggs, J. L. ‘‘AIDS Transmission in Drinking Water: No Threat,’’ *JAWWA,* Vol. 81, No. 9, p. 69, 1989.

USEPA, *Community Water System Survey—Volume 11. Detailed Survey Results Tables and Methodology Report.* U.S. EPA publication 815-R-97-001b, January 1997.

CHAPTER 5

Water Chemistry

INTRODUCTION

This chapter provides basic information to assist the engineer in understanding the role of water chemistry in determining the cause-and-effect relationships of water treatment processes, and includes information on:

- Elements commonly of concern in water systems
- pH, acidity, and alkalinity
- Solubility
- Oxidation
- Corrosion
- Water stability

ELEMENTS AND COMPOUNDS

Elements are substances that cannot be subdivided into simpler substances, and constitute the fundamental constituents of matter. Compounds represent substances formed by one or more elements, the compounds having properties unlike the elements from which they are formed.

Elements differ from one another in weight, size, charge, and chemical properties. Table 5–1 lists elements common to water engineering, along with other symbols, atomic weights, and common valences.

Atomic weight is the weight of an element relative to that of carbon, which has an atomic weight of exactly 12. The atomic weight expressed in grams is called the *gram atomic weight* of the element, or commonly called a *mole.* A mole represents a quantity equal to Avogadro's Number (6.0238×10^{23}) and can be thought of as a collective number, such as a "dozen." For example, 1 mole of chlorine contains 6.0238×10^{23} atoms and weighs 35.5 grams.

Elements can combine to form molecules. The molecular weight of these compounds equals the sum of the atomic weights of the individual elements. It is termed the *gram molecular weight of a molecule, and represents a mole of molecules*. For example, the molecular weight of sodium chloride (NaCl) is 58.5 grams (23.0 + 35.5), while 1 mole of ammonia (NH_3) is 17.0 grams ($14.0 + 3 \times 1$). One mole of sodium chloride therefore contains 6.0238×10^{23} molecules and has a mass of 58.5 grams.

TABLE 5–1. Elements Commonly Used in Water Engineering

Name	Symbol	Atomic Weight	Common Valence*	Combining Weight†
Aluminum	Al	27.0	3+	9.0
Arsenic	As	74.9	3+	25.0
Barium	Ba	137.3	2+	68.7
Boron	B	10.8	3+	3.6
Bromine	Br	79.9	1−	79.9
Cadmium	Cd	112.4	2+	56.2
Calcium	Ca	40.1	2+	20.0
Carbon	C	12.0	4−	
Chlorine	Cl	35.5	1−	35.5
Chromium	Cr	52.0	3+ 6+	17.3
Copper	Cu	63.5	2+	31.8
Fluorine	F	19.0	1−	19.0
Hydrogen	H	1.0	1+	1.0
Iodine	I	126.9	1−	126.9
Iron	Fe	55.8	2+ 3+	27.9
Lead	Pb	207.2	2+	103.6
Magnesium	Mg	24.3	2+	12.2
Manganese	Mn	54.9	2+ 4+ 7+	27.5
Mercury	Hg	200.6	2+	100.3
Nickel	Ni	58.7	2+	29.4
Nitrogen	N	14.0	3− 5+	
Oxygen	O	16.0	2−	8.0
Phosphorus	P	31.0	5+	6.0
Potassium	K	39.1	1+	39.1
Selenium	Se	79.0	6+	13.1
Silicon	Si	28.1	4+	6.5
Silver	Ag	107.9	1+	107.9
Sodium	Na	23.0	1+	23.0
Sulfur	S	32.1	2−	16.0
Zinc	Zn	65.4	2+	32.7

*Valence is based upon the combining value of the hydrogen atom, which is assigned the unit value of 1. The valence of any other atom represents the number of atoms of hydrogen that will be replaced by it, or that are equivalent to it in combining value. Thus, an atom with a valence of 2 will replace two hydrogen atoms, or react with two hydrogen atoms. Some elements have multiple valence numbers, depending on the chemical reaction.

†In ordinary chemical work, the atomic and combining weights are expressed as whole numbers, rather than using a decimal place as shown in this table for fundamental reference. Thus, the atomic weight of cadmium used in ordinary chemical work would be 112, and the combining weight would be 56.

Valence

Valence is a measure of how many atoms of one substance will combine with the atoms of another. Hydrogen has been selected as a reference substance having a reaction value of 1. Any atom that will combine with one atom of hydrogen has the same value, 1. The atom chlorine has a combining value of 1, because it combines with one atom of hydrogen to make one molecule of hydrochloric acid ($H^+ + Cl^- \rightarrow HCl$). The nitrogen atom in ammonia (NH_3) has a valence of 3.

Combining Atomic Weights to Form Molecules

Atoms will combine to form molecules in the ratio of their valence number. The weight of the molecule equals the sum of the weights of the atoms constituting the molecule. For example, the molecular weight of sodium chloride (NaCl) equals the sum of the atomic weights of sodium and chlorine ($23 + 35.5 = 58.5$). Thus, 23 pounds of sodium will react with 35.5 pounds of chlorine to produce 58.5 pounds of salt. However, if atoms of higher valence than 1 are involved, such as calcium with a valence of 2, the combining weights are not equal to the atomic weights, but are equal to the atomic weights divided by the valence. Thus, when calcium (atomic weight 40) combines with chlorine (atomic weight 35.5) to form calcium chloride ($CaCl_2$, molecular weight $40 + 2 \times 35.5 = 111$), the ratio of the weights of the two constituents is not 40:35.5, but 40:71. This means that adding 500 pounds of $CaCl_2$ will add $500 \times 40/111 = 180$ lb of calcium and $500 \times 71/111 = 320$ lb of chloride.

Ions

Many substances—such as salt, acids, or bases—can be dissolved in water. The molecules of dissolved substances dissociate into ions in solution. Ions commonly encountered in natural waters are sodium, calcium, magnesium, chloride, sulfate, bicarbonate, carbonate, nitrate, iron, fluoride, silica, and potassium. Metals may also occur as trace ion constituents. Table 5–2 lists several ions commonly found in water treatment.

Units of Expression

The concentration of molecules or ions in solution is normally expressed as the weight of the compound or element in milligrams per liter of water (mg/L) or parts per million (ppm). When the solution is water, the mg/L and ppm concentration is the same because 1 mg/1,000,000 mL of water is the same as 1 part by weight to 1 million parts for low concentrations.

Concentrations are also expressed as pounds per million gallons. To convert milligrams per liter to pounds per million gallons:

$$1 \text{ mg/L} = 8.34 \text{ lbs/million gallons} \tag{5–1}$$

Milliequivalents per liter (meq/L) expresses the concentration in terms of its combining weight. Milliequivalents are calculated from mg/L for the element or compound times the valence and divided by the molecular atomic weight.

TABLE 5–2. Ions Commonly Found in Water

Name	Formula	Molecular Weight	Common Valence	Combining Weight
Ammonium	NH_4^+	18.0	1+	18.0
Hydroxyl	OH^-	17.0	1−	17.0
Bicarbonate	HCO_3^-	61.0	1−	61.0
Carbonate	CO_3^{-2}	60.0	2−	30.0
Orthophosphate	PO_4^{-3}	95.0	3−	31.7
Orthophosphate, mono-hydrogen	HPO_4^{-2}	96.0	2−	48.0
Orthophosphate, di-hydrogen	$H_2PO_4^-$	97.0	1−	97.0
Bisulfate	HSO_4^-	97.0	1−	97.0
Sulfate	SO_4^{-2}	96.0	2−	48.0
Bisulfite	HSO_3^-	81.0	1−	81.0
Sulfite	SO_3^{-2}	80.0	2−	40.0
Nitrite	NO_2^-	46.0	1−	46.0
Nitrate	NO_3^-	62.0	1−	62.0
Hypochlorite	OCl^-	51.5	1−	51.5

$$\text{meq/L} = \frac{\text{mg/L} \times \text{valence}}{\text{atomic weight}} \tag{5–2}$$

pH, Acidity, and Alkalinity

The pH is an expression of the acid or base condition of a solution. Pure water dissociates into hydrogen ion (H^+) and hydroxide ion (OH^-), as shown in Equation 5–3:

$$H_2O \rightleftharpoons H^+ + OH^- \tag{5–3}$$

pH is defined as the negative log of the hydrogen ion molar concentration. At neutral, the pH is negative log of 10^{-7}, or 7. The pH scale ranges from 0 (strong acid) to 14 (strong base). Most natural waters tend to have pH values of 5.5 to 8.0. Most soft waters have a pH between 6.0 and 7.0. A pH less than 7 is often due to the presence of carbon dioxide and other acid constituents. Alkaline waters have pH values between 7 and 8. For instance, waters of the Great Lakes have pH values of approximately 7.8. Figure 5–1 shows the pH of some common water sources and the compound typically used to adjust the water pH.

Alkalinity

Alkalinity is a measure of the ability to accept hydrogen ions or neutralize acid without changing the pH of the water. The ability of a solution to accept hydrogen ions is called *buffering capacity.* The three main chemical forms that contribute to alkalinity in natural water are bicarbonates, carbonates, and hydroxides. Bicarbonates represent the major form of alkalinity, because they originate naturally from the reaction between carbon dioxide and water.

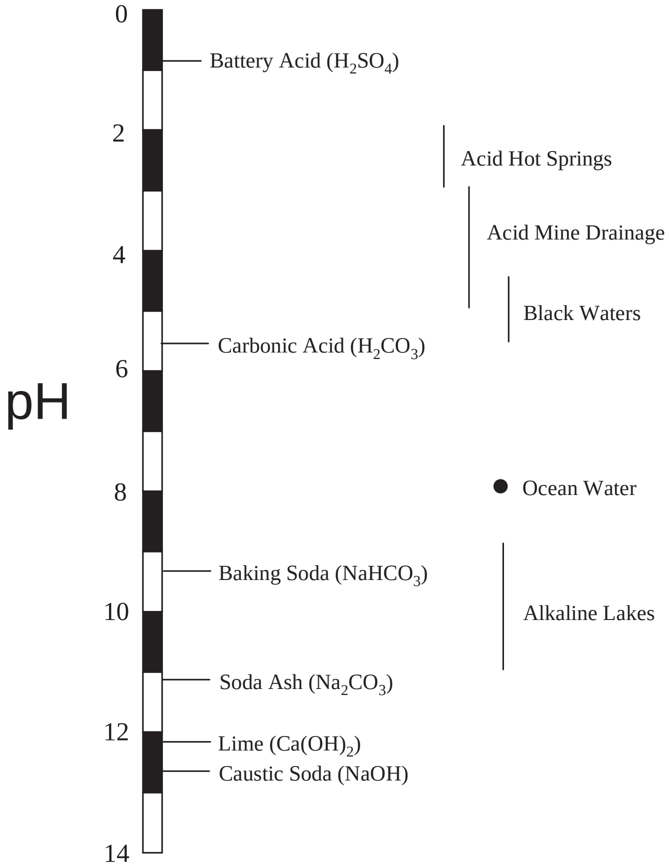


Fig. 5–1. pH ranges of common chemicals used in water treatment and natural environments

Alkalinity is determined by titration with a strong acid to specific endpoints. When acid is added to a strong basic solution, the majority of the hydrogen ions from the acid combine with the hydroxide ions of the base until the pH is about 10. This is the *carbonate equivalence point.* Then excess hydrogen ions lower the pH gradually until at pH 8.3 all carbonate ions have been converted to bicarbonates. This is the *bicarbonate equivalence point.* Additional hydrogen ions reduce the bicarbonates to carbonic acid below pH 4.5, which is the *carbonic acid equivalence point.* Phenolphthalein is used to indicate a pH of 8.3; methyl orange indicates a pH of 4.5. See Table 5–3 regarding the alkalinity species present at various pH points. A description of the alkalinity present at various pH ranges follows:

- *Caustic alkalinity.* At a pH greater than 10.0, carbonate and hydroxide ions represent the majority of the ion species present. Hydroxide ions appear at a pH above 9.0 along with a mix of bicarbonate and carbonate.
- *Phenolphthalein.* At a pH of 8.3, phenolphthalein marks the amount of acid added to consume hydroxide ions and the alkalinity associated with carbonate ions.

TABLE 5–3. Definitions and Species Contributing to Alkalinity

Endpoint pH	Name	Species Contributing to Alkalinity	Main Species Present at Endpoint
~10	Caustic alkalinity	OH^-, $CO_3^=$	OH^-, $CO_3^=$, HCO_3^-*
8.3	Phenolphthalein alkalinity	OH^-, $CO_3^=$	HCO_3^-
~4.5	Methyl orange alkalinity, or total alkalinity	OH^-, $CO_3^=$, HCO_3^-	H_2CO_3

*Endpoint species depend on water chemistry.

- *Methyl orange.* At a pH of 4.5, methyl orange determines the amount of acid added to consume the buffering capability of bicarbonate.
- Below a pH of 4.5, there is no alkalinity.

Because the primary buffers are bicarbonate and carbonate, the pH of most natural water falls between 6 and 9. When acid is added, a portion of the H^+ ion combines with the HCO_3^- to form un-ionized H_2CO_3, maintaining the same pH because only un-ionized H^+ affects pH. If a base is added, the OH^- ions react with free H^+, increasing pH; however, the added hydroxide ion can react with bicarbonate:

$$OH^- + HCO_3^- \rightarrow CO_3^= + H_2O$$

In this case, H^+ is not reduced by OH^- and the pH remain constant. The water is ‘‘buffered’’ against pH change. Similarly, when a strong acid is added, the reverse reaction could take place:

$$H^+ + CO_3^= \rightarrow HCO_3^-$$

Once again, the added acid (H^+) reacts with bicarbonate and does not increase the H^+ concentration and depress the pH. In this fashion, alkalinity represents the buffering capacity of a water.

Acidity

Acidity is a measure of a water’s tendency to accept hydroxide ions or neutralize a base. The three different types of acidity are determined at the same equivalence points as the alkalinities. However, acidity is a measure of the resistance to a pH increase (see Table 5–4).

TABLE 5–4. Definitions and Species Contributing to Acidity

Endpoint pH	Name	Species Contributing to Acidity	Main Species Present at Endpoint
~4.5	Mineral acidity	H^+	H_2CO_3
8.3	Carbon dioxide acidity	H_2CO_3	HCO_3^-
~10.0	Total acidity	H^+, H_2CO_3, HCO_3^-	$CO_3^=$, HCO_3^-, OH^-*

*Endpoint species depend on water chemistry.

- *Mineral acidity.* The initial solution pH is below the H_2CO_3 equivalence point. The equivalents of base required to raise the pH to the H_2CO_3 equivalence point (pH $\approx$ 4.5) are a measure of the mineral acidity.
- *CO_2 acidity.* The initial solution pH lies below the HCO_3^- equivalence point. The equivalents of base required to raise the pH to the HCO_3^- equivalence (pH $=$ 8.3) are a measure of the CO_2 acidity.
- *Total acidity.* The initial solution pH lies below the CO_3^{-2} equivalence point. The equivalents of base required to raise the pH to the CO_3^{-2} equivalence point (pH $\approx$ 10) are a measure of the total acidity.

SOLUBILITY

Gases, liquids, and solids dissolve in water. The amount of solute present may vary continuously below a certain limit, called the *solubility limit.* Solubility is the concentration of solute present when the solution is in a state of equilibrium with an excess of the pure solute, and is a fixed number for a given temperature, pressure, and solvent. This equilibrium and its application are important in water engineering.

Liquids and Solids

In general, the solubility of solids and liquids is highly dependent on temperature but only slightly on pressure. In most water engineering situations, solubility may be considered as a function of temperature alone. Exceptions are situations where extreme pressures exist, such as deep underground aquifers, great ocean depths, or certain industrial applications. The solubilities of most inorganic salts increase with temperature, but a number of calcium compounds, such as $CaCO_3$, $CaSO_4$, and $Ca(OH)_2$, decrease in solubility with increasing temperature. The activity of a solid in regard to solubility is constant at a given temperature, and this activity is stated mathematically by the *solubility product* (K_{sp}), which is expressed as $K_{sp} = [A^+][B^-]$, where $[A^+][B^-]$ represents the product of the molar concentrations of the ions making up the substance. The solute concentrations (i.e., $[A+]$) are usually expressed in moles per liter:

$$\text{Molarity} = \frac{\text{milligrams per liter}}{\text{gram-molecular weight} \times 10^3}$$

Tables 5–5 and 5–6 list typical K_{sp} constants of interest in water engineering. In an unsaturated solution, the ion product ($[A^+][B^-]$) is less than K_{sp}. If the ion product is greater than K_{sp}, the solution is supersaturated and will tend to form a precipitate.

Other ions present can affect the solubility of a substance through either the *common-ion effect* or the *secondary salt effect.* For example, in a solution containing 100 ppm of carbonate alkalinity, $CaCO_3$ has a solubility of only 0.5 ppm, although its solubility in pure water is about 13 ppm. This repression of solubility in the presence of an excess of one of the ions concerned in the solubility expression is known as the *common-ion effect.* The solubility of slightly soluble salts is increased when other salts that do not have an ion in common with the lightly soluble substance are present. The increased ionic strength of the solution resulting from the foreign salt causes a decrease in the activity coefficients of the slightly soluble substance. For example, calcium

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TABLE 5–6. Additional Solubility-Product Constants

Compound	K_{sp} at 25°C
Carbonates	
$CaCO_3$ (calcite)	4.6×10^{-9}
$CaCO_3$ (aragonite)	6.0×10^{-9}
$MgCO^3$	1.0×10^{-5}
Hydroxides	
$Mg(OH)_2$	1.8×10^{-11}
$Pb(OH)_2$	5.0×10^{-15}
Phosphates	
$AlPO_4$	1.0×10^{-21}
$MgNH_4PO_4$	2.5×10^{-13}
Sulfates	
$CaSO_4$	2.6×10^{-5}
$PbSO_4$	1.6×10^{-8}
Sulfides	
ZnS	3.1×10^{-22}
FeS	5.0×10^{-18}
Fe_2S_3	1.0×10^{-88}

Source: Adapted from Reference 2.

carbonate is several times more soluble in seawater than in freshwater as a result of the secondary salt effect.

Chemical Reactions Changing Equilibrium

Chemical reactions in water treatment rely on shifting equilibria to achieve desired results. The most common methods for competing reactions are by formation of insoluble substances, weakly ionized compounds, gaseous end products, and oxidation and reduction. The best example is the precipitation of calcium by adding lime. At a pH of approximately 10, hydroxyl ions convert bicarbonates to carbonates to allow the formation of calcium carbonate precipitation as follows:

$$\begin{array}{ccccccc} Ca(HCO_3)_2 & + & Ca(OH)_2 & \rightarrow & 2CaCO_3 & + & 2H_2O \\ \text{bicarbonate hardness} & + & \text{lime slurry} & \rightarrow & \text{solid precipitation} & + & \text{water} \end{array} \qquad (5\text{–}4)$$

Gas Solubility

Henry's law is used to calculate the amount of a gas that will dissolve in water. It states that the concentration of any gas dissolved in a liquid, at a constant temperature, is directly proportional to the pressure that the gas exerts above the liquid. Equation 5–5 is a practical form of Henry's law.

Note that Henry's law can be presented in different forms, depending on the units of measurement and approximating assumptions used.

$$C_{eq} = HP_{gas} \qquad (5\text{–}5)$$

where:

C_{eq} = the equilibrium gas solubility, mg/L
P_{gas} = the partial pressure of the gas above liquid, atm
H = Henry's law constant for the gas at a given temperature, mg/L-atm

Table 5–7 shows the Henry's law constant for some gases commonly used in water treatment.

The solubility of oxygen, called dissolved oxygen (DO), in water is greatly affected by temperature and to a lesser extent by barometric pressure:

TABLE 5–7. Henry's Law Constants at 20°C

Compound	H, mg/L-atm
Oxygen	41
Carbon dioxide	16,200
Hydrogen sulfide	3,700
Ozone	535

Source: Adapted from Reference 3.

$$C_p = C_s \frac{P - p}{1 - p} \tag{5–6}$$

where:

C_p = oxygen saturation (mg/L) in water at barometric pressure P
C_s = saturation DO concentration (mg/L) at atmospheric pressure
P = barometric pressure, atmospheres
p = partial pressure of water vapor in air, atmospheres

The solubility of oxygen in water as a function of temperature and pressure is shown in Appendix E.

Total Dissolved Solids

The dissolved solids in water, called total dissolved solids (TDS), are determined by the weight of the residue after evaporation. The residue is primarily inorganic and organic salts.

OXIDATION

A chemical is oxidized when it loses electrons to a second substance; the second substance is then reduced as it acquires the electron transfer. Oxidation indicates an increase in valence of the substance being oxidized because of the loss of negatively charged electrons; reduction involves a decrease in valence. For example, ferrous iron is oxidized to ferric by electron transfer. The equation for iron oxidation is:

$$\begin{array}{c} \mathrm{Fe}^{2+} \rightarrow \mathrm{Fe}^{3+} + e^{-} \\ \text{Ferrous} \rightarrow \text{Ferric} + \text{Electron} \end{array} \tag{5–7}$$

Electrons released by one compound must be absorbed by another to maintain electron neutrality. The above equation is called a half-reaction because it represents half of the reaction that takes place. The half-reaction shows that the iron has become more positive as the result of the loss of an electron.

A reduction reaction occurs when a compound accepts an electron. An example is chlorine as shown in the following half-reaction:

$$\begin{array}{c} \mathrm{Cl}_2 + 2e^{-} \rightarrow 2\mathrm{Cl}^{-} \\ \text{Chlorine} + \text{Electrons} \rightarrow \text{Chloride} \end{array} \tag{5–8}$$

By combining half-reaction 5–7 (twice) with 5–8, we have a complete reaction:

$$2\mathrm{Fe}^{2+} + \mathrm{Cl}_2 \rightarrow 2\mathrm{Fe}^{3+} + 2\mathrm{Cl}^{-}$$

An *oxidizing agent* is an element or compound that gain electrons in a reaction. Oxidizing agents used in water treatment include:

- Oxygen or air
- Ozone
- Hydrogen peroxide
- Potassium permanganate
- Chlorine (or hypochlorites)
- Chlorine dioxide

Selection of the most effective oxidants depends on economics, availability, safety, ease of handling, and the ability to oxidize the constituent of concern.

METAL CORROSION

Metal corrosion is the tendency of all metals, when exposed to the elements, to revert to the more stable forms in which they are found in the earth. It is a process by which metals erode. The products of corrosion are usually in the form of oxides, carbonates, and sulfides. The phenomena of corrosion are the same for all metals and alloys, differing by degree but not in kind. Water that exhibits corrosiveness can cause problems in distribution pipelines and home plumbing systems. These problems can include health, aesthetic, and economic impacts.

Electrochemical corrosion can be viewed in terms of oxidation and reduction reactions. Two reactions are possible for the corrosion of iron pipe, as shown in Equations 5–9 and 5–10:

$$\mathrm{Fe} + 2\mathrm{H_2O} \rightarrow \mathrm{Fe(OH)_2} + 2\mathrm{H^+} + 2e^- \tag{5–9}$$

$$\mathrm{Fe} + \mathrm{HCO_3^-} \rightarrow \mathrm{FeCO_3} + \mathrm{H^+} + 2e^- \tag{5–10}$$

The first reaction occurs when the carbonate concentration is low, whereas the latter occurs when the carbonate concentration is high.

Corrosion can be inhibited when the oxidation products form a stable solid, such as an oxide, hydroxide, or carbonate, that adheres to the pipe surface and prevents or slows direct contact between the metal and the corrosive water, thus providing a barrier to the exchange of electrons. This is known as *passivation protection.*

The compounds that form when iron corrodes depend on several factors, including pH and buffer capacity. The factors favoring corrosion include a low pH and high concentration of oxidizing substances, such as dissolved oxygen and free residual chlorine.

Excessive corrosion in water distribution systems can lead to the failure of plumbing systems, leakage, and the occlusion of pipes, representing a complex problem with economic and public health consequences. Chapter 21 provides an in-depth review of issues associated with corrosion in distribution systems, including the different forms of corrosion, corrosion monitoring, and discussions of how calcite saturation (water stability) affects corrosion rates.

REFERENCES

1. Sawyer, C. N., McCarty, P. L., and Parkin, G. L., *Chemistry for Environmental Engineers,* McGraw-Hill Book Co., 4th ed., New York, 1994.
2. Snoeyink, V. L., and Jenkins, D. *Water Chemistry,* John Wiley & Sons, New York, 1980.
3. Pontius, F. W. (ed.), *Water Quality and Treatment,* American Water Works Association, 4th ed., McGraw-Hill, New York, 1990.

CHAPTER 6

Specific Contaminant Removal Methodologies

INTRODUCTION

This chapter summarizes information on the sources, health effects, and applicable treatment technologies for contaminants of health concern. The information is presented for four categories of contaminants:

- Inorganic
- Organic
- Radionuclides
- Microbial

Applicable treatment technologies are identified in this chapter, with detailed information on the treatment technologies presented elsewhere in this book. Information on the drinking water standards for these contaminants is presented in Chapter 5.

INORGANIC CONTAMINANTS

Inorganic constituents found in natural water are sometimes of concern, even in the absence of sources of contamination. In some cases, contact with piping can contribute inorganic contaminants, such as lead, copper, and asbestos. Although many of the inorganic constituents are important to proper nutrition, high concentrations of constituents such as arsenic, selenium, chromium, copper, molybdenum, nickel, zinc, and sodium can have adverse health effects. Some are suspected carcinogens, such as lead, arsenic, and cadmium. Some (sodium and barium) can cause high blood pressure. Table 6–1 summarizes the applicable treatment technologies to remove inorganic constituents of greatest concern.[1] The best available treatment technology shown in the table has proven to be successful in meeting the MCL requirement at the SDWA.

ORGANIC CONTAMINANTS

The greatest source of organic constituents is the breakdown of naturally occurring organic materials, such as humic materials, microorganisms and their metabolites, and

TABLE 6-1. Inorganic Contaminant Removal Technologies

Contaminant	Best Available Technology	Alternative Treatment Technologies
Antimony	Coagulation-filtration, reverse osmosis	
Arsenic	NA	Trivalent arsenic must be oxidized to pentavalent form to be effectively removed by coagulation-filtration. Coagulation-filtration (ferric sulfate pH 6–8, alum pH 6–7), lime softening (pH>10.5), activated alumina, activated carbon, ion exchange, reverse osmosis and electrodialysis.
Asbestos	Coagulation-filtration, direct filtration, diatomaceous earth filtration, corrosion control, ion exchange, reverse osmosis	
Barium	Lime softening, ion exchange, reverse osmosis	
Beryllium	Coagulation-filtration, lime softening, activated alumina, ion exchange, reverse osmosis	
Cadmium	Coagulation-filtration, lime softening, ion exchange, reverse osmosis	Electrodialysis. Ferric and alum coagulation less effective than lime softening.
Chromium	Coagulation-filtration, lime softening (Cr III), ion exchange, reverse osmosis	
Copper	Corrosion control, source water protection	Coagulation-filtration, ion exchange, lime softening, reverse osmosis
Cyanide	Chlorination, ion exchange, reverse osmosis	Electrodialysis
Fluoride	Activated alumina, reverse osmosis	Electrodialysis, ion exchange, coagulation-filtration (iron salts not effective), lime softening (most effective in high magnesium waters)
Lead	Corrosion control, public education, source water treatment, lead service, lime replacement	Coagulation-filtration, lime softening, reverse osmosis, electrodialysis
Mercury	Coagulation-filtration (influent = 10 μg/L) lime softening, granular activated carbon, reverse osmosis (influent = 10 μg/L)	Inorganic mercury-ion exchange, electrodialysis. Organic mercury-activated carbon adsorption, ion exchange.
Nickel	Lime softening, ion exchange, reverse osmosis	Electrodialysis
Nitrate	Ion exchange, reverse osmosis, electrodialysis	Biological denitrification
Nitrite	Ion exchange, reverse osmosis	Breakpoint-chlorination
Selenium	Coagulation-filtration (SeIV), lime softening, activated alumina, reverse osmosis, electrodialysis	Ion exchange
Sulfate	Reverse osmosis, ion exchange, electrodialysis	
Thallium	Ion exchange, activated alumina	

high-molecular-weight aliphatic and aromatic hydrocarbons. Humics can also serve as precursors for formation of trihalomethanes and other organohalogen oxidation products during disinfection. Organics can also be contributed by such sources as wastewater effluents, agricultural and urban runoff, and leachate from contaminated soils. Many of the organics of greatest health concern come from these sources. The concentrations of naturally occurring organics is often in the mg/L range, while the synthetic organics from sources of contamination are often in the microgram- or nanogram-per-liter range.[2]

Organic contaminants can also be formed during water treatment, including trihalomethanes, such as chloroform, haloacetonitriles, and aldehydes. As water passes through the transmission system, organic materials, such as polynuclear aromatic hydrocarbons, can be leached from pipes, coatings, linings, and joint adhesives. Table 6–2 summarizes information on the organic constituents with known health effects. The best available treatment technology often does not recognize that oxidation is a viable treatment technology for several organic compounds, such as atrizine.

Taste and odor together is not addressed as an organic contaminant, but is a critical contaminant for the acceptability of a treated water. The individual treatment chapters on coagulation, activated carbon, and oxidation discuss taste and odor control.

RADIONUCLIDES

Radioactive atoms break down and release radioactivity in the form of alpha radiation consisting of large, positively charged helium nuclei; beta radiation consisting of electrons or positrons; or gamma radiation consisting of electromagnetic, wave-type energy. Alpha particles are massive, travel at extremely high speeds, and can be very damaging when ingested. The smaller size of beta particles allows greater penetration but creates less damage. Gamma radiation has tremendous penetrating power but limited effect at low levels.

Radiation can be naturally occurring or manmade. Naturally occurring radiation comes from elements in the earth's crust or from cosmic ray bombardment. Manmade radiation comes from three major sources: nuclear fission (weapons testing), radiopharmaceuticals, and nuclear fuel processing and use.

Based on occurrence in drinking water and related health effects, the naturally occurring radionuclides (radium 226 and 228, uranium, and radon 222) are of greatest concern. Radium 228 is a beta emitter, while the others and the decay products of radium 228 are alpha emitters. Table 6–3 summarizes information on radionuclides of greatest concern.

MICROBIAL CONTAMINANTS

Pathogenic species of bacteria, virus, and protozoans are of concern. Some pathogenic microbes, including bacteria and protozoans, are found in natural waters, but the most significant sources are human and animal feces that may enter a water supply from a variety of sources, including inadequately treated sewage and surface runoff.

Bacteria of concern and the related diseases include *Salmonella* (typhoid or paratyphoid fevers), *Shigella* (dysentery), *Yersina enterocolitica* (acute gastroenteritis), *Campylobacter jejuni* (acute gastroenteritis), *Legionalla* (pneumonic illness),

TABLE 6–2. Organic Contaminant Sources and Removal Technologies

Contaminant	Sources	Best Available Treatment Technology
Acrylamide	Water treatment polymers	Polymer addition practices
Alachlor (Lasso)	Herbicide	Granular activated carbon
Aldicarb (Temik)	Pesticide, herbicide	Granular activated carbon
Aldicarb sulfone	Pesticide, herbicide	Granular activated carbon
Atrazine (Atranex, Crisazia)	Herbicide	Granular activated carbon
Benso(a)pyrene	Fossil fuel, wood burning, coal tar, forest fires	Granular activated carbon
Benzene	Leaking fuel tanks, manufacture of chemicals, pharmaceuticals, pesticides, paints, plastics	Granular activated carbon, packed tower aeration
Carbofuran (Furadan 4F)	Pesticide, herbicide	Granular activated carbon
Carbon tetrachloride	Cleaning agents, coolant manufacture	Granular activated carbon, packed tower aeration
Chlordane	Pesticide, herbicide (most uses banned in 1980)	Granular activated carbon
2,4-D (Formula 4.0)	Herbicide	Granular activated carbon
Dalapon	Herbicide	Granular activated carbon
Di(2-ethylhexyl) adidpate	Plastics	Granular activated carbon, packed tower aeration
Di(2-ethylhexyl) phthalate	Plastics	Granular activated carbon
Dibromochloropropane (DBCP)	Pesticide (banned in 1977)	Granular activated carbon, packed tower aeration
o-Dichlorobenzene	Industrial solvent, pesticide	Granular activated carbon, packed tower aeration
p-Dichlorobenzene	Insecticide, moth balls, air deodorizers	Granular activated carbon, packed tower aeration
1,1-Dichloroethylene	Manufacture of plastics, dyes, perfumes, paints, synthetic organic chemicals	Granular activated carbon, packed tower aeration
1,2-Dichloroethane	Insecticides, gasoline	Granular activated carbon, packed tower aeration
Cis-1,2-Dichloroethylene	Extraction solvent, dyes, perfumes, pharmaceuticals, lacquers	Granular activated carbon, packed tower aeration
Trans-1,2-Dichloroethylene	Extraction solvent, dyes, perfumes, pharmaceuticals, lacquers	Granular activated carbon, packed tower aeration
Dichloromethane (methylene chloride)	Solvent	Packed tower aeration
1,2-Dichloropropane	Pesticide, solvent	Granular activated carbon, packed tower aeration
Dinoseb	Herbicide	Granular activated carbon
Diquat	Herbicide, defoliant	Granular activated carbon
Endothall	Herbicide, defoliant	Granular activated carbon
Endrin	Pesticide, insecticide	Granular activated carbon
Epichlorohydrin	Water treatment polymers	Polymer addition practices
Ethylbenzene	Styrene manufacture	Granular activated carbon, packed tower aeration
Ethylene dibromide (EDB)	Gasoline additive, soil fumigant solvent, pesticide	Granular activated carbon, packed tower aeration

TABLE 6–2. (*Continued*)

Contaminant	Sources	Best Available Treatment Technology
Glyphosate (Rodeo, Roundup)	Herbicide	Oxidation
Heptachlor (H-34, Heptox)	Insecticide (most uses restricted in 1983)	Granular activated carbon
Heptachlor epoxide	Insecticide (most uses restricted in 1983)	Granular activated carbon
Hexachlorobenzene	Pesticide manufacture	Granular activated carbon
Hexachlorocyclopentadiene	Pesticide manufacture	Granular activated carbon, packed tower aeration
Lindane	Insecticide (some uses restricted in 1983)	Granular activated carbon
Methoxychlor (DMDT, Marlate)	Insecticide	Granular activated carbon
Monochlorobenzene	Solvent, pesticide	Granular activated carbon, packed tower aeration
Oxamyl (Vydate)	Pesticide	Granular activated carbon
Pentachlorophenol	Wood preservative	Granular activated carbon
Picloram	Herbicide	Granular activated carbon
Polychlorinated biphenyls (PCBs)	Transformers, capacitors (production banned in 1987)	Granular activated carbon
Simazene	Herbicide	Granular activated carbon
Styrene	Manufacture of polystyrene plastic	Granular activated carbon, packed tower aeration
2,3,7,8-TCDD (Dioxin)	Herbicide manufacture, pulp and paper mill effluents	Granular activated carbon
Tetrachloroethylene	Dry-cleaning solvent	Granular activated carbon, packed tower aeration
Toluene	Solvent, gasoline additive	Granular activated carbon, packed tower aeration
Toxaphene	Pesticide, herbicide (most uses banned in 1977)	Granular activated carbon
2,4,5-TP (Silvex)	Herbicide (banned in 1983)	Granular activated carbon
1,2,4-Trichlorobenzene	Herbicide manufacture, dye carrier	Granular activated carbon, packed tower aeration
1,1,1-Trichloroethane	Manufacture of food wrappings, synthetic fibers	Granular activated carbon, packed tower aeration
1,1,2-Trichloroethane	Manufacture of vinylidene chloride	Granular activated carbon, packed tower aeration
Trichloroethylene	Dry cleaning wastes, manufacture of pesticides, paints, waxes, varnishes, paint stripper, metal degreasers	Granular activated carbon, packed tower aeration
Trihalomethanes	Chlorination by-product	Use alternative disinfectant, remove precursors
Vinyl chloride	Manufacture of plastics, synthetic fiber	Packed tower aeration
Xylenes	Solvent; manufacture of paint, dyes, adhesives, detergents, fuel additive	Granular activated carbon, packed tower aeration

TABLE 6-3. Radionuclide Sources and Removal Technologies

Radionuclide	Sources	Best Available Treatment, Applicable Treatment
Alpha emitters	Naturally occurring and synthetic	Coagulation-filtration, reverse osmosis
Beta particle and photon emitters	Naturally occurring and synthetic	Coagulation-filtration, ion exchange, reverse osmosis
Radium 226	Naturally occurring	Lime softening, ion exchange, reverse osmosis
Radium 228	Naturally occurring	Lime softening, ion exchange, reverse osmosis
Radon	Naturally occurring	Aeration
Uranium	Naturally occurring	Coagulation-filtration, lime softening, anion exchange

enteropathic *E. coli* (gastroenteritis), *Vibrio cholerae* (acute intestinal disease), and *Mycobacteria* (pulmonary disease). In addition, there are naturally occurring opportunistic bacteria that can cause significant disease in newborns, the elderly, and others with preexisting health problems.

Viruses of greatest interest in drinking water are hepatitis A, Norwalk-type viruses, rotaviruses, adenoviruses, enteroviruses, and reoviruses. Norwalk-type viruses and rotaviruses cause acute gastroenteritis. Adenoviruses, enteroviruses, and reoviruses can infect both the enteric and upper respiratory tract.

Pathogenic protozoans of interest in drinking water are *Giardia lamblia*, *Entamoeba histolytica, Cryptosporidium,* and *Niegleria fowleri*. *Giardia* can cause diarrhea, fatigue, and cramps. *Entamoeba histolytica* can cause amoebic dysentery, with symptoms ranging from acute diarrhea and fever to mild gastroenteritis. The primary symptom of cryptosporidiosis is acute diarrhea; symptoms may also include vomiting and fever.

TABLE 6-4. Microbial Contaminants

Contaminant	Sources	Best Available Treatment, Applicable Treatment
Coliforms, *E. coli*	Human and animal fecal matter	Disinfection
Giardia lamblia	Human and animal fecal matter	Coagulation-filtration, slow sand filtration, disinfection, diatomaceous earth filtration, direct filtration
Heterotrophic bacteria	Naturally occurring	Coagulation-filtration, slow sand filtration, disinfection, diatomaceous earth filtration, direct filtration
Legionella	Water aerosols	Coagulation-filtration, slow sand filtration, disinfection, diatomaceous earth filtration, direct filtration
Viruses	Human and animal fecal matter	Coagulation-filtration, slow sand filtration, disinfection, diatomaceous earth filtration, direct filtration

Niegleria fowleri can cause amoebic encephalitis, which is normally fatal; however, the disease is usually contracted by swimming in water with high concentrations of the organism rather than by ingesting drinking water.

Because of the difficulty of directly measuring pathogenic organisms, indicator organisms are typically used to measure the effectiveness of drinking water processes and, indirectly, the safety of the treated water. The most common indicator organisms are total coliforms, *E. coli*, fecal streptococci, enterococci, and heterotrophic plate counts.

Table 6–4 presents information on microbial contaminants of greatest concern. EPA's list of best available treatment technology does not include microfiltration and ultrafiltration, which are very effective in removing microbial contaminants. See Chapter 15.

REFERENCES

1. Pontius, F. W., ‘‘An Update of the Federal Regs,’’ *JAWWA* 88, March, 1996, pp. 36–46.
2. Tate, C. H., and Arnold, K. F., ‘‘Health and Aesthetic Aspects of Water Quality,’’ *Water Quality and Treatment*, 4th ed., American Water Works Association, McGraw-Hill, p. 63 (1990).

CHAPTER 7

Water System Planning

INTRODUCTION

This chapter provides a brief summary of the kinds of topics that may be covered in typical comprehensive water system plans. It is not intended to provide all of the detail needed to prepare a water plan; rather, it is meant to serve as a framework that water utility managers and engineers may want to consider in scoping the specific contents of their water-planning document. Of course, the content of a water plan will depend upon the size and complexity of the system, critical water system issues and needs, and state and local plan requirements.

Why Plan?

The primary purpose of a water system plan is to provide utility administrators, planners, and engineers with a ''road map'' to operate and improve the water system to meet changing needs. To achieve this goal, the water system plan should be a practical tool that the utility can use to efficiently prioritize, schedule, and fund capital improvements, operation and maintenance (O&M), replacement, and technology improvement projects. A good plan is routinely updated to incorporate changes in such factors as policy changes, growth rates, types of service needs, and regulatory requirements. The planning for water utility infrastructure needs should address water system reliability and risk of service failure. Historically, reliability has been addressed in a qualitative manner. Harberg provides information for the water utility planner to consider and evaluate risk.[1]

The water system plan also serves to illustrate the basis and justification for features of projects that may otherwise be perceived as premature, overdesigned, or unnecessary. Given the high costs of developing new sources of supply or constructing water treatment and conveyance facilities, utility administrators, planners, and engineers must prove the need to commit to courses of action that provide for anticipated needs in the distant future. Water projects usually require approval by utility boards, city councils, or water commissioners who are influenced by ratepayers and special-interest groups. Therefore, a utility must clearly demonstrate to elected officials that current commitments are required—if not for short-term needs, then to ensure long-term water supply service.

Project scheduling to meet environmental permitting and regulatory requirements is another reason for developing water supply plans. For more complex projects, it can take years to complete environmental or routing studies, obtain easements and permits, and complete predesigns and final designs. Without the justification provided by long-term planning, many projects could not be implemented until the level of service and system reliability had declined to substandard levels. Planning horizons are typically 5 years or so for short-term planning and 20 years or longer for long-term planning. Twenty-year or much longer planning horizons are especially important for systems where new sources are needed.

Finally, water system plans are required by most state government regulators. Many states also require that these plans be routinely updated. Approved water plans may be required before operating permits are issued, water rights applications are processed, or state funding sources are made available to a water utility.

Typical Water Plan Contents

A typical water system plan may contain the following information:

- Water system description
- Water demand forecast
- Conservation plan
- Evaluation of sources of supply
- Water rights assessment
- Source protection plan
- Water system analysis
- Capital improvement plan
- Financial plan
- Operation and maintenance plan
- Other supporting documents

The following sections describe some of the items that may be considered during development of this information.

WATER SYSTEM DESCRIPTION

The water system description includes a summary of the system history, ownership, service area characteristics, and infrastructure. Its purpose is to provide a basic understanding of the system and to establish a foundation for the remainder of the water plan.

Background and Ownership Information

The system description should characterize the organization and legal ownership of the system and provide background information, including the type of growth that has occurred in the past. It should also describe the geographic features that impact service area and growth. These features may include natural or constructed barriers to service.

Related Systems' Plans and Policies

The water system description should also summarize relationships and agreements with, as well as connections to, adjacent utilities. Adjacent utility planning documents and service area agreements should be reviewed and referenced to determine their consistency with the overall water system plan. These related planning documents include general plans, wastewater and sewer plans, stormwater plans, basin plans, and growth policies. Any discovered inconsistencies should be resolved or at least cited in the water plan.

Service Area Description

The service area description includes service area policies and agreements, geographic characteristics, zoning, and land use. Service area policies comprise conditions of service, wholesaling and satellite service agreements, annexation and extension policies, and other related ordinances and programs. Existing and future land use classifications and zoning describe the current and anticipated densities and types of service that are expected within the service area during the planning period. In addition to a brief narrative describing land use patterns, a good set of maps serves as the best way to describe service area characteristics.

The first water plan map should show the current and future service area boundaries, other political boundaries, such geographic features as basic topography and water bodies, and major roads and highways. Depending upon the size of the system and the extent of change anticipated, one or two figures may be needed to show current and future service area features. In addition, a second set of maps may be required to illustrate current and future zoning and land uses. Sample land use and service area maps are shown in Figures 7–1 and 7–2.

Description of Existing Facilities

The description of existing facilities provides an introduction to the operation of the existing facilities. The description typically includes a tabulated listing of system components, as well as a water system map and schematic showing pressure zones and locations of these components. The initial description should include the numbers and types of current and approved service connections and a description of interties.

An excellent way to describe system components is to use tables to list the sizes and characteristics of each type of facility. Separate tables should list intakes, wells, treatment facilities, pump stations, storage reservoirs, control stations, pressure-reducing valves (PRVs), and transmission-and-distribution piping. Of course, complex treatment plants and other systems may require the use of additional text and figures to adequately describe their function. For example, to describe a treatment plant, a flow schematic and hydraulic profile are "worth a thousand words." The point is to use tables and graphics to minimize narrative text as much as possible. A listing of sample tabular headings is shown in Table 7–1.

Graphics to describe the system should include a water system map that shows the locations of all of the features listed in Table 7–1. Figure 7–3 shows an example of a water system map. The range of sizes of conveyance can be keyed to colors or line weights/types. Another helpful graphic for systems with multiple pressure or service

Fig. 7–1. Example of a land use map

zones is a hydraulic profile schematic showing pressure zones and relationships of supply and booster pump stations to storage reservoirs.

The condition of, and problems with, the existing infrastructure should be identified and documented. Corrosion, leakage, low pressure, poor quality, and line breaks are examples of problems that need to be identified and evaluated.

WATER DEMAND FORECAST

Once the foundation for the water system plan has been established, the water demand forecast section builds upon current and future land use projections to predict growth rates and patterns for residential, commercial, industrial, and agricultural water demands. The goal of this analysis is to develop flow design criteria for system improvements to meet future needs. Water system demands vary for each system depending upon climate, population characteristics, water rates and availability, percentages of commercial and industrial customers, and overall system size.

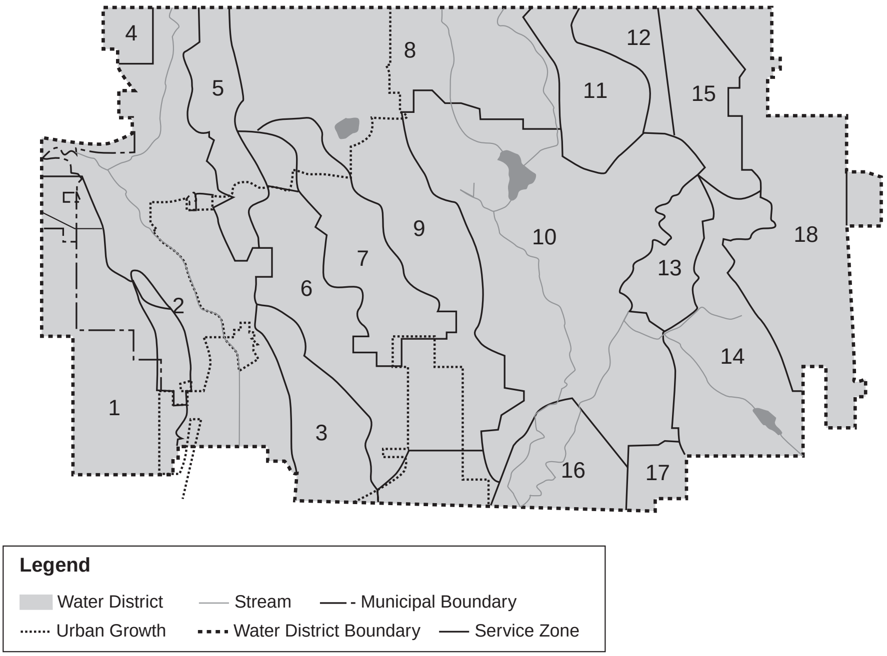


Fig. 7–2. Example of a service area map

Land Use and Population

One section of the water demand forecast should summarize current and projected population based upon land use categories and projected growth rates. Population forecasts must consider previous and anticipated growth rates, as well as changes in service area and land use during the planning period.

For growing systems, population projections may include high-, medium-, and low-growth-rate scenarios to account for impacts of unpredictable influences, such as local employment. Projections may be based upon uniform, increasing, or declining growth rates. For ‘‘built-out’’ or largely developed older systems, there may be no projected growth or growth may occur because of anticipated changes in zoning. For example, zoning of a built-out area may change from single- to multifamily residential.

The water utility should coordinate its method of population forecasting with other state and county planning agencies. Population growth projections can best be presented in tabular and graphical formats. A sample forecast for a range of population growth is shown in Figure 7–4.

Current Water Demands

Current water demands should be described in as much detail as possible based upon available metering data from sources of supply, transmission/distribution facilities, pump stations, storage reservoirs, and service connections. Source-of-supply and other

TABLE 7–1. Sample Column Headings for Describing System Components

Intakes					
Source/Capacity	Type	Year Constructed	Screening/Cleaning	Pump No./Size/ Capacity	Types of Controls
Wells					
Name/Designation	Capacity, Initial/Current	Depth/Screen Depth	Pump Type/Size	Year Installed	Treatment
Treatment					
Source	Capacity	Type of Treatment	Other Features	Year Constructed	Control Features
Pump Stations					
Name/Location	Pumps From/To	Total/Firm Capacity	Number/Types/Sizes of Pumps	Year Constructed	How Controlled
Storage Reservoirs					
Name/Zone	Size/Volume	Type/Features	Elevations, Overflow, Max., Min.	Year Constructed/ Last Coated	Water Source
PRVs					
Location and Size	Zones From/To	High Pressure	Low Pressure	Year Constructed	Controls
Conveyance					
Length	Diameter	Pipe Material	Year Constructed	Condition	Repairs

Fig. 7–3. Example of a water system map

transmission/distribution metering information is especially valuable to determine peak-hour demands if it is kept in the form of time-of-delivery (continuously recording) records.

Number and Types of Service Connections As appropriate for the system, the number of current and approved connections should be listed based upon the following service classifications:

- Single-family residential
- Multifamily residential
- Commercial/governmental

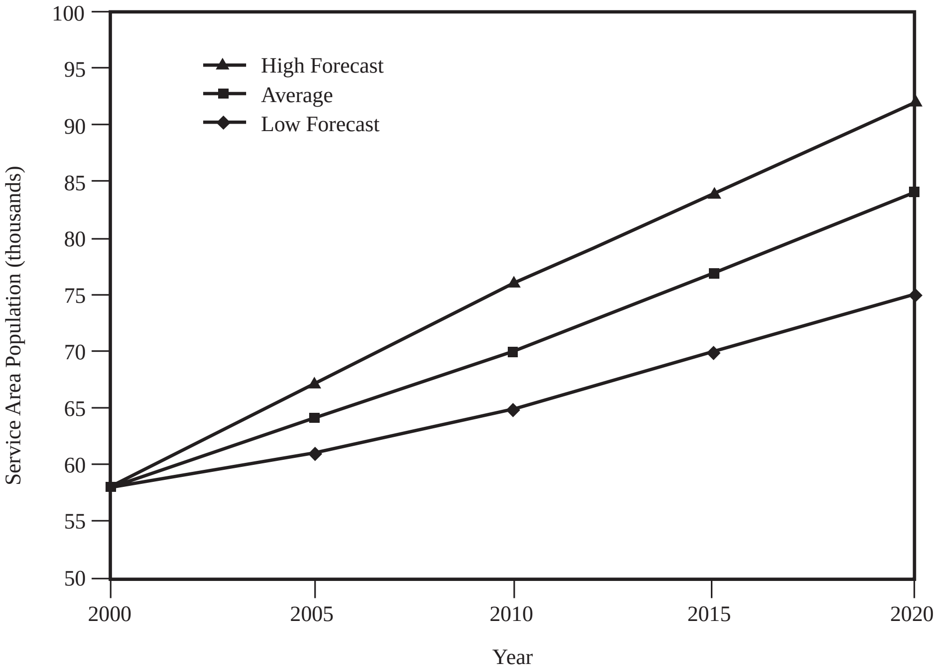


Fig. 7–4. Sample population forecast

- Industrial
- Agricultural

The meter use and water use variation for each land use classification are determined. In addition, the amount of unaccounted-for water should be determined.

Unit Demands Of course, where service connections are metered, a utility can easily add them to determine its total demand. Metered water use data are generally available only on a monthly basis, although some utilities are metering remotely and can measure individual customer demands on a real-time basis.

Actual demands in gallons (or liters) per day for each connection classification will vary depending on climatic conditions and rate structures, but typical ranges of average daily demand (ADD) are as follows:

- Single-family residential: 200–350 gpd (760–1,300 L/d)
- Multifamily residential: 150–250 gpd (570–950 L/d)
- Commercial: 200–500 gpd (760–1,900 L/d) or more

Of course, large lot sizes and residential irrigation demands can result in much higher average summer demands than those just listed. Some find it more convenient to express water use on the basis of gallons per day per acre for each land use category.

Commercial and industrial demands will vary significantly, depending upon the type of use.

Residential Equivalents Some planners describe average connection demands in terms of equivalent residential units (ERUs). An ERU is generally defined as the total single-family residential demand divided by the number of single-family residential connections. Other demand classifications can then also be defined in ERUs by dividing the total demand for that classification by the calculated ERU unit value. For example, demand for a typical multifamily unit connection may be determined to be 0.8 ERU, while that for an average commercial connection may be 2.0 ERU. Although convenient to use this shorthand technique, it does not translate well to maximum-day or peak-hour demand since water use variation for different user classes is not the same.

Per Capita Demands The residential and total system demands may be divided by the population to determine per capita demands for both categories. Typical units of measurement are gallons per capita per day (gpcd) or liters per day per capita (l/d per capita). If the information is available, the population may also be divided into single- and multifamily categories to determine the average number of people for each connection category. Typical single-family residential connections average between 20 and 30 people per connection; typical multifamily connections average between 1.0 and 2.0. Depending upon location, climate, cost, and mix of residential versus commercial/ industrial connections, typical average-day demands may vary between 80 and 150 gpcd (300 and 570 L/d per capita) for residential demand and between 100 and 200 gpcd (380 and 760 L/d per capita) when all system demands are included. Of course, unusual mixes of connections and large industrial and irrigation demands can cause significant variations in these typical values.

Peaking Factors Although multipliers can be used to estimate demand variations, good planning requires measurement of maximum-day and peak-hour demands. Flow demand variation depends on climate, size of system, and the characteristics of the connections. For example, a system dominated by residential connections will exhibit significantly different weekly and diurnal flow variations than a system dominated by industrial/commercial users.

For determining source-of-supply, storage, and conveyance needs, the following three demand periods need to be determined: average daily demand (ADD), maximum daily demand (MDD), and peak-hour demand (PHD). For typical water systems serving a mix of both residential and commercial connections, MDD ranges from 1.5 × ADD to 3.0 × ADD. PHD ranges from 1.25 × MDD to 2.5 × MDD, or 2.5 × ADD to 5 × ADD. (Note that these values do not apply to all systems in all areas.)

For most typical systems, peak-hour demands (other than emergency/fire flows) generally occur during weekdays in the late-afternoon or early-evening hours. This corresponds to peak residential use after the range of working and school hours. In most climates, peak demands also occur during summer months as a result of increased domestic use and irrigation demands. Figure 7–5 illustrates diurnal flow variation in a typical mixed-use water system.

Fire Fire demands are determined based upon land use densities and the critical structure classification (i.e., highest fire flow requirement) for each pressure or service

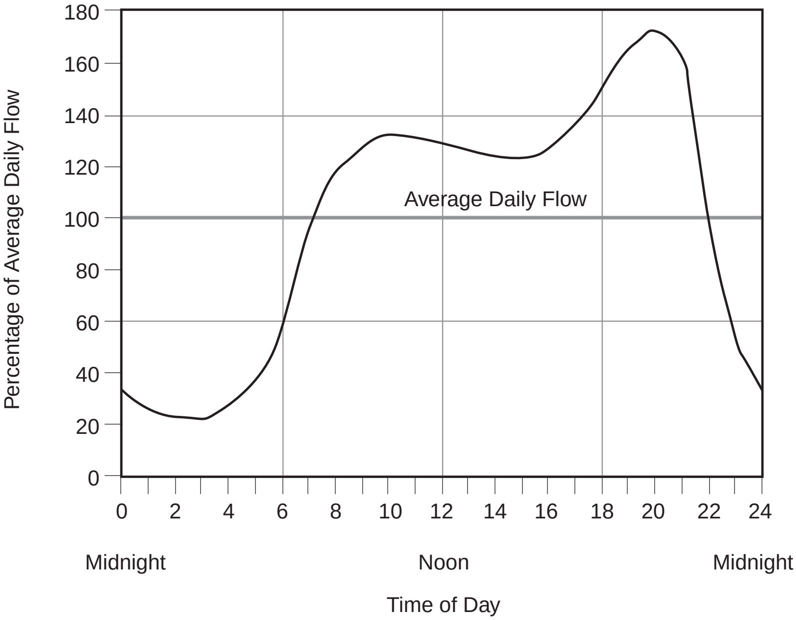


Fig. 7–5. Sample diurnal flow variation

zone within the water service area. Usually the local fire marshal establishes required fire flows based upon building and fire code standards and insurance agency rating requirements.

Fire flow requirements are typically defined in terms of a rate of flow at a minimum pressure for a specified duration. For example, in a residential area the minimum fire flow may be defined as 1,000 gpm (63 L/s) for 2 hr; a commercial area or school may require 4,000 gpm (250 L/s) for 4 hr. During a fire demand, a water system should be capable of maintaining a minimum pressure of 20 psi (140 kPa) at the point of fire flow delivery as well as throughout the rest of the system. In some systems, a high fire demand at one location can cause very low or even negative pressures in other parts of the system. These low or negative pressures can cause problems with other services and future water quality. Water system planners and designers should use a network model to simulate design fires at key locations to determine impacts on the distribution system. Alternative improvements to fix low-pressure problems can then be evaluated.

A practical limit on fire flow for typical midsized municipal water systems is between a minimum of 1,000 gpm (63 L/s) and a maximum of 5,000 gpm (320 L/s). Larger systems may need to provide for much higher fire flows. For typical systems, fire flows may be critical in determining minimum water main sizes. In addition, depending upon the criteria a utility uses, emergency fire flow volumes can account for a significant percentage of volume required in water storage reservoirs.

The American Water Works Association (AWWA) Manual M31, *Distribution System Requirements for Fire Protection*,[2] provides an excellent source of information regarding planning for fire protection. This manual references other materials from the Insurance Services Office and the National Fire Protection Association.

Unaccounted-For Water Unaccounted-for water results from system leakage, inaccuracies in source-of-supply and service metering, line flushing, unmetered connections, and authorized and unauthorized use of system hydrants. It can represent anywhere from 5 to more than 50 percent of a system's water production. A typical system operations goal is to reduce unaccounted-for water to less than 10 percent of the total supply.

Projected Water Demands

Given adequate information on current system demands, a utility can use values for land use, zoning, current population, and various unit demands to predict future system demands. The methods and unit demands used to predict future flows will depend largely upon the extent and nature of available data and methods used in other coordinating plans. Water planners should coordinate this process with local governing (city and county) planning agencies.

Service Connections or ERUs Based on planning department growth rates and zoning information for developable land, a utility can estimate the future numbers of residential, commercial, and industrial ERUs that will be added to the system over the planning period. It can then apply the average demand to the number of projected ERUs to determine total future system demands. If sufficient data are available, the utility can perform cross-checks to determine if the projections per capita and per connection are reasonable.

Depending upon the complexity of the system, it may also be necessary to determine the rates of growth in water system demands for each pressure or service zone. For more complex systems, the utility may need to know these system demands to determine source-of-supply, pumping, distribution, and storage requirements by pressure zone.

Water Conservation Impacts The water conservation plan is discussed later in this chapter. The conservation plan recommends programs to reduce water demands. Implementation of these water conservation programs can have a significant impact on future water system demands. Conservation and reuse measures may be as significant as switching from a nonmetered flat-rate system to a metered system with an increasing or inverted block rate structure; or they may be as simple as a public education program to reduce water use. Implementing typical conservation measures may serve to reduce future average unit demands. Of course, the impacts of conservation measures on future demands will vary greatly, depending upon conservation opportunities, climate, and other system characteristics. Figure 7–6 shows a typical demand forecast for a growing system. The range of demand growth accounts both for varying growth rates and for implementation of conservation.

Geographic Information System Tools

If available, geographic information system (GIS) data can provide water planners with new sources of information for the development of water demand forecasts. Relevant GIS coverages may include parcel development information, zoning, and traffic data. Manipulation of this data can be used to predict future parcel development, population growth, and future water demands.

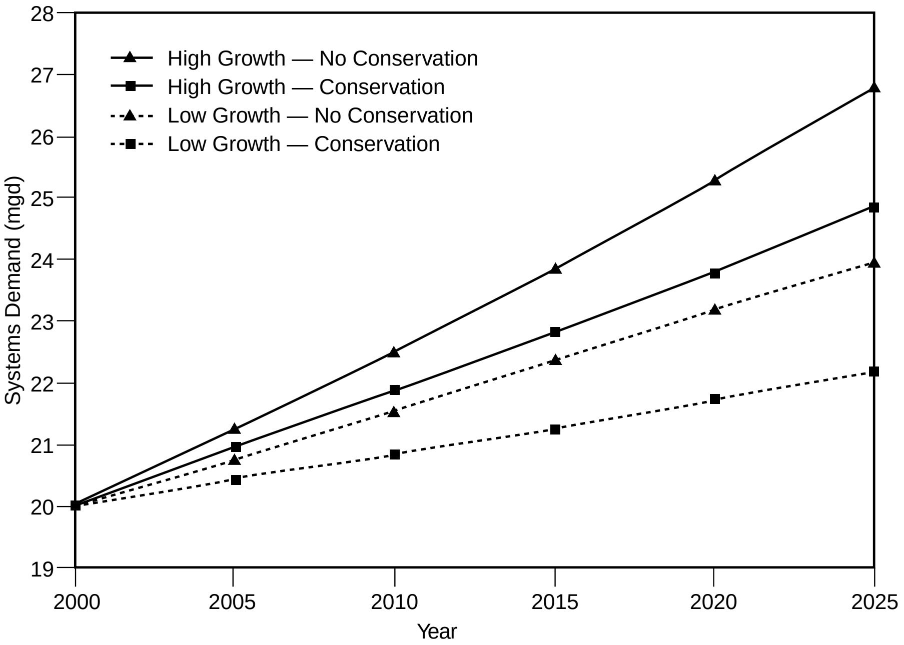


Fig. 7–6. Typical demand forecast

PLANNING DATA MEMORANDUM

At this point in the planning process, system planners should summarize all of the service area and growth assumptions for the service area in a planning data memorandum that is issued for review by the utility staff and other interested parties. These other parties may include adjacent water purveyors, city and county planning departments, and the state agency governing water system planning. Often the planning department will have an approved service area plan that defines the future service area land use and population.

Obtaining everyone's agreement on the planning assumption *before* the detailed system analysis is completed is an important step. Without this intermediate review and comment process, much or all of the remaining detailed analysis could be based upon incorrect assumptions. It could then be very costly and time-consuming to redo the analysis with new planning assumptions.

CONSERVATION PLAN

As mentioned earlier in the context of demand forecasting, implementation of a conservation plan can have a significant impact on future water demands. A water plan should include discussion of and planning for appropriate water conservation measures. These measures may be divided into four general categories: public education, technical assistance, system improvements, and customer incentives. The following list itemizes typical conservation measures in each of these categories that water utilities may implement to reduce system demands:

- Public education
  - School outreach
  - Speakers
  - Bill mailings
  - Local notices
  - Education booths at local fairs and festivals
- Technical assistance
  - Specific customer assistance
  - Water use studies
  - Inclusion of water consumption histories on bills
- System measures
  - Source and service meter checking and replacement programs
  - Leak detection and repair programs
- Customer incentives
  - Low-flow shower or faucet kits
  - Xeriscaping
  - Conservation rate structure
  - Financed retrofits
  - Water reuse

The water plan should provide an overview of the conservation plan. The effectiveness of specific conservation activities should be addressed. There is little value in time-consuming and expensive efforts to influence conservation if the efforts are ineffective. The length and complexity of the conservation plan will vary significantly, depending on utility size and state and local regulatory requirements. The conservation plan text may also be included in the appendix of the water system plan.

EVALUATION OF SOURCES OF SUPPLY

The next step is to use the planning data and agreed-upon assumptions to evaluate the details of the water system. This evaluation begins with a review of the adequacy of the sources of supply. Typical water systems will be supplied either by surface water sources, such as lakes, rivers, and streams, or by wells tapping groundwater sources. Some systems may have a combination of both surface water and groundwater sources. Other systems may not have any of their own sources of supply, instead relying completely upon receiving water from a regional supplier or another adjacent purveyor.

The purpose of this section of a water system plan is to assess both the quantity and quality of existing sources and to provide an:

- Assessment of the need for developing additional sources to meet the range of projected future system demands.

- Evaluation of water quality to determine whether different or additional water treatment is needed for existing and potential sources to meet current and future water quality requirements.

Changing Regulations

In its evaluation of water quality issues, a utility must consider and try to anticipate changing regulations. For example, current water quality evaluations should consider what changes in source of supply and/or treatment strategies may be required as a result of implementing new federal Safe Drinking Water Act (SDWA) and state regulations for both surface water and groundwater. Chapter 1 provides a discussion of anticipated water supply regulatory changes.

Existing Sources

As specified earlier in this chapter, existing sources of supply have already been listed in the description of existing facilities. Thus, this section of the source-of-supply evaluation should briefly summarize the capacities and water quality characteristics of each of the existing sources of supply.

Quantity The total reliable capacity of each of the sources should be clearly defined. For example, if the capacity of a well has been degrading over time, the actual firm capacity of the well should be listed. For wells, any available water table depth information should be summarized. Some surface water sources may have restrictions on summer pumping because of low flows or minimum in-stream flow requirements in the stream or river. The maximum and sustainable withdrawal rates, age, condition, and estimated remaining life of each source should be listed.

Water Quality The utility should summarize the raw and, if applicable, treated water quality characteristics of each of the existing sources of supply. Any developing water quality trends, such as increasing turbidity or more frequent positive coliform tests, should be described. In addition, current applicable federal and state regulations regarding water quality of the system sources should be listed. Generally, the water system will have to comply with provisions of the SDWA. Various states have codified provisions of the SDWA in different ways and in some cases have added some of their own, more stringent water quality requirements. The utility should make a brief evaluation of how well each of the sources complies with the provisions of the governing water quality law.

Needs Assessment The goal of this evaluation of existing sources is to determine the need for additional sources of supply. The total firm capacity of the existing sources should be compared to the range of projected demands to assess whether any additional sources of supply will need to be developed. This comparison will determine both the volume required and the schedule of development of new sources, if they are needed.

Finally, the needs assessment should state whether additional treatment of existing sources is needed. In some cases, treatment can also serve to increase the firm capacity of existing sources. For example, in cases where unfiltered surface water sources are shut down because of high-turbidity events, filtration could add significantly to the firm capacity of the sources.

Potential Sources

If a need for additional sources of supply has been identified, all of the potential alternative sources should be listed. New sources of supply can include the following:

- New wells
- Rehabilitation of existing wells
- Artificial groundwater recharge
- New surface water intakes
- Adding capacity to existing intakes
- Treatment to improve firm yields of existing sources
- More aggressive conservation measures (to reduce demands)
- New interties
- Water reuse

Water Rights Any discussion of developing new sources should include a summary of needed water rights. Utilities may often find it worthwhile to consider trading or reassigning existing water rights to new or different sources of supply. Water rights information should be included in the water rights assessment section (discussed below).

Reuse Reuse involves using nonpotable water or reclaimed treated wastewater in place of other potable water sources. In some areas, opportunities for reuse can significantly reduce the need for potable water. Sources of water for reuse can include stormwater impoundments, wastewater/sewage treatment plant effluent, and industrial/commercial process water. Under certain conditions, the potential uses of nonpotable or reclaimed wastewater can include the following:

- Irrigation of public areas, such as landscaping and athletic fields
- Irrigation of public and private golf courses
- Aquifer recharge
- Fire protection
- Industrial and commercial cooling and process water
- Gravel washing and processing
- Nonpotable uses such as toilet flushing, heating, and cooling in large buildings and downtown developments
- Street washing
- Storm and sanitary sewer flushing

As water supplies become more critical, reuse is gaining more acceptance as a viable source of new supply. Of course, the use of reclaimed and nonpotable wastewater is subject to state and local requirements. Indirect potable water reuse—which takes place when wastewater is treated to high standards before being discharged into a water body that is also used as a raw-water source for drinking water—is being more frequently considered to supplement natural flows into lakes and streams that are used for water supply. Regulations are being developed and modified to fit regional needs.

WATER RIGHTS ASSESSMENT

Water rights law varies significantly from state to state. Most states require surface water and/or groundwater water rights permits. For growing systems, water rights become critical to the ability of a water utility to serve future customers. For built-out utilities experiencing little or no growth, the issue may be the protection of senior water rights to maintain capacities of existing groundwater or surface water supplies. In some states, junior water rights may be especially vulnerable to changing environmental requirements or drought.

Because water rights laws vary by state, a water planner must consult with the governing state agency regarding specific requirements for water rights applications. *Water Rights of the Fifty States and Territories,* published by the American Water Works Association,[3] provides an excellent general reference for water rights in the United States.

Current Status

The water plan should summarize the current status of all existing water rights. This summary should include the designations, dates, and types of water rights, as well as annual and maximum withdrawal capacities. The key is establishing a basis for determining whether a utility will need to apply for additional water rights to meet future demands.

Needs Assessment

Water rights needs may be expressed in terms of either meeting total annual demand (typically expressed in acre-feet or cubic meters) or of meeting maximum-day demand (typically in cubic feet [meters] per second, gallons [liters] per minute, or million gallons [megaliters] per day). It is possible for a utility to have adequate rights to meet total annual demands but not maximum-day demands, and vice versa. The water rights assessment should summarize available water rights and itemize the needs for additional rights. It should also give a preliminary evaluation of the water system's ability to obtain additional water rights. To obtain these rights, a water system often must demonstrate that it is using all of its existing sources of supply efficiently and that it has a plan to implement an appropriate level of conservation.

Plan and Schedule

The water plan should utilize demand forecasts to itemize and schedule additional water rights needs. The schedule for submitting a water rights application should allow for anticipated approval periods. The goal of this part of the water system plan is to lay out a process to ensure that adequate water rights will be available to meet future system needs.

SOURCE PROTECTION PLAN

The source protection program is that part of the water plan summarizing how the water system intends to protect the quantity and quality of its sources of supply. For groundwater systems, this may take the form of a wellhead protection program. For surface water sources, protection may involve elements of a watershed control program.

Wellhead Protection

A state wellhead protection program is required for public water systems by the federal Safe Drinking Water Act. Each state has established its own criteria and program for implementing the federal mandate.

The details of a wellhead protection program are not necessarily included as part of a water system plan. However, the water system plan should either outline and schedule the development of a new wellhead protection plan or define projects that are being implemented as part of an existing plan. Following is a basic list of items that should be included in a wellhead protection plan:

- An overview of the plan
- An assessment of how susceptible the well sources are to contamination
- A map and an explanation of how the wellhead protection areas are delineated
- An inventory of known and potential land uses and sources of contamination within the wellhead protection area
- A spill response plan, including coordinating documentation for those who might respond to emergency spills

This information can then be used to devise a contingency plan in the event that a source becomes contaminated and a new source of supply needs to be developed. A spill response plan should include documentation of coordination with those who might respond to emergency spills. Any efforts at regional coordination with other purveyors should also be documented. Finally, the plan should define or include methods to educate and notify the public regarding groundwater protection efforts within the wellhead protection area.

Watershed Control

Public water systems with surface water sources of supply are required to have some measure of watershed control for source protection. Individual states have interpreted this federal mandate differently. The extent of watershed control needed or possible varies significantly, depending on the size of the watershed, ownership, land use, and treatment. Because each watershed and water supply combination is different, each watershed control program will be unique.

A complete watershed control program will not necessarily be included in a water system plan. However, the water system plan should summarize any existing plans and/or itemize and schedule the implementation of control program recommendations. In general, a watershed control program should contain:

- A description of watershed characteristics
- A map and description of ownership and land use within the watershed
- Watershed management and control plans

- A summary of the watershed-monitoring program
- System operations procedures to maintain water quality

WATER QUALITY MONITORING

The water quality monitoring section of the water plan should summarize applicable state and federal regulatory monitoring requirements and outline current and planned water system monitoring programs. Monitoring requirements vary with the type and size of water system and with specific state interpretations of the federal Safe Drinking Water Act and its amendments.

Typically, minimal monitoring programs require monitoring of raw water sources of supply, treated water effluent from treatment plants, the point of entry into the distribution system, and key locations within the distribution system. The number and types of samples required will depend upon the system size, types and number of sources, number of connections, and results of previous monitoring and testing. Some monthly and annual monitoring requirements will not change, whereas others may be modified as a history of analytical results develops.

For larger systems, this section may not specify the full details of the water quality monitoring program. Rather, a detailed monitoring program may be included in the operation and maintenance plan; it would specify monitoring locations, a schedule, and sampling and testing protocols. These details will be further discussed later in this chapter (see ‘‘Operation and Maintenance Plan’’).

WATER SYSTEM ANALYSIS

The water system analysis presents guidelines and results of comprehensive performance evaluations of key physical components of the water system: treatment, storage, and transmission and distribution. Basic source-of-supply needs have already been identified and evaluated. The related impacts of possible source-of-supply projects should now be evaluated in the water system analysis. For example, one option for expanding or adding a new source of supply may have significantly more costly transmission-and-distribution piping needs than another.

This section of the water system plan should also contain a brief evaluation of system vulnerability. If necessary, it should include recommendations to reduce the likelihood of adverse system impacts during emergency situations.

Treatment

The need for any additional treatment was already discussed in the source-of-supply evaluation. In this section, specific water treatment alternatives should be conceptually described, including treatment process schematics and preliminary order-of-magnitude cost estimates. The range of costs should account for both capital facilities and operation and maintenance expenses. Downstream impacts on transmission and conveyance should be described for each treatment alternative.

Storage

This section briefly discusses planning aspects of water storage. Water storage facilities are discussed in more detail in Chapter 27. The water plan’s inventory of the existing system already itemized existing storage capacity; reservoir types, materials, ages, conditions, and date of last inspection and/or cleaning and coating. The water system analysis should contain an evaluation of water storage needs for the system. For systems with multiple pressure zones, storage needs should be determined for each zone. Storage needed within a water system (in excess of unusable, or ''dead,'' storage) is generally divided into four categories: operational, equalizing, standby (or emergency), and fire flow.

Operational Storage Operational storage is used to provide water during normal operation when sources of supply are not delivering water. It enables a utility to avoid having short pump-cycling times. This storage volume does not apply to systems operating under continuous pumping or to the supply from a water treatment plant.

Equalizing Storage Equalizing storage is used to supply the difference between peak-hour demands due to normal diurnal flow variations and the capacity of the sources of supply. Generally, it is not economically efficient to develop sources of supply to meet peak-hour system demands. However, if the sources of supply can meet peak hourly demands, the equalizing storage volume needed is minimized.

Detailed daily (diurnal) flow records, if available, can be used to determine the volume of equalizing storage required to supplement sources of supply to meet peak demands. However, in most cases detailed diurnal flow information is not available. Equalizing storage requirements can then be calculated by comparing the capacity of all of the sources of supply versus peak system demands.

Standby Storage Standby storage is used to provide water to a system during unusual or emergency conditions when a source of supply may be temporarily unavailable. A system with several independent sources of supply may require less standby storage than a system with only one source.

For systems with one source of supply, a typical rule of thumb is to provide a minimum standby storage volume equal to twice the average-day demand. Systems with multiple sources can reasonably subtract the capacity of all but the largest source from this volume to determine their minimum. Another accepted criterion is to provide a minimum of 200 gal (760 l) per equivalent residential unit. In making its final determination of standby storage, a water system should account for the likelihood of losing primary and/or secondary sources of supply.

Fire Flow Storage Fire flow storage is used to provide the required volume of water to meet fire-fighting needs within the water system's service area. In systems with multiple pressure zones, fire flow storage volumes may vary for each zone. The volumes required will depend upon the type and density of construction within the areas served by the system. The simple way to determine the volume of storage for any service zone is to multiply the rate of fire flow by the required duration. For example, a fire flow of 1,000 gpm (63 L/s) for 2 hr would require 120,000 gal (450,000 L) of storage. The local fire authority will generally determine the amount of fire flow storage required for various types and densities of development within each service zone. Storage reservoirs should be designed so that, at the lowest level of usable storage, a minimum residual pressure of 20 psi (140 kPa) can be maintained within the distribution system during a fire.

Combining Standby and Fire Flow Storage Depending upon the configuration of the water system and the assessed likelihood of needing both emergency standby and fire flow storage, a utility may determine that providing both types of storage is too conservative. In that case, the local fire authority and water system planners may agree that the system should provide enough storage to meet only the more stringent of either criterion rather than the summation of both. If so, the water system plan should clearly state the rationale for this decision and specify under whose authority it was made.

Impact of Storage on Water Quality The configurations, relative locations, and operation of storage reservoirs have important water quality impacts that must be considered in the water system analysis. Standpipes and reservoirs that ''float'' on the system through a single inlet/outlet pipe may have poor circulation, which can lead to stagnant water and bacterial growth within the reservoir. Reservoirs that are poorly located in the system can have long retention times and create water quality problems. In many cases, a utility can remedy these problems by changing the inlet and outlet configuration or by installing baffles to circulate flow within the reservoir.

Utility personnel can use water surface elevation and flow data to determine how well a reservoir is operating. Engineers can also develop hydraulic models to make extended-period simulations to determine how an existing or proposed reservoir may operate under future conditions.

Transmission and Distribution

Transmission-and-distribution pipelines and pump stations convey the water from the source of supply to storage reservoirs and ultimately to the users.

Facilities The water transmission system consists of pump stations and larger-diameter piping that convey water from a source of supply to the point of entry into the distribution system. This point of entry may be a reservoir, pump station, or the first service connection to the system. The distribution system consists of the network of pipes, booster pump stations, and pressure-reducing valves that convey flow from the system point of entry to the users. Isolation and control valves are located throughout the system. Fire hydrants are typically spaced as needed to serve the type and density of construction. In some larger systems, secondary disinfection may be required to maintain a chlorine residual within the distribution system.

As part of the system analysis, the planning engineer should evaluate the condition and performance of the transmission and distribution system. As deficiencies are identified, alternative improvements should be developed to mitigate the deficiencies.

A schematic diagram is an excellent way to describe the existing water transmission-and-distribution system, as well as recommended improvements. Figure 7–7 is an example schematic.

Evaluation of Condition The ability of the transmission-and-distribution facilities to meet current and future needs is related to the age, material type, condition, and hydraulic capacities of these facilities. As part of the water plan, the engineer should tour facilities with utility operational staff to note the general condition and operational problems. Of course, the condition of aboveground facilities, such as booster pump stations, can be directly observed. However, the condition of water pipelines must

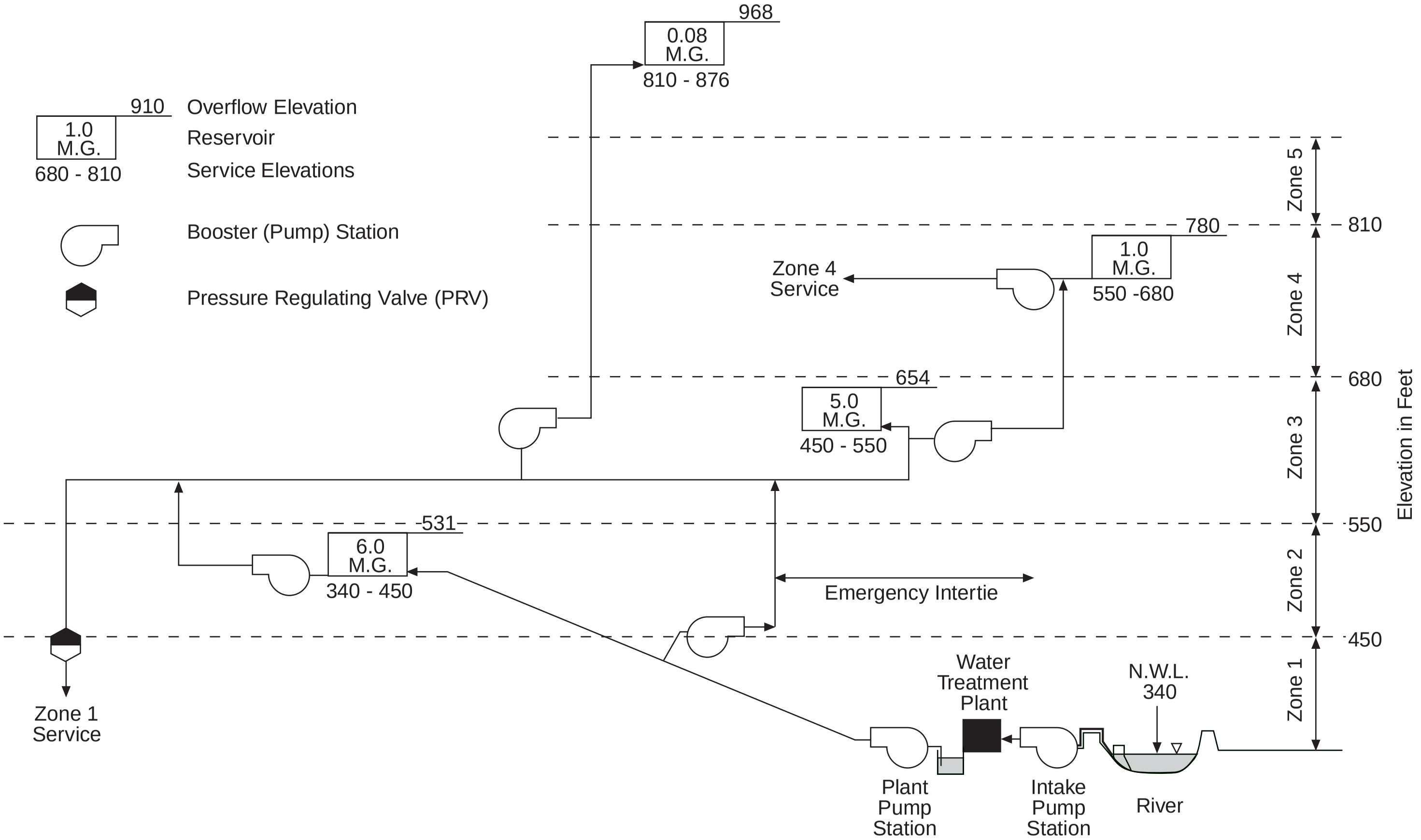


Fig. 7–7. Example of a water system schematic

often be surmised based upon age, material, number of breaks, and observations made during system maintenance.

Water Quality Water quality is another key element of a distribution system's performance. See Chapter 22 for more discussion.

Some solutions to water quality problems involve looping of dead-end lines or more frequent line flushing. In some cases, secondary or more complex disinfection facilities may be required to maintain disinfectant residuals. System planners must carefully evaluate these solutions in more detailed water quality studies to avoid causing other problems, such as high levels of disinfection by-products in the distribution system.

Hydraulic Modeling Computerized hydraulic models are valuable tools for analyzing the ability of a transmission-and-distribution system to serve both current and future demands. Hydraulic modeling is especially important for growing and expanding distribution systems. These models can be used to simulate current and future system pressures, in-pipe water velocities, and other system operation characteristics. See Chapter 28 for more discussion.

A water utility can use a calibrated hydraulic model to help identify current system deficiencies and to simulate future conditions. This will enable the utility to answer such questions as:

- What diameter pipe should be used in a pipe replacement project?
- What diameter pipe should be used for a line extension to serve a newly annexed area?
- What fire flow can be served to a proposed school site?
- What system improvements would be necessary to serve the desired fire flow to a proposed school site?
- What size booster pump station would be needed to serve an area proposed for development?
- Which new storage reservoir site would provide the most benefit in terms of equalizing system pressures under MDD conditions?
- How would looping of two dead-end lines impact chlorine concentrations in that part of the system?
- What initial chlorine concentration is necessary to maintain a residual at the farthest point in the system under minimum demand conditions?

Telemetry and Controls

The telemetry and control system should be reviewed to identify any needed improvements. The system should be able to provide:

- Adequate controls and monitoring functions to operate the system under normal conditions.
- Suitable alarms and emergency systems to ensure safe operation of the system during unusual conditions.
- Data collection systems sufficient for monitoring flow and water quality parameters to comply with regulatory reporting requirements.

Vulnerability Evaluation

Certain water system facilities may be susceptible to damage or failure during emergency conditions. Older buildings, reservoirs, and other structures may be especially vulnerable. Depending upon the location of the water system, emergency situations of note may include earthquakes, floods, blizzards, hurricanes, or tornadoes. In addition, there may be a high probability of long-term power failures in certain systems.

Conducting a detailed analysis of many facilities as part of a water plan may not always be possible. However, the water plan should at least identify vulnerable facilities in its list of deficiencies (see next subsection), and it should describe a program to begin reducing system vulnerability. Examples of typically vulnerable facilities include source-of-supply pump stations (flooding), pipeline bridge crossings (earthquake and flooding), and elevated storage tanks and standpipes (earthquake).

Summary of Deficiencies

The water system analysis section of the water plan should conclude with a summary table of system deficiencies listed by project category. Table 7–2 is an abbreviated example of some of the types of deficiencies and the level of detail that may be appropriate for this summary.

CAPITAL IMPROVEMENT PLAN

The capital improvement plan (CIP) brings together all of the proposed alternatives to upgrade the system or improve system deficiencies into a prioritized and practical list

TABLE 7–2. Sample Summary of Deficiencies for a Hypothetical Water System

Deficiency	Category	Possible Solutions
Poor circulation and stagnant water in pressure Zone 2's storage reservoir	Water quality	• Modify the inlet and outlet configuration and add baffles to the tank.
An additional 500,000 gal of storage needed in pressure Zone 1 for standby/fire storage	Storage	• Add 500,000 gal to Zone 1 storage at existing site. • Add storage to new site in Zone 1.
Inadequate fire flow pressure at Central High School	Distribution	• Replace 8-in. pipe on 3rd Ave. from South St. to Central St. with 12-in. pipe. • Loop 12-in. pipe on 5th Ave. to connect parallel 12-in. pipe to 8-in. pipe on 3rd Ave. • Add booster pump station at 3rd Ave. and South St.
No ability to monitor condition of pressure Zone 2's booster pump station	Telemetry and controls	• Add remote monitoring and alarm telemetry.

of projects that can be implemented over the next 20 years or so. The most critical projects should typically be planned in annual schedules over the first 5 years of the plan. The remaining identified long-term projects should be scheduled to occur in 5-year increments for the remaining 15 years. The CIP schedule should then be updated during the annual budgeting process with a major plan review and update every 5 years. An important consideration in the development of the CIP is that the project implementation schedule be practical and financially viable for the water system.

Selected projects should be classified by project category, such as source development, transmission, treatment, storage, distribution, water quality, or telemetry and controls.

Evaluation of Alternatives

Some deficiencies may have more than one potential solution. The selection of a particular solution to an identified deficiency may depend upon an assessment and comparison of some of the following criteria:

- Capital cost
- Operation and maintenance costs
- Design life
- Land-use and visual impacts
- Construction noise and traffic impacts
- Reliability and risk
- Ability to meet long-term project goals
- Environmental impacts
- Flexibility to meet changing conditions
- Potential for regional benefit

Projects that are expected to be implemented within the next 5 years should be evaluated in more detail than those that are expected to be built in the more distant future. Changing conditions may have an impact on future decisions. Hence, the evaluation of these more distant projects can be fine-tuned in annual plan reviews as more information becomes available and as higher-priority projects are put into operation.

Selected Project Descriptions

Brief descriptions should be prepared for each major project. The project description should include a project designation, the reason for the project, a description of the improvement, potential sources of funding, and a budgetary cost estimate. The cost estimates should include permitting, engineering, construction, administration, and contingency costs, as well as any applicable taxes, in current dollars. Budget costs will be escalated to the time of construction in the CIP spreadsheet summary table. Selected projects should be highlighted on the system map as shown in Figure 7–8.

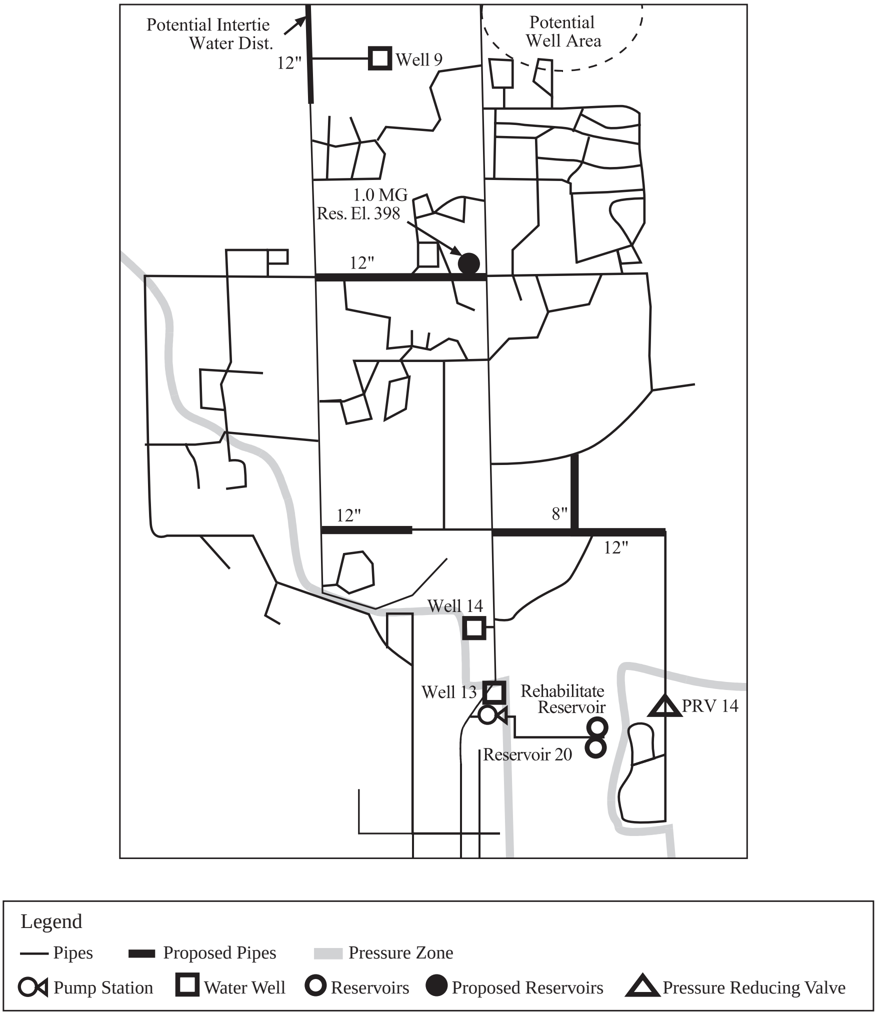


Fig. 7–8. Example of a capital improvement map

Prioritization and Scheduling

Selected projects are then prioritized and scheduled in order of importance and benefit to the water system. In scheduling these projects, planners should consider the practical funding capabilities of the system.

Projects to mitigate health and safety deficiencies in the system should generally receive the highest priority for implementation. Improvements that are necessary to accommodate growth should be scheduled based upon development and growth rate estimates. Although actual development demands and growth rates may vary from plan estimates, scheduling of growth projects is important to the development of a financial plan (as discussed later in this chapter).

Ongoing and annual improvement projects should also be listed in the capital improvement plan. These may include ongoing water line replacement programs, water quality monitoring programs, and water meter upgrading/replacement projects.

Capital Improvement Summary Table

Capital improvements can then be summarized in a spreadsheet table that will use a preset escalation rate to calculate the project cost for the scheduled year of construction. The spreadsheet can also be used for comparing ‘‘what if’’ scenarios of project scheduling to meet realistic annual funding objectives. Figure 7–9 presents sample CIP spreadsheet output.

FINANCIAL PLAN

The purpose of the financial plan is to determine the total costs for providing water service and for funding capital improvement projects. The financial plan should include the following: a summary of recent (last three years) of operating expense and income, a current budget and improvement-financing plan, a listing of revenue sources and allocations to projects, and a rate assessment.

Operating Budget Summary

A summary of recent past and future planned system operating expense and income will serve to provide a basis for the financial plan. The typical water system operating budget should include:

- Revenues from water rates, fees, and other sources.
- Capital sources from loans, bonds, grants, and special charges.
- Operating and maintenance expenses, including salaries and benefits, power, chemicals, monitoring and testing, materials and supplies, and transportation.
- General and administrative expenses, including salaries and benefits, office supplies and expenses, insurance, legal and accounting, engineering, fees, training, taxes, and depreciation.
- Annual debt payments, including principal and interest on loans and bonds.
- Capital improvement project costs.

The plan should then also summarize the planned installment payments to operating cash and emergency reserve accounts, as well as the total amounts of available revenue. The amount in the emergency reserve account may be established based upon the funding cost of replacing the most vulnerable critical facility in the system.

Revenue Sources and Allocation

Anticipated revenue sources for financing capital improvements may include general obligation or revenue bonds, system development charges, low-interest loans, grants, or water sales. The planned revenue source and allocation for each identified capital improvement project should be summarized in the CIP so that total amounts available from each source can be shown in projected operating budgets.

Water System Plan
Capital Improvement Schedule

Cost Estimating Year: 2000
Annual Cost Escalation: 3.5%

	Type of Project				Construction Costs in $1,000						
								Funding Sources			
CIP No.	Classification	Improvement	Pressure Zone	Title/Description	2000 Cost	Constr Year	Constr Year $	SDC %	SDC $	Wfund %	Wfund $
100	Storage	Miscellaneous	100	Miscellaneous	75.0	2000	75.0	0%	0.0	100%	75.0
101	Distribution	Infrastructure	101	Infrastructure	50.0	2000	50.0	0%	0.0	100%	50.0
102	Transmission	Infrastructure	102	Infrastructure	300.0	2000	300.0	0%	0.0	100%	300.0
103	Distribution	Infrastructure	103	Infrastructure	300.0	2001	310.5	0%	0.0	100%	310.5
104	Transmission	Infrastructure	104	Infrastructure	250.0	2002	267.8	0%	0.0	100%	267.8
105	Storage	Miscellaneous	105	Miscellaneous	75.0	2002	80.3	0%	0.0	100%	80.3
106	Transmission	Growth Extension	106	Growth Extension	150.0	2002	160.7	60%	96.4	40%	64.3
107	Transmission	Growth Extension	107	Growth Extension	325.0	2003	360.3	100%	360.3	0%	0.0
108	Distribution	Infrastructure	108	Infrastructure	150.0	2003	166.3	0%	0.0	100%	166.3
109	Transmission	Growth Extension	109	Growth Extension	450.0	2004	516.4	100%	516.4	0%	0.0
110	Transmission	Growth Extension	110	Growth Extension	215.0	2005	255.4	100%	255.4	0%	0.0
111	Treatment	Infrastructure	111	Infrastructure	150.0	2005	178.2	0%	0.0	100%	178.2

SDC = System Development Charges
Wfund = Water Fund

Water System Plan
Capital Improvement Plan
Annual Construction Cost Summary ($1,000)

Year	2000 Costs	Const Yr Costs	SDC $	SDC %	Wfund $	Wfund %
2000	425.0	425.0	0.0	0%	425.0	100%
2001	300.0	310.5	0.0	0%	310.5	100%
2002	475.0	508.8	96.4	19%	412.4	81%
2003	475.0	526.6	360.3	68%	166.3	32%
2004	450.0	516.4	516.4	100%	0.0	0%
2005-2009	365.0	433.6	255.4	59%	178.2	41%
2010-2014	0.0	0.0	0.0	0%	0.0	0%
2015-2019	0.0	0.0	0.0	0%	0.0	0%
Totals:	2,490.0	2,720.9	1,228.5	45%	1,492.4	55%

Fig. 7–9. Example of a capital improvement schedule

The goal of this budgeting process is to achieve a ‘‘bottom line’’ on the projected operating budget that shows a budgetary balance or even a surplus. If the planning process results in a deficit, adjustments have to be made on either the revenue or expense side of the ledger to achieve that balance. This may involve adjusting rates and fees charged to water customers.

Rate Assessment

Existing water rates should be summarized and evaluated in terms of the planned operating budgets and capital improvement schedule. A utility should determine any impacts on future rates prior to developing the final operating and improvement plan. Each system is going to have a unique combination of current water rates, debt ratios, operation and improvement needs, financial viability, and local political factors that will have to be considered to balance scheduled improvements with acceptable water rates.

Many regulatory agencies may require an analysis of rate structures that promote water conservation. This may mean changing from a flat rate or declining block rate to an inverted block rate. Of course, in the absence of careful planning, these kinds of rate structure changes can have significant adverse impacts on system demands and revenues.

Two AWWA references may be particularly helpful in financial planning and rate analysis: *Financial Planning Model for Water Utilities*,[4] which is a spreadsheet model; and AWWA Manual M1, *Water Rates*.[5]

OPERATION AND MAINTENANCE PLAN

The water plan should not include all of the details of a complete operation and maintenance plan, but it should summarize the following system O&M components:

- Staffing organization, responsibilities, and certifications
- Operation and control features and procedures
- Emergency response procedures
- Safety program
- Water quality compliance procedures
- Cross-connection control
- Record keeping and filing procedures

The full operation and maintenance plan is often completed as a separate document and referenced in the water system plan. The summary may be included as a chapter in the plan or as an appendix document.

Staffing Organization

A conventional organization chart showing the names and titles of staff members, with linkages to describe the supervisory and reporting structure, should be followed by a table with the following column headings:

Name	Title	System Responsibilities	Certifications

Operation and Control

A section of the O&M summary should include a basic description of the operation and control system. A system plan and schematic illustrating the locations and functions of major system components, as shown in Figures 7–3 and 7–8, respectively, are good references for describing system components. References to this plan and schematics should be followed by a table with the following column headings:

System Component	Normal Function	Relationship to Other Components	Alternative Operation

The next section should describe normal operation of the system, including tabulated procedures for each component as follows:

System Component	Normal Start-up Procedure	Normal Shutdown Procedure	Emergency Procedures

Routine and special maintenance required for each system component can similarly be described in a table with the following headings:

System Component	Inspection Frequency and Procedure	Maintenance Frequency and Procedure	Staff Responsible

The next section should include a table listing the equipment information, including make, model, and serial number; date of manufacture or installation; warranty coverage; and service representative information:

Make, Model, and Serial Number	Date of Installation or Manufacture	Warranty	Service Representative Address, Phone, FAX

The types of chemicals, amounts stored, special handling procedures, and supplier information should be included in a table for each location where chemicals are stored or used:

Chemical	Minimum/Maximum Amount Stored	Ordering and Special Handling Procedures	Supplier Information

Emergency Response Procedures

The emergency response procedures section contains the specific information outlining system operational procedures in the event of power failure, equipment failure, system damage due to natural disaster or vandalism, and accidental release of regulated substances. The emergency response program should at minimum include:

- An emergency call-up list that identifies, in ranked order, the personnel responsible for making decisions for each type of emergency situation. This list should include primary and secondary telephone numbers and addresses.
- Notification procedures in the event of a public health threat related to the water system. The procedures should be in conformance with state and federal requirements for notification of the public via local media resources.
- Names, telephone numbers, and system responsibilities of operational staff of adjacent water systems who may need to be called to coordinate emergency response programs.
- Names, telephone numbers, and responsibilities of the local fire and police personnel and operational staff of facilities, such as railroads, large industrial plants, and local military installations, who may need to be contacted to coordinate emergency response measures.
- A risk management plan (RMP) that covers the requirements of EPA and OSHA for all affected processes and chemicals.

Safety Program

The safety program section should provide the necessary details of safety issues related to the water system, and include:

- Procedures for confined space access
- Procedures for work in and around electrical services
- Procedures for chemical handling, including material safety data sheets for each chemical
- Safety training requirements
- Accident-reporting procedures

Water Quality Compliance

This section of the O&M plan outlines the procedures to collect data needed to comply with water quality monitoring requirements. Sampling and testing procedures for source water, treated water, and distribution system water should be summarized. Water sampling and testing requirements will vary depending upon sources of supply, system size, and state regulations. Some monitoring requirements, such as for coliforms, will remain relatively constant, while others will vary depending upon results of prior sampling and changes in law. The water quality monitoring procedures should be adjusted to reflect current system regulatory requirements. At a minimum, the procedures should:

- List the types of samples and tests that must be completed
- List what water quality parameters have compliance standards that must be met
- Describe the location where each sample is to be collected
- List the schedule of when each sample should be collected
- Describe handling and/or testing procedures or where outside tested samples should be sent for testing
- Describe follow-up testing in the event of test results that exceed limits or indicate a public health concern
- Reference the emergency response procedures in the event of test results indicating a public health concern

The monitoring procedures should also summarize future anticipated changes in monitoring requirements as a result either of waivers or of changing regulations.

Cross-Connection Control

State and local health agencies generally require that each water purveyor implement a cross-connection control program to cover equipment standards, installation, and testing of cross-connection control devices. The AWWA publication *Recommended Practice for Backflow Prevention and Cross-Connection Control*[6] is a standard cross-connection control reference.

Records and Filing

As part of the O&M summary, a water system should document its procedures for preparing, filing, and submitting records. This part of the report should specify (1) the records that should be kept, (2) paper and computer record file locations, (3) computer record backup procedures, (4) how long each type of record should be kept, and (5) the necessary procedures for submitting compliance information to regulatory agencies.

SUPPORTING DOCUMENTS

The appendix of the water plan should contain any other supporting documents. For example, it might make reference to the standard construction specifications the water system has adopted. These standard specifications may reference adopted industry standards, such as those published by AWWA or the American Public Works Association, as well as other state and local utility standards. Local standard policies for development by outside parties should also be summarized and referenced.

In addition, some states may require an environmental review or checklist before the water plan can be approved. If necessary, this document may be included in the appendix.

As applicable, copies of agreements regarding service areas, interties, sources of supply, wheeling, joint use, and mutual aid should be included in the supporting documents, as should listings of any local ordinances governing water system service.

REFERENCES

1. Harberg, Robert J., *Planning and Managing Reliable Urban Water Systems,* AWWA, 1997.
2. AWWA, *Distribution System Requirements for Fire Protection,* Manual M31, 2nd ed., Denver, CO, 1992.
3. Wright, Kenneth, *Water Rights of the Fifty States and Territories,* American Water Works Association, Denver, CO, 1990.
4. AWWA, *Financial Planning Model for Water Utilities: A PC-Based Program for Utility Rate Making,* Denver, CO, 1991.
5. AWWA, *Water Rates,* Manual M1, 4th ed., Denver, CO, 1991.
6. AWWA (American Water Works Association), *Recommended Practice for Backflow Prevention and Cross-Connection Control.* Manual M14, Denver, CO, 1990.

CHAPTER 8

Source Water Development

OVERVIEW

Procurement of water supplies is an inherently public activity. Physical, legal, regulatory, economic, political, and social drivers factor into changes in water supply. Where supplies are plentiful, water supply planning is relatively straightforward with physical and economic factors being most important. Where supplies are stressed, issues are more diverse and often contentious.

Economics play a significant, but not always dominant role in water development. Urban water users must pay whatever it costs for water, because it is the most fundamental requirement for life and virtually non-attainable if procured on an individual basis. People may limit their water usage in response to cost, through conservation, amenities, and choice of whether to locate water intensive businesses in a given community. This is in contrast to agriculture and water intensive industries, which can pay only what the market will bear.

The focus of urban water supply planning is to identify and fully utilize the least cost alternative first, then the next lowest cost alternative, and so on. The degree of use of each alternative is constrained by physical, legal, regulatory, economic, political, and social realities. Non-cost issues also weigh into water supply decisions.

INTEGRATED WATER RESOURCES MANAGEMENT

In supply-short regions, it is important to understand and manage the overall water resources available. The concept of a water budget is a very useful management tool. A study area boundary is defined, and all water crossing the boundary or stored within the boundary is counted (Figure 8–1).

The study area may be defined by watershed boundaries, such predictable groundwater boundaries as fixed head or constant flow boundaries, other physical boundaries, or political boundaries. Water may enter the study area via precipitation, rivers and streams, canals and pipelines, surface sheet flow, and groundwater inflow. Water may leave via evaporation, evapotranspiration, rivers and streams, canals and pipelines, surface sheet flow, and groundwater outflow. The difference between inflow and outflow is change in storage. Storage can be either on the surface or underground.

In water budgeting, each of these inflows and outflows is quantified with the best available data, then calibrated to known changes in storage, as reflected by groundwater

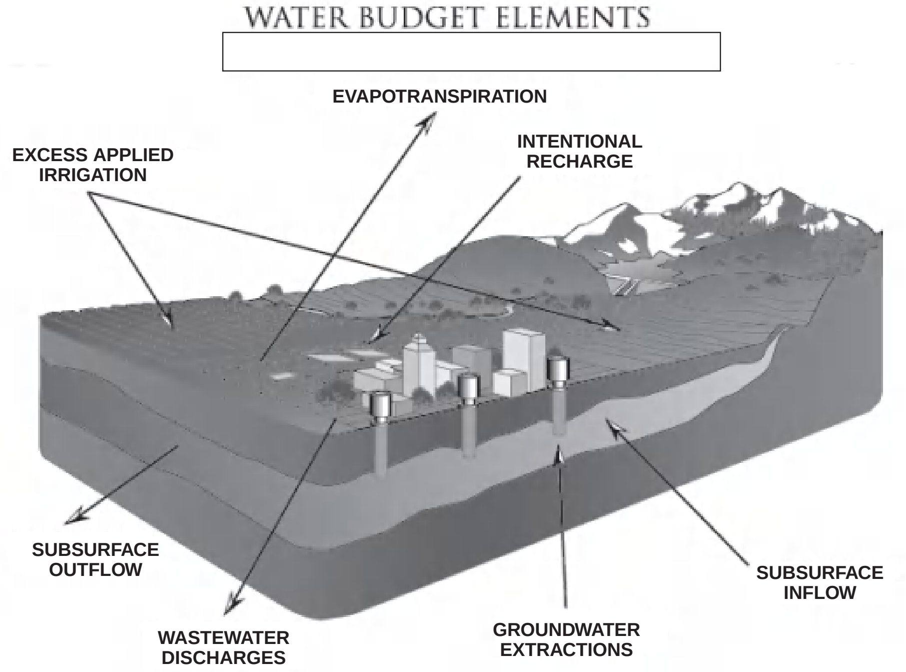


Fig. 8–1. Water budget elements

elevations and lake/reservoir stages. It is desirable to calculate the water budget over a number of years, including a major drought period and a wet period. A monthly or semiannual time step is usually the smallest useful increment, considering the accuracy and availability of data. After putting together an initial water budget, it is often necessary to develop a focused data collection effort to strengthen the analysis in future years.

The water budget is the vehicle to quantify the water supplies available and then to determine if additional water sources must be located. A balanced water budget will have no cumulative decline in groundwater and/or surface water storage. A negative result indicates a groundwater overdraft condition. A positive result indicates a surplus. Depending on the nature of the water budgets for areas adjoining, an overdraft condition may or may not stabilize at a lower level. The goal of integrated water resources management is to achieve a balanced or slightly surplus groundwater budget.

Managing an area's water resources as a whole is Integrated Water Resources Management. One can explore the effects of changes in land use and/or management strategies on the water budget. For instance, an intuitive assumption is that water conservation will have a positive effect on a water budget. Conservation reduces required water supplies. If the supply is from outside the study area, an input to the water budget is reduced. If the supply is from groundwater, water budget inputs remain the same. If the conservation program addresses only inside uses, this simply reduces the quantity of water that flows to a wastewater treatment facility.

ALTERNATIVE SOURCES

Potential supply sources include:

- Surface water (rivers, reservoirs, and lakes)
- Conjunctive use of surface and groundwater
- Imported water (canals, pipelines from distant sources)
- Groundwater (wells or sub-irrigation)
- Wastewater (agricultural returns, wastewater, power plants, gray water)
- Stormwater
- Conservation and demand management
- Seawater and brackish water

The source selected is a key factor in determining the nature of the required purification, transmission, and storage facilities. The supply must provide a reliable quantity of water for the long-term needs of the community, and preferably will have quality that minimizes the amount of treatment required. A detailed evaluation of all alternative sources should be made to compare yield, reliability, quality, treatment, collection, and distribution costs. Some of the general advantages and disadvantages of the alternative sources are discussed below.

Surface Water Supplies

The majority of the water served in the United States is derived from surface supplies, which require treatment to make them suitable for use as public water supplies. It is increasingly difficult, because of easier access and greater recreational use of streams, lakes, and watersheds, and urban, agricultural, and industrial development, for unfiltered surface water from protected watersheds to meet the federal drinking water regulations.

The quality of lake water is not as consistent as the quality of groundwater, but is more consistent than that of river water. The turbidity of river water may change rapidly during a heavy rainstorm, or from runoff due to melting snows. Lake water quality may change from wind-generated currents or when thermal stratification occurs. If the lake freezes over, the dissolved oxygen may be depleted in portions of the lake, with the result that the bottom deposits become anaerobic and many compounds become soluble. Many lakes and reservoirs have substantial quantities of iron, and occasionally manganese, in their bottom deposits. Normally they are of no particular importance to water engineers because they are oxidized and precipitated, or chelated, with organic compounds. However, when the bottom deposits in the relatively shallow areas of the lake become suspended because of wind-induced currents, these deposits can enter the intakes.

Deep lakes are affected by changes in density, which cause the water at the bottom of the lake to ‘‘turn over’’ and rise toward the surface. Because water is at its greatest density at 39.2°F (4°C). This effect can occur in both spring and fall. In the spring, surface water heats up to 39.2°F (4°C) and then sinks to the bottom, forcing water from the bottom to the surface. As winter approaches, the surface water cools, increasing in density and displacing the warmer water immediately underneath it. Significant changes in water quality can occur during these periods.

In order to divert water, the water purveyor must hold one or more state water rights or wholesale contracts from another water right holder. Reliability is defined by the availability of supply during an extended drought, and the ability to maintain (and preferably increase) diversions over time. Water rights may or may not guarantee the legal ability to divert. Public trust issues surrounding the health of the watershed and particularly its threatened and endangered species are placing new limits on diversion rates, schedules, and construction.

Groundwater Supplies

About one-fifth of the fresh water withdrawals in the United States are from groundwater resources.[1] When available in sufficient quantity, groundwater is often the preferred source. Most is clear, cool, colorless, and quite uniform in character. Underground supplies are generally of better bacterial quality and contain less organic material than surface water, but may be more highly mineralized. Natural protection of groundwater involves purification of water by infiltration into the soil, by percolation through underlying material, and by storage below the groundwater table. Groundwater is usually more uniform in temperature than surface water.

Groundwater sources also have some disadvantages. The cost of pumping groundwater may be greater than the cost of pumping surface water. Unless there are good geological data on the area, exploration to define the quality and quantity of groundwater could be expensive and speculative. Some groundwaters are highly mineralized, and contain large quantities of iron, manganese, sulfates, chlorides, calcium, magnesium, and other elements that are expensive to remove. Some groundwaters are high in color. Elements such as iron and manganese are held in solution at low pH values in the aquifers, because of the presence of carbon dioxide. Once the water is pumped to the surface, the free carbon dioxide is liberated, and the ferrous and manganous ions precipitate out of solution. If the groundwater is overpumped, water quality can change significantly as water from other formation flows into the system.

‘‘Groundwater under the direct influence of surface water’’ means any water beneath the surface of the ground with significant occurrence of insects or other microorganisms, algae or large diameter pathogens, such as *Giardia lamblia,* or significant and relatively rapid shifts in water characteristics, such as turbidity, temperature, conductivity or pH, which closely correlate to climatological or surface water conditions.

Conjunctive Use

When multiple sources of water with different characteristics are available, as is the case with groundwater and surface water systems, it may be possible to develop an operating strategy that capitalizes on the best features of the sources. This strategy is known as *conjunctive use.*

Some advantages offered by conjunctive use include the use of the water storage capacity of aquifers, improved water conservation (less evaporation from surface storage), and more uniform availability of water.

Surface water physical variables include availability, quality, losses (especially from reservoirs), and possible transfer. Groundwater aquifer physical variables include type of aquifer, storage capacity and hydraulic characteristics, losses, recharge features, and quality of groundwater.

Legal constraints for surface water consist of low flow requirements, surface water transfer, operation of reservoirs, navigation requirements, and allocation rights of users. Groundwater constraints include interaquifer water transfers, allocation rights of users, quality of recharge waters, and land subsidence.

Economic and financial variables include kinds of demands for water (agriculture, industry, municipal, hydroelectric, recreation, and waterborne commerce), return from economic activities, cost functions for technological activities, and project financing.

By proper balancing of the above variables, it may be possible to develop a conjunctive use system that is superior to reliance on either a ground or a surface supply.

Imported Water

Water brought in from outside the watershed is considered to be imported water. The actual source of the water could be surface water, groundwater, agricultural drainage, treated wastewater, or combinations. The water is conveyed to the study area through a canal or pipeline. The imported water could be developed remotely by the water purveyor, such as was the case with the Los Angeles Department of Water and Power and the Owens Valley/Los Angeles Aqueduct system or purchased from others through contracts.

The latter arrangement is common in the west, where states, local improvement districts, and the federal government, have developed large water supply projects. Depending on the source of the imported water, the reliability of the supply can be highly variable. Many water projects are under the same public trust pressures as local surface water supplies, with the result being a general reduction in supply, especially in the western United States.

Wastewater

As traditional water supplies are strained, purveyors look for anything wet. Treated municipal or industrial wastewater, agricultural drainage, and groundwater cleanup effluent are sources being put to beneficial use in many areas of the United States. Once these waters are treated, they are useful for agricultural irrigation, urban landscape irrigation, industrial cooling, and other water management functions, such as groundwater recharge, injection barriers to seawater intrusion, and wetland formation or enhancement. Efforts are underway in several communities to utilize more rigorous treatment to make these waters potable.

Depending on the study area, wastewater reuse may not be considered a new supply. If the treated wastewater were percolated to the groundwater, for instance, reuse on the surface would simply be a tradeoff of waters within the study area, and would have no impact on the water budget. If treated wastewater is discharged to a river and carried across a study area boundary or to the ocean, however, there would be a water budget benefit through reuse.

Treated wastewater is one of the most reliable sources of water, because of its consistent availability and yield increase with population growth. The obvious downside is quality and public perception. Even though water may be highly treated, the public or marketplace may demand that it not be used for some purposes. Also, unless the wastewater is de-salted in some manner, salt is recycled back through the system, leading to a long-term buildup in salt in the soil or aquifer(s).

Stormwater

Rainfall and snowmelt within a study area can sometimes be captured for beneficial use, reducing outflows from the study area and increasing the water budget. Stormwater can be used in the same manner as treated wastewater. Some uses may require physical treatment. Stormwater can be utilized for groundwater recharge by capturing stormwater in detention or retention basins and allowing the water to percolate into the ground, or releasing the water slowly to a leaky outlet channel.

Reliability is relatively poor for this water supply. Stormwater, by definition, comes at a time when irrigation demands are minimal. Effective utilization requires storage. Because stormwater is generally impractical as a potable water source, quality is not as serious an issue. The National Pollutant Discharge Elimination System (NPDES) permitting program requires best management practices (BMPs) to minimize urban runoff pollutants. Detention and retention ponds are examples of BMPs. Typically, urban runoff doesn’t have the dissolved solids load that treated wastewater has, so salt buildup is not as serious an issue. Metals and other pollutants tend to build up in the surface sediments of the ponds, which can be managed if removed in a regular maintenance program.

Conservation and Demand Management

Reductions in water demand are often considered as new water supplies. This is done primarily for comparison against other water supply alternatives. Typical water conservation measures that reduce inside demands include low-flush toilets, low-flow showerheads, automatic cutoff faucets, toilet dams, and horizontal axis (front-loading) washing machines. Measures for outside water conservation include recycle systems on car washes, irrigation systems that minimize runoff and deep percolation, low-water-use landscaping, and watering restrictions. Finally, a number of water pricing mechanisms, public education programs, and system-wide pressure reduction can be used to reduce water use.

Nearly all water used inside the home exits the home via the sewer. Very little is actually consumed. Therefore, inside conservation reduces both water use and wastewater production. Whether inside conservation actually reduces the strain on the water budget depends on whether treated wastewater leaves the study area. Inside conservation produces economic benefits, regardless of whether there are water budget benefits.

Outside water conservation can positively affect the water budget if runoff out of the study area and/or consumptive use are reduced.

Demand management refers to measures that alter the water demand to mimic the supply or reduce costs. Such measures may include ‘‘beat the peak’’ programs, which encourage people to reduce water use during peak periods during the day, by restricting landscape watering times and through public education. Because water systems are sized to provide peak-hour water demands, typically a factor of 2 to 4 times the average-day demand, peak demand reductions can have a marked effect on capital costs (Figure 8–2).

Another type of demand management is drought response. Here, more severe water use restrictions are implemented on a short-term basis. Such measures may include outside watering prohibitions, multiple toilet uses per flush, capturing shower water for plant watering or toilet flushing, etc. Interagency agreements are in place in some arid regions for seasonal agricultural land fallowing during drought years. Land fallowing or land retirement saves water by removing farmland from productive use for one or more years, and letting the land fallow, or unplanted. The water saved by not irrigating is used by the municipality in exchange for compensation to the idled farm economy. The water savings is only the avoided consumptive use of the crop, not the avoided diversion. For example, an acre of row crops in California might require 3.5 acre feet of diverted water to supply a 2.8 acre feet consumptive demand. The 0.7 acre feet difference percolates to the groundwater, or runs off, and is available for use by others. Therefore, the 2.8 acre feet is the only ‘‘true’’ water savings due to fallowing. The 2.8 acre feet would supply about 3 houses for 1 year in that climate.

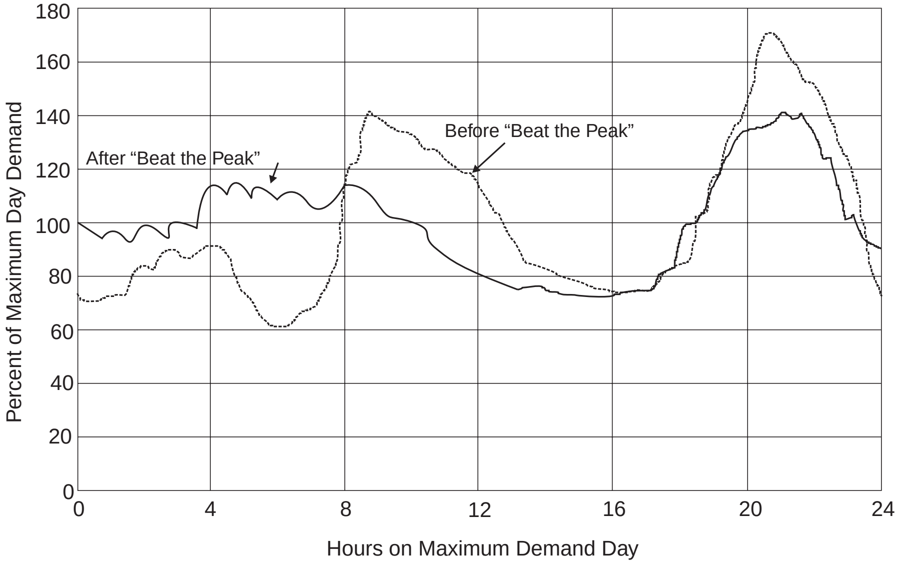


Fig. 8–2. Effectiveness of ‘‘Beat the Peak’’ program in Fresno, California

Once conservation measures are implemented and people become accustomed to the lifestyle change, conservation becomes a very reliable water supply alternative in terms of its consistent and long-term availability. However, many communities rely on conservation measures to weather droughts. Full-time conservation restricts the ability to further conserve in emergencies, effectively reducing the ‘‘factor of safety’’ of a water system.

Seawater and Brackish Water

Recent developments in membrane treatment have lowered the cost of desalinating brackish water and seawater to competitive levels in some communities. It follows that over the next 5–10 years, further advances are likely to make desalination the supply of choice in many coastal communities. Seawater supplies are virtually unlimited, and environmental factors are less critical than for fresh water supplies, so treated seawater may be one of the most reliable supplies. Brackish water from agricultural drainage and near-coastal surface water and groundwater sources are also becoming viable supplies in some communities. Brine disposal concerns limit the feasibility of inland desalination, however.

DESIGN PERIODS FOR WATER SOURCES

The quantity of water from a source(s) should be adequate to supply the total water demand of a community, as well as a reasonable surplus for anticipated growth. An analysis of the elements making up the total water demand of a community should be conducted, including but not limited to the following items: location, climate, population growth, type and character of community, fire protection, air conditioning, metering practice, cost of water, water quality, and pressure on mains.[2]

Surface water systems either use a source of water that is continuously adequate in quantity to satisfy present and reasonable future demands, or they convert an intermittently inadequate source into a continuously adequate supply by storing water during periods of surplus for use during periods of insufficiency.

In the case of multipurpose reservoir projects, the various demands for water should be carefully integrated. For an impounded source, allowances should be made for required water releases, evaporation seepage, and losses due to siltation. For major, costly projects, such as impoundments, which require difficult planning, property acquisition, and financing, the design period should be at least 50 years.

If the source of supply is located some distance from the point of use, a long supply pipeline is required. A study should be made of the economic size of the pipeline, taking into consideration cost of construction, expected future growth, and cost of operation based on power costs and other factors. Parts of the supply works that can be expanded without difficulty or excessive costs may be designed for a shorter period, such as 10 to 20 years. Examples include intakes, pumping stations, or certain pipelines.

Major projects, such as centralized well field and long transmission lines, may not be suited to construction in phases without incurring excessive extra costs, or they may involve difficult planning, property acquisition, or financing. For such projects, a design period on the order of 50 years may be appropriate.

For development of scattered wells feeding individually into the distribution system through short pipelines, and for other projects that are readily adaptable to construction in stages, shorter design periods of 10 to 20 years are appropriate.

SURFACE WATER SUPPLIES

Safe Yield

The following is a brief discussion describing safe yield concepts for streams and reservoirs and lakes.

Flowing Streams When no storage is provided, the minimum available (considering any competing water rights) stream flow of record must exceed the estimated future water demand on the maximum day. The best hydrologic data on minimum stream flows are those recorded on the specific watershed in question. In the absence of such data or with a short period of record, it may be necessary to use estimating methods. For such estimates, stream flow and weather records of contributing or adjacent watersheds should be used. Empirical formulas and ratios contained in published literature vary widely for watersheds of the same size; therefore, they should not be considered satisfactory criteria for judging the adequacy of a source unless they are supported by hydrologic data obtained from the specific watershed.

In making estimates, a careful study should be made of all factors that affect and determine the safe yield of a proposed surface water source, including such data as: geographical location, storm paths, prevailing winds, type and intensity of precipitation, topography and size of basin, orientation of basin, types of soil, types of vegetation, condition of ground surface, type and extent of artificial drainage, extent of surface storage in lakes and swamps, condition and slope of stream channel, average slope of basin, character of drainage net, and evaporation, infiltration, and other losses.

There are many methods of estimating runoff. The accuracy of the various methods depends upon the ability and the experienced judgment of the estimator in finding and supplying the correlating factors that will produce a realistic synthetic record. Consultation with the U.S. Weather Bureau and the U.S. Geological Survey is recommended.

Reservoirs and Lakes When the demand for water is greater than the minimum rate of flow in the stream from which the water is to be taken, an impounding reservoir may be required. The development of reservoir sites is discussed later in this chapter. In general, the ideal topographic conditions for a reservoir are a narrow gorge in which a dam may be built at minimum expense, and an expanding valley immediately above the gorge that will afford a large amount of storage per unit of surface area. This minimizes evaporation loss and the growth of algae and aquatic vegetation.

A reservoir will yield only part of the long-term average runoff of the watershed that it controls. The rest either will go over the spillway in times of flooding or will be lost to evaporation, bank storage, seepage, or siltation. Up to a certain point, increasing the amount of storage space in the reservoir will cause additional water to be on hand in drought times, and will enable the project to produce a greater dependable yield. However, the volume of water lost to lake surface evaporation also increases with reservoir size. Therefore, there is progressively less benefit from each increment of storage. For any specific site, there is a maximum practical capacity beyond which further enlargement of storage will not increase the yield, even though it might allow retention of more water during wet years.

The necessary reservoir capacity is dependent upon the quantity of water required to supply the community, the amount of water lost by evaporation from the reservoir surface, the loss of volume due to siltation and by seepage through and around the dam, and the minimum flow of the stream. The minimum stream flow can be determined accurately only by actual measurement over an extended period of time. Even where such flow records are available, there is always the possibility of a more severe drought than any previously recorded. However, most requirements will be met if provision is made for droughts that occur less than once in 50 to 100 years.

The dependable yield of a reservoir is defined as the amount of water that can be provided on a continuous basis, without deficit, under the full range of hydrologic conditions that might reasonably be expected to occur during the life of the project.[3] It is normally expressed in terms of acre-feet per year. Yields vary widely and are influenced by a large number of interrelated factors. Geographic location, rainfall, runoff, evaporation, reservoir storage capacity, the area-versus-capacity relationship, drainage area size, sedimentation, minimum drawdown limitations, and various other considerations will affect a reservoir's performance and effectiveness.

A key factor is the probable critical drought condition. As applied to reservoir yields, the severity of a drought should be considered in two ways: in terms of how much below average the runoff is likely to be, and also how long the deficient runoff conditions may be expected to last. Both aspects are important. The runoff deficiency must be made up by taking stored water out of the lake. A long drought means more loss to evaporation and less water available for supplementing the low runoff.

Potential critical drought conditions are usually derived from actual past records of runoff, rainfall, and evaporation on the stream in question or on other similar watersheds in the same general area. In most places, such records have been collected only during the past 30 to 50 years, but they constitute the most realistic estimate of what might happen in the future and the approximate duration of droughts for a given area. There is no assurance that there will not be even worse conditions in the future. Because of this uncertainty, it is frequently desirable to include a factor of safety in the yield estimates.

One way to do this is to assume some of the storage capacity to remain unused at the low point of the historical critical drought, as a reserve allowance. An alternative approach is to decrease the estimated historical critical drought, thus assuming a potential drought with that much less runoff than occurred in the past. It is desirable to have a comfortable margin between supply and demand.

The amount of storage required to carry the community through any of the recorded past droughts at the estimated rate of consumption may be determined by preparation of a mass diagram based on the best hydrologic data available. Mass curves of runoff should cover a period of several years of minimum rainfall.

Approval of the safety features of any planned structures should be obtained from the appropriate agency. Meeting safety requirements in the design of a dam for a reservoir requires that the spillway design flood be determined and provision made for adequate spillway capacity.

DEVELOPMENT OF RESERVOIRS

Volume Requirements

As noted above, the amount of storage required can be determined with a mass diagram. As shown in Figure 8–3, two curves are plotted on a single graph: one a plot of the cumulative stream flow over a period of years, and the other a plot of the cumulative demand over the same period of time. The stream flow curve is examined to determine the beginning of the longest dry period. This period will be evidenced by the portion of the stream flow curve with the flattest slope (begins at point A in Figure 8–3). A line is then drawn from the start of the longest dry period (point A) parallel to the cumulative demand curve (line AB in Figure 8–3). This line must intersect the cumulative stream flow curve if the reservoir is to refill (it does so at point B in this example). The ordinate value, DC, represents the volume of storage required to maintain the flow rate represented by the slope of the demand line.

The mass curve of water utilization need not be a straight line. Figure 8–4 shows a curve of irregular demand plotted with a curve of supply for the design dry period. The lower curve shows the total flow from October 1 to May 7 to be 110,000 acre-feet (135.7 Mm^3). The maximum vertical distance between the curves is the storage required to meet the needs of the project. If the worst period of record is selected for the stream flow mass curve, the maximum storage requirements are obtained from the mass curve. In the illustration, it was assumed that the reservoir was full on October 1, the beginning of the period. The greatest amount of storage has been used on

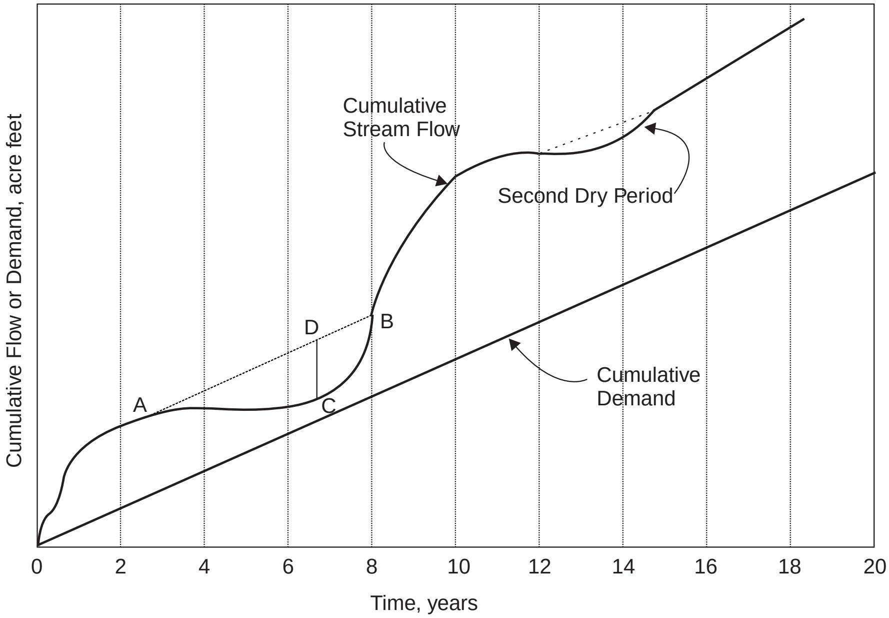


Fig. 8–3. Typical mass diagram

Image Not Available

Fig. 8–4. Mass curves of water utilization and weekly flow (From Davis, C. V., *Handbook of Hydraulics,* 5th ed. Copyright © 1973 by the McGraw-Hill Companies. Reproduced by permission of the McGraw-Hill Companies)[4]

January 31, 74,000 acre-feet (91.28 Mm^3). The reservoir was full again on May 4, when the two curves intersected.

The mass curve is plotted from the hydrograph of reservoir inflow. In most cases, this is taken as the hydrograph at the dam site.

Area–Volume Relationships

The shape of the reservoir site, the width of the valley floor, and the steepness of the adjoining hills will determine how much storage can be provided and how much reservoir surface area will be created for a given volume of storage. Through analysis of the contour lines on topographic maps, it is possible to develop the relationships between reservoir surface elevation, reservoir area, and volume of storage for a given site. These relationships are normally presented in the form of tables, but can also be presented graphically as in Figure 8–5. The arrangement shown in Figure 8–5, with two lines plotted in opposite directions, is used so that the curves do not overlap and can be easily read.

It is usually desirable that the reservoir have as little surface area as possible in relation to the volume of storage because land costs and natural evaporation losses will be proportioned to the surface acreage. Alternative sites can be compared by plotting their respective area-versus-capacity curves on the same graph and selecting the one that offers the most storage per unit of area.

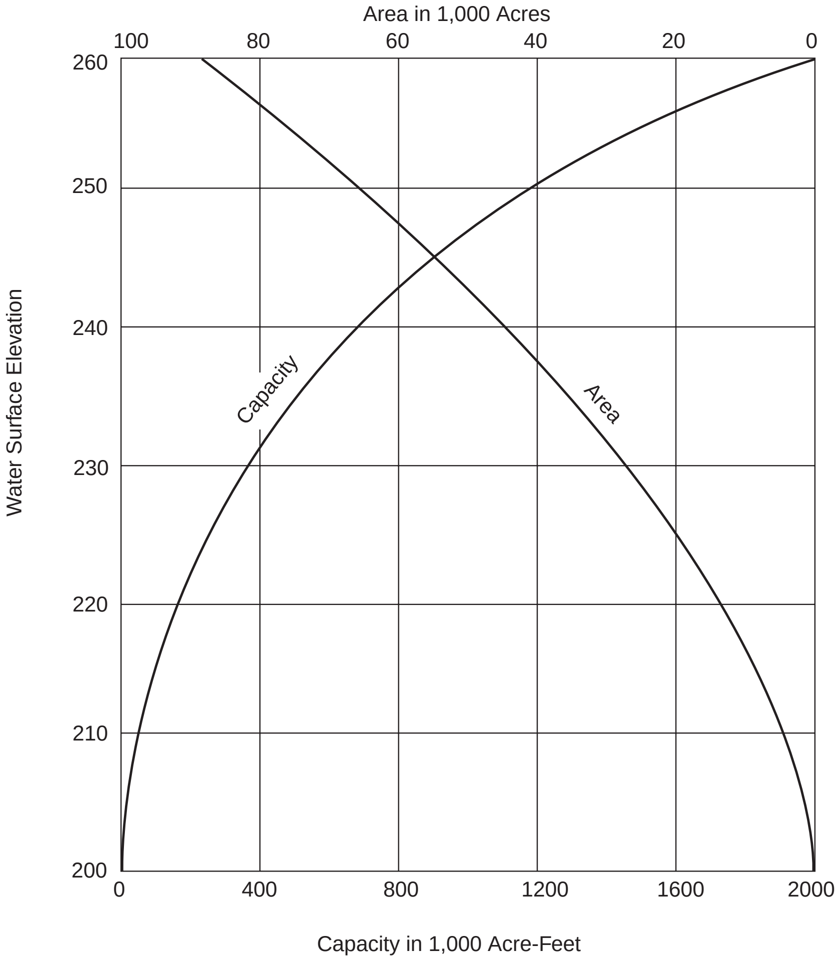


Fig. 8–5. Typical reservoir area and capacity curves

Site Selection

The selection of a suitable site for a reservoir depends upon a number of interrelated factors that establish the adequacy, economy, safety, and palatability of the supply. The surface topography should create a high ratio of water storage to dam volume. The topography should also present a favorable site for an adequate spillway to pass the flood flow and a suitable route for an aqueduct or pipeline.

The subsurface geology should provide useful materials for the construction of the dam and appurtenant structures, safe foundations for the dam and spillway, and tightness against seepage of the impounded waters beneath the dam and through its abutments.

The reservoir area that is to be flooded should be sparsely inhabited, not heavily wooded, and not traversed by important roads or railroads, pipelines, or other facilities. It should contain few wetlands. The area should constitute a reservoir of such shape as not to favor short-circuiting of the incoming waters to the intake, and of such depth, especially around its margins, as not to create large shallow areas. Purification of water by storage is an important asset of impounding reservoirs. Narrow reservoirs with their major axis in the direction of prevailing winds are especially subject to short-circuiting. Shallow areas often support a heavy growth of aquatic vegetation when they are submerged, and of land plants when they are uncovered by the lowering of the water surface. Decaying vegetation imparts odors and tastes to the water, supports algal growths, and liberates color.

The reservoir should interfere as little as possible with existing water rights, the intake should be as close as possible to the community it is to serve, and the development should preferably at such elevation as to supply its waters by gravity.

The character of the soil and rock of the drainage basin influences the kind and amount of mineral matter in the water. These are important considerations in the selection of a site source. A geologist, or aquatic biologist with knowledge of soils and their effect on the growth of taste- and odor-producing organisms, may need to be consulted on the selection of a reservoir site. Consideration should be given to seepage of poor-quality groundwater into proposed impoundment areas.

Precipitation, temperature, sunshine, evaporation, and air movements all influence the quality of surface waters and should be evaluated. For example, if heavy rains and floods, with their resulting turbidities, all occur during winter months, and summer rains are light, the water becomes clear. Algal growth will be much greater than if the conditions were reversed. The vertical circulation of water in the lake or reservoir also is related closely to climatic conditions and should be considered.

The extent of soil erosion on the drainage basin should be evaluated, as this will determine the quantity of sand and silt reaching the reservoir. Soil erosion also affects the biological productivity of a stream, lake, or reservoir, which in turn influences the quality of the water. Under severe soil erosion conditions, subsequent siltation may seriously limit the effective life of the reservoir as a source of water supply.

The uses of the land located on potential drainage basins should be investigated because they may affect the water quality. For example, reservoirs that are fed by water draining from highly cultivated farming areas often produce extensive algal growths, the products of which are difficult to remove. Fertilization for field crops results in the addition of nitrates and phosphates that may stimulate aquatic growths. Large algal blooms have been found in small reservoirs under these conditions and have created difficult water treatment problems. Pesticide use should be investigated and evaluated.

Peat bogs, mucky areas, swamps, and marshes on a drainage basin contribute a great amount of organic material to the waters draining from them, which may cause foul odors, undesirable tastes, acid conditions, and high color. These areas should be avoided, or reduced to a minimum by artificial drainage.

The extent and character of present and future recreational activities on the drainage basin should be known as they may affect the sanitary quality of the water. The health laws, rules, and regulations of the local health departments should be consulted on this matter.

Vegetation should be removed from the area to be flooded to prevent decay and subsequent impairment of the physical character of the stored water. Even then, the organic content and color of the stored water will likely increase for a period of 3 to 5 years. The ideal reservoir site is one where the slopes are relatively steep, thus ensuring deep water over most portions of the reservoir and minimizing the growth of algae, which are most prolific in shallow portions of reservoirs. The periodic exposure of shallow areas during periods of low water also leads to the growth of semiaquatic vegetation, which will die and decay when the reservoir fills again. Shallow areas, therefore, should be filled, or isolated by dikes when economically feasible.

RESERVOIR LOSSES

Water will be lost from a reservoir by bank storage, seepage, evaporation, and siltation, and overflows when the inflow exceeds the storage capacity of the reservoir. As the water level rises and falls, water will move into and out of the surrounding reservoir banks. Generally, as the reservoir level rises, water is lost into the surrounding soils.

Where a reservoir is underlain by porous strata that have ample outlets beneath the surrounding hills, or under the dam, seepage may amount to several hundred cubic feet per second (cfs). In the usual case, a maximum loss of 10 cfs (0.28 m^3/s) would be considered large.[4] The basin of a reservoir should be studied carefully, and if porous conditions are present, experts in geology and soil mechanisms should be called in to analyze potential seepage losses and recommend methods to reduce these losses.

Evaporation is a direct function of reservoir surface area and is usually expressed in inches. The rate of evaporation varies directly with temperature and wind velocity and inversely with humidity. Reservoirs at high elevations generally show lower rates of evaporation, due to lower temperatures. Typically, about 75 percent of the annual evaporation occurs in the 6 months from April to September, and about 20 percent occurs in the maximum month. The volume of water lost to evaporation (annual evaporation in inches × reservoir surface area) is offset to some degree by the gain of rainfall on the surface area of the reservoir (annual rainfall in inches × reservoir surface area). A negative value for evaporation-rainfall indicates a net loss; a positive value, a net gain of water.

Sedimentation (siltation) in the reservoir can cause a significant loss of volume over time. The sediments can come from erosion of the streambed itself or from erosion of the watershed.

In general, the problem is more serious in arid regions where the ground does not support a good vegetative cover. As a result, heavy rains cause excessive sheet erosion and a high percentage of silt in the streams. In humid climates, the effect of vegetative cover reduces erosion. The type of land use affects the siltation of reservoirs. One square mile of urban development has been reported to produce about the same amount of sediment as 100 square miles (259 km^2) of rural land.[5] Where reservoir capacity is small relative to annual stream flow, silting may be an important consideration.

Only in rare instances is the removal of silt from a reservoir by mechanical means justified economically. Once silt has been deposited, removal efforts have never been particularly effective. However, where large gates can be installed and reservoir operation permits the passing of floodwaters, a large portion of the silt can be passed through the reservoir.

Sometimes certain small areas in a watershed are major silt contributors. In such cases, there may be economic justification for special check dams and debris barriers. However, reduction of soil erosion is generally a long-range undertaking. Factors involved are: proper farming methods, such as contour plowing; terracing of hillsides; reforestation or afforestation; cultivation of permanent pastures; prevention of gully formation through construction of check dams or debris barriers, and revertment of stream banks.[6]

In the design of impounding reservoirs for silt-bearing streams, suitable allowance must be made for loss of capacity by silting. Understandably, deposition is most severe in reservoirs that are large in volume relative to inflow. The proportion of sediment retained is called its ‘‘trap efficiency.’’ Nearly 100 percent of the sediment transported by influent streams may be retained in reservoirs storing a full year’s tributary flow. Trap efficiency drops to a point between 65 and 85 percent when the storage ratio is reduced to 0.5 (half a year’s inflow) and to 30 to 60 percent when the storage ratio is lowered to 0.1 (5 weeks’ inflow). Silting is often fast when reservoirs are first placed in service and proceeds toward a steady state as time goes on. The typical unit weight of silt is 70 lb/cu ft (1,135 kg/m^3). Thus, 1,500 tons (1.361 metric tons) of silt will occupy about 1 acre-foot (1,234 m^3) of reservoir volume. A silt content of 250 mg/L is equivalent to 1 ton per million gallons (0.24 metric tons/m^3) of reservoir influent flow.

INTAKES

The purpose of the plant intake is to withdraw adequate quantities of the best available grade of raw water continuously. In selecting the intake location, the lake or river bottom character, currents, and potential sources of pollution must be considered. To provide for the variability of environmental influences, the intake structure should be designed and built to permit raw-water withdrawal at various levels, or locations, or both. The intake capacity, including pumping facilities, should provide sufficient raw water for the treatment plant at all times. The quantity of finished water in storage provides a buffer and is a factor in determining the necessary intake capacity. In reservoirs, intake capacity generally equals the average rate of demand on the maximum day. Dual facilities should be provided for mechanical equipment.

Intake facilities should be constructed to ensure continuous raw-water flow despite floods, icing, plugging with debris or sand, high winds, power failure, damage by boats, or any other occurrences. They should be inaccessible to trespass, contain adequate toilet facilities located and installed to prevent chance contamination of the raw-water supply, and contain an immediate warning system for the treatment plant operator in case of failure of automatic or semiautomatic pumping operations. Because many intakes will be constructed on permeable materials, the design of the structure must consider underflow and hydrostatic uplift pressure.

Intakes in large rivers should be located so that the ports are submerged at all stages of the river to a sufficient depth to avoid trouble with ice cakes or floating debris and to preclude the entraining of air. The ports should also be several feet above the bottom of the stream so that sand and gravel being transported on the bottom will not be drawn into the intake. In order to meet these requirements, it is usually necessary to locate the intake in the deepest part of the stream and away from the shore, particularly if the river is subject to large fluctuations in stage.

Intakes in small streams frequently require the construction of small diversion dams for the dual purpose of providing a sufficient depth of water at all flows to divert water into the intake port and a settling period in order to reduce the turbidity of the water. A small period of quiescent flow will also permit suspended leaves and wood either to rise to the surface or to sink if they have become waterlogged, and it will favor the formation of sheet ice in cold weather and thus reduce the difficulties of ice.

Both bar racks and mesh screens are frequently used on the openings into the intake structure. Bar racks with spacings of 2 to 4 inches (51 to 101 mm) protect the intake from large floating objects.

Screens are used to protect against floating materials such as leaves. They should have not less than 2 and sometimes have as many as 8 meshes to the inch (79 to 315 per meter), depending upon the character of the floating matter in the water. Screens should be of corrosion-resistant metal and easily removable. Screens should have a velocity of not more than 3½ inches/sec (8.89 mm/s). Low velocities and small openings are necessary to prevent the entrance of fish.

In cold climates, ice troubles are reduced in frequency and intensity if intake ports lie as much as 25 feet (7.62 m) below the water surface and entrance velocities are less than 3 to 4 inches/sec (75 to 100 mm/s). At such low velocities, frazil ice, leaves, and debris are not entrained in the flowing water, and fish are able to escape from the intake current. Also, the use of fiberglass-reinforced plastic (RFP) for intake bells has been reported to be effective in reducing frazil ice adherence.[7]

Bottom sediments may be kept out of intakes by raising entrance ports 4 to 6 feet (1.22 to 1.832 m) above the lake or reservoir floor. Ports controlled at numerous depths permit water-quality selection and optimization. A vertical interval of 15 feet (4.57 m) is common.[8] Submerged gratings are given openings of 2 to 3 inches (51 to 76.2 mm). Specifications for screens commonly call for 2 to 8 meshes to the inch (79 to 315 per meter) and face (approach) velocities of 3 or 4 inches/sec (76.2 to 102 mm/s). Typical intakes are shown in Figures 8–6 and 8–7.

Impacts of Zebra Mussels

The zebra mussel, *Dreissina polymorpha,* is regarded as the most potentially damaging natural intrusion into the United States' water distribution systems in years.[9,10] Believed to have been introduced to the Great Lakes in 1985 or 1986, they have now spread throughout the central states as shown in Figure 8–8. This fast growing, freshwater mollusk can rapidly clog submerged intakes to water supplies, as shown in Figure 8–9. Water intakes provide a hospitable environment for the mussels because of the continuous flow of water that contain algae, phytoplankton, and other organisms that represent the food chain for mussels. The water also provides a continuous source of veligers, the free-swimming early-life form of the mussels that settle out of the water column and begin to attach to substrates. Among the requirements needed to support a viable population of zebra mussels are: freshwater, a satisfactory temperature regime, adequate pH, sufficient calcium levels, suitable substrate, suitable flow conditions and an adequate food supply. Of these, temperature, calcium, and pH may be limiting in some areas in the eastern United States.

Zebra mussels are small, often less than 1 inch in length, and can colonize virtually any hard, nontoxic surface. All surfaces contacted by raw water up to the point of primary disinfection or sand filtration are at risk. Structures typically fouled in water treatment facilities include the intake crib, trashracks, the main intake line, the screen wells, traveling screens, and any strainers and metering equipment that may exist.[11] They also attach to other zebra mussels creating dense colonies that may be 0.5 meters thick.

The primary impact is reduction of flow. Even before colonization is enough to appreciably reduce the inside pipe diameter, the increased roughness coefficient can reduce flow. Complete, immediate blockages can result from the detachment of mussel clumps that travel down the piping system until they reach a restriction they cannot pass through.

Another indirect impact of zebra mussels is increased microbially induced corrosion of metal pipes that results from the anaerobic conditions formed at the byssal attachment. Zebra mussels can also increase taste and odor problems.

Methods for controlling zebra mussels are discussed in Chapter 16, ‘‘Oxidation.’’

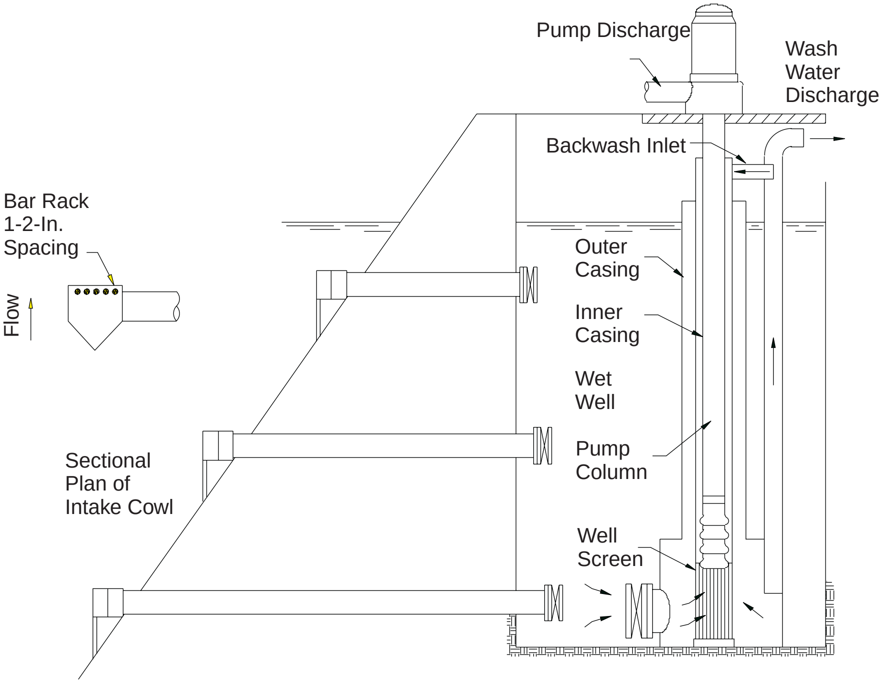


Fig. 8–6. Intake with vertical pump and backwashed well-type screen (From Fair, Gordon M., Geyer, John C., and Okun, Daniel A., *Water and Wastewater Engineering,* Vol. 1. Copyright © 1966 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

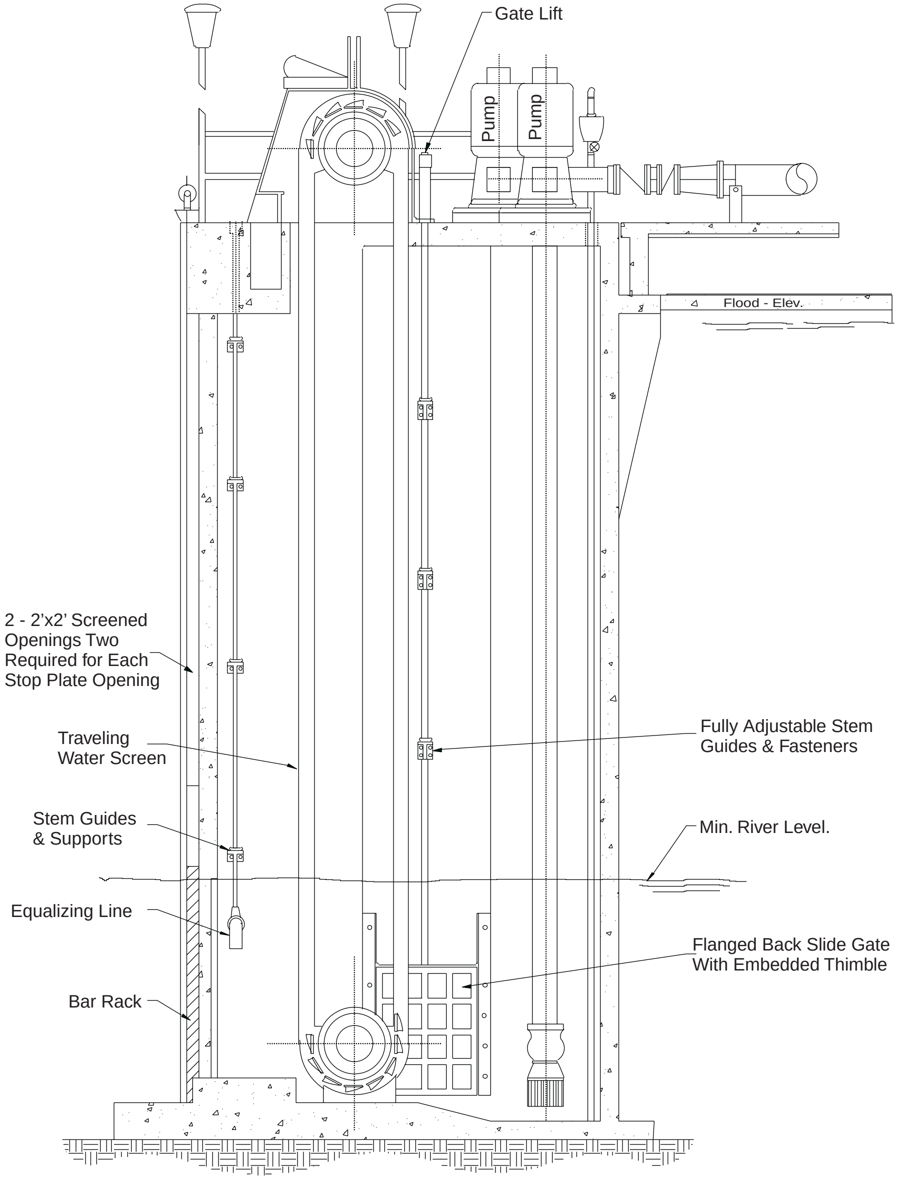


Fig. 8–7. Intake with vertical pump and traveling water screen (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

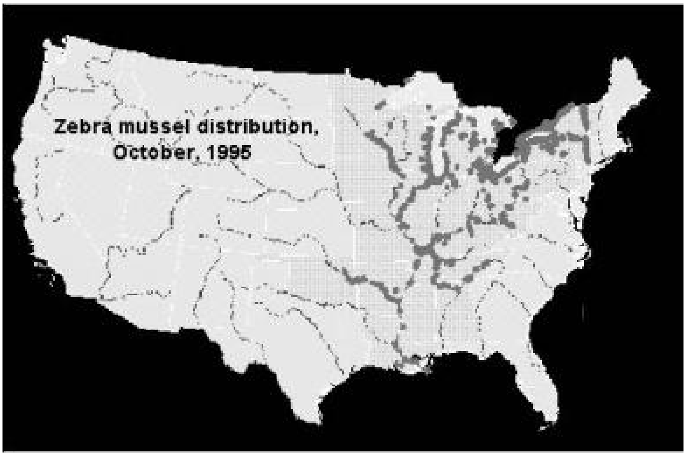


Fig. 8–8. Zebra mussel occurrence (National Aquatic Nuisance Species Clearinghouse, New York Sea Grant, State University College, Brockport, NY)

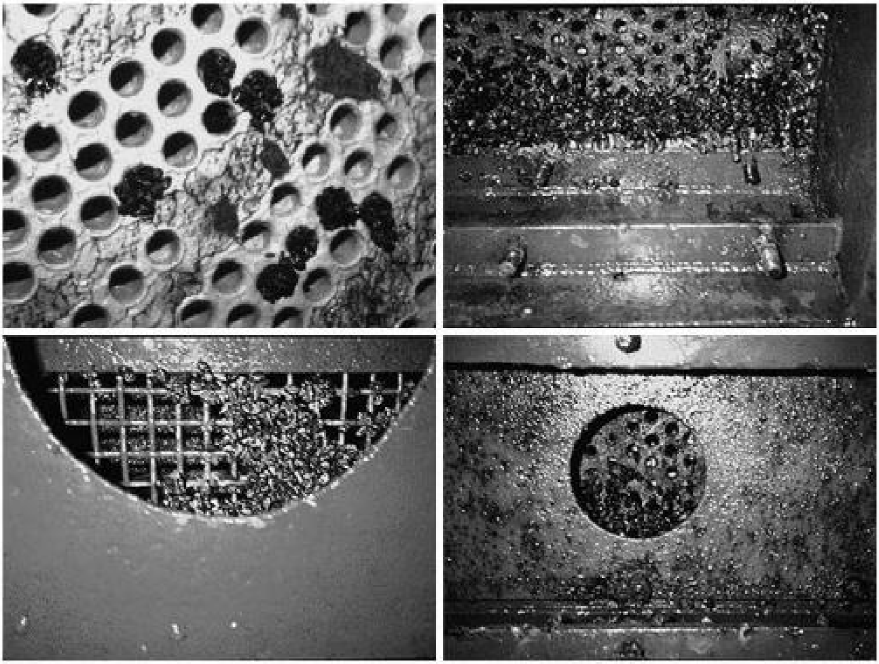


Fig. 8–9. Zebra mussels in intakes (*Source:* U.S. Army Corps of Engineers)

CONTAMINANTS IN SUPPLY SOURCES

A common practice is for one community to discharge its treated wastewater into a stream or lake ultimately used by another community as a water supply source. One study estimated that 1 gallon (3.785 L) out of every 30 gallons (100 L) used for water supply had passed through the wastewater system of an upstream community, based on 155 cities studied.[12] Another showed that 90 percent of 1,246 municipal water supplies studied contained wastewaters.[13] Several utilities were forced to use water from a source when low flow was less than the combined upstream wastewater discharge flows. Water supplies drawn near the bottom of large river basins were found to contain wastewater from several thousand dischargers. About 15 million people were estimated to be served by supplies containing at least 10 percent wastewater at low flow conditions, and 4 million were served by supplies containing 100 percent wastewater. In addition to municipal wastewaters, many supplies contain contaminants from industrial wastewaters, agricultural runoff, and urban stormwater.

The practice of using water supplies containing these discharges has resulted in extensive, although often unintentional and unplanned, water reuse. Many municipalities have no alternative but to use such supplies. Careful consideration of discharge-related constituents in water supplies and their fate in receiving streams can contribute to choosing intake locations that will improve supply quality.

Types of Concern

Contaminant concerns fall into four main categories: toxicological, microbiological, aesthetics, and possible chronic effects of trace organics.

1. Toxicological (pesticides, residues, mine drainage, accidental spills of chemicals in transport; municipal, industrial, and agricultural wastewaters; urban and rural runoff). Many surface waters throughout the country are subject to contamination from accidental spills of toxic materials in transport, and some are so contaminated from urban and agricultural runoff and other nonpoint pollution sources that their health risks are only slightly different from those involved in the use of river water receiving treated municipal wastewater. Increasing the distance points of discharge and an intake provides more time for detection of toxic materials that have not been removed in treatment. The removal of toxic materials by stream self-purification is not so important or effective as that in water or wastewater treatment. Advanced wastewater treatment (AWT) processes can remove nearly all the dissolved and suspended contaminants in wastewater, and can provide a high degree of protection against possible toxic contaminants.

2. Microbiological (bacteria, viruses, fungi, and other organisms from human waste). Agricultural runoff from pastured areas or feedlots can be a concentrated source of pathogenic protozoa. Increasing the distance between such sources or a wastewater outfall and water intake provides more time for natural die-off of the pathogens. However, in cases of limited distance between a wastewater outfall and intake, any necessary removal up to complete disinfection can be obtained prior to effluent discharge. The minimum separation required then depends upon the degree and reliability of pathogen removal provided by the wastewater prior to discharge. Because microbiological tests require several days for completion, time of travel or storage in the stream is not a monitoring advantage unless 4 days or more are available.

3. Aesthetics (acceptance, taste and odor, and color). Water from streams that do not receive treated wastewater is usually more aesthetically pleasing than water obtained from streams receiving wastewater. Increasing the separation between outfall and intake, up to a point at least, probably provides greater public acceptance of the water. Physical problems of taste, odor, and color may arise from the discharge of wastewaters to water sources, although these problems can be avoided if proper water and wastewater treatment are provided.

4. Possible Chronic Effects of Trace Organics (carcinogens, mutagens, tetratogens). Many water supply sources contain trace organics. Either unreacted or in the form of their reaction products with chlorine, ozone, or other oxidants, these substances may have adverse health effects following their long-term ingestion in trace amounts. These organics may originate in nature or be present in wastewater, which also may contain synthetic organics not found in nature, and they are discussed in Chapter 3, "Organic Compounds in Drinking Water." The degree of hazard probably is not affected to any practical extent by the time of stream travel or storage. Removal of organics by AWT or upgraded water treatment is more important than separation distance between outfall and intake.

Organics in Wastewaters

The organic material in municipal wastewater is a mixture of many compounds that are only partially known. The three broad classes of organics in municipal wastewater are fats, carbohydrates, and proteins. They are usually considered removable by primary and secondary biological treatment, although protein is somewhat less readily removed than fats and carbohydrates. A properly operating biological treatment plant treating residential wastewater is capable of producing secondary effluent with a soluble COD of 30 to 50 mg/L and a soluble biochemical oxygen demand (BOD) of 1 to 2 mg/L. Data on TOC levels in wastewater indicate that secondary effluent concentrations are in the 30 to 50 mg/L range. The results of an extensive study including activated sludge effluents are summarized in Tables 8–1 and 8–2.[14] Another study found 60 percent of the organics in secondary effluent had molecular weights less than 700, and 25 percent had apparent molecular weights greater than 5,000.[15]

In one study of municipal wastewater, 77 organic compounds were detected in the primary effluent, and 38 in the secondary effluent. Several compounds found in the secondary effluent were not present in the primary effluent.[16] The concentrations of individual compounds in the secondary effluent were estimated to be less than 20 μg/l. It was found in another study that soluble organics are produced in biological treatment that are more refractory to further treatments than are the organics in raw sewage.[17] It was hypothesized by these same investigators that the residual organics not removed by activated carbon are intermediate breakdown products of protein, and that these are most likely proteins that originate from the cell walls of microorganisms present in biological treatment processes. The fate of organics during chlorination is of particular concern, and is discussed in Chapter 3.

In addition to the presence of organic compounds contributed directly by wastewaters, there is also the potential that inorganic nutrients, such as nitrogen and phosphorus, can contribute to organics of concern by stimulating algal growths. There is evidence that chlorophyll, algal biomass, and algal extracellular products can serve as

TABLE 8–1. Distribution of Organic Groupings

	Percent of Total COD		
Organic Grouping and Fractions	Municipal Wastewater High Rate Trickling Filter	Municipal Wastewater Stabilization Pond	Domestic Wastewater Extended Aeration Activated Sludge
Proteins	21.6	21.1	23.1
Carbohydrates	5.9	7.8	4.6
Tannis and lignins	1.3	2.1	1.0
Anionic detergents	16.6	12.2	16.0
Ether extractables	13.4	11.9	16.3
Fulvic acid	25.4	26.6	24.0
Humic acid	12.5	14.7	6.1
Hymathomelanic acid	7.7	6.7	4.8

(Reprinted with permission from Manka, J., et al., ''Characterization of Organics in Secondary Effluents,'' *Environmental Science and Technology*, 8:1017, November 1974. Copyright 1974 American Chemical Society)[14]

trihalomethane precursors.[18] Both green algae and blue-green algae produce extracellular products that are found upon chlorination to yield at least as much chloroform per unit of organic carbon as has been reported for studies of humic and fulvic acids.[15]

Pathogenic Organisms in Wastewater

Domestic sewage contains agents of human disease. Thus, the location of drinking water intakes downstream from the discharge of wastewater, treated or untreated, is of concern to health authorities because of the potential for disease transmission.

In considering the transmission of disease, one must take the following factors into account:

TABLE 8–2. Molecular Weight Distribution of Humic Substances in Municipal Wastewater

	Percent of Humic Compound Present		
Molecular Weight Range	Fulvic Acid	Humic Acid	Hymathome Acid
<500	27.5	17.9	4.5
500–1,000	7.8	6.2	12.2
1,000–5,000	35.7	29.4	48.0
5,000–10,000	15.3	7.8	28.0
10,000–50,000	9.4	36.7	7.3
>50,000	4.3	2.0	0

(Reprinted with permission from Manka, J., et al., ''Characterization of Organics in Secondary Effluents,'' *Environmental Science and Technology*, 8:1017, November 1974. Copyright 1974 American Chemical Society)[14]

- The presence of agents of disease
- The concentration of the agent or the dose
- The dose response
- The host contact

A number of bacterial diseases have been associated with the consumption of sewage contaminated water, including typhoid fever, salmonellosis, shigellosis, cholera, and infections due to enterocytopathic *Escherichia coli* and *Yersina entercolitica.* During the period 1969–74, the majority of waterborne outbreaks of bacterial disease in the United States were due to *Shigella* sp., followed by *Salmonella* sp., typhoid fever, and pathogenic *Escherichia coli.* Less than 10 percent of the cases of these diseases in the United States were waterborne.[19]

The waterborne parasitical diseases commonly associated with contaminated water are amoebic dysentery, *Cryptosporidiasi*s and *Giardiasis,* both protozoan diseases.

There are at least 101 types of viruses that may find their way into water via fecal contamination. Of these, the most serious threat to the public health (in terms of disease severity) is the virus of infectious hepatitis (hepatitis A). Less than 1 percent of the reported cases of hepatitis in the United States are attributable to contaminated drinking water. A large proportion of cases were associated with the consumption of contaminated shellfish.

Infectious disease agents have traditionally been monitored in water using a surrogate parameter, the coliform or fecal coliform test. The presence of these bacteria in water is indicative of the presence of fecal material and thus the potential presence of pathogenic enteric organisms. Through the years this test has been effectively used in the quality control of finished drinking water. However, the absence of the latter organisms in finished drinking water may not guarantee the absence of enteric viruses, *Giardia* or *Cryptosporidium.*

Fate of Contaminants in Streams

There are many factors that affect contaminants in streams. Contaminants may be contributed to streams from a number of sources, including:

- Soil and rock
- Waste discharges
- Storm runoff
- Precipitation and atmospheric fallout
- Biological organisms

Flowing streams are dynamic systems, whose chemical and biochemical reaction rates are such that equilibrium conditions are only slowly, if ever, attained. The organic and inorganic chemicals of concern in water supply interact with one another and with other materials and organisms in various chemical and biochemical reactions. Reactions that can take place in streams to decrease, or in some cases increase, the concentration of contaminants include:

- Precipitation
- Complexation

- Oxidation-reduction
- Ion-exchange
- Adsorption and/or absorption, often termed *sorption*
- Flocculation
- Biological uptake and/or release

Physical Factors The important stream physical characteristics include: velocity, depth, turbulence, degree of mixing, temperature, turbidity, changes in cross-section, and such bottom characteristics as slope, type of material, and sorptive capacity. The effects of some of these parameters are so interrelated that their relative importance is difficult to determine. For example, the bottom slope affects the depth and the velocity of the stream, and turbulence is affected by all three characteristics.

The degree of mixing in a stream is an important parameter, and is determined by the physical characteristics. Mixing in a stream can occur vertically, laterally, and/or longitudinally. Vertical mixing normally occurs within a few tenths of a mile. Density gradients from temperature differences in the stream tend to overcome vertical mixing, but this rarely occurs. Lateral mixing normally is complete within miles and increases with the number of relatively sharp reverse bends in the stream reach. Longitudinal mixing, caused by variations in cross section and changes in direction that permit areas of quiet water and eddy currents, may require many miles. Typically, the ratio of longitudinal-to-lateral-to-vertical mixing is about 100:10:1.

Other parameters determined by the physical characteristics of the stream are the re-aeration rate and solids deposit. The re-aeration rate is a function of the turbulence of the river and can be determined from the velocity, depth, and temperature of the stream; it increases with increasing velocities and decreases with increasing temperature. Sedimentation of suspended particles is dependent on the degree of turbulence in the stream; settled solids can be scoured off the bottom and resuspended during periods of increased flow. This raises the stream turbidity and increases oxygen demand.

Precipitation and Complexation In general, all the constituents of a natural chemical system, including trace metals, ions, and inorganic and organic compounds, are related to each other through complex formation and solid precipitation reactions. Variation of any constituent will give rise to variations in at least some of the other constituents. The organic component of fresh natural water is poorly known analytically, and the complexing properties of organics are only hypothesized in many cases.

The complexation of metal ions with either naturally occurring organic material or organics in waste discharges may be important reactions. Complex formation is the combination of metal cations with molecules or anions to form a coordination or complex compound. There is some indirect evidence that complexing agents play a significant role in the form of metal concentration in treated wastewater. It is possible that pollutant complexing agents in flowing streams play an important part in transporting heavy metals and preventing their removal by conventional water treatment.

Many of the metal ions found in natural waters, particularly those found at trace levels, form strong complexes with a variety of chemical species. The formation of complex compounds may have several effects:

- Formation of insoluble compounds may remove metal ions from solution
- Complexation also may solubilize metal ions from otherwise insoluble metal compounds
- Strong complexation may shift oxidation-reduction potentials

Little has been reported about levels of complexing agents and stable metal complexes in natural waters. Complexing agents are not normally determined in water analyses.

Humic and fulvic acids probably are the most important naturally occurring complexing agents. These acids are rather loosely defined; they refer to a family of compounds, similar in structure and chemical properties, formed during the decomposition of vegetation. They can strongly bind metal ions, and they are found in both water and soil. Synthetic complexing agents, such as sodium tripolyphosphate, sodium ethylenediaminetetraacetate (EDTA), sodium citrate, and sodium nitrilotriacetate (NTA), are produced in large quantities and almost certainly find their way into streams through waste discharges. NTA may also solubilize heavy metals from sediments on stream bottoms, depending on pH, bicarbonate concentration, calcium concentration, and the nature of the sediments.

Effects of Clay Minerals and Other Suspended Material Clay minerals are among the most common suspended matter found in natural waters. In many streams, clays might be considered the most important mineral solids present in colloidal suspension or as sediments, for these reasons:

- Clay minerals can fix dissolved chemicals in water and thus exert a purifying action. The ability of clays to exchange cations is an important phenomenon having an impact on the availability of trace-level metal nutrients in water.
- Because of their high surface area and other properties, clays also may sorb organic compounds, such as pesticides and herbicides, and are important in the transport and removal of organic pollutants in streams.

It is also believed that some microbiological processes occur at clay mineral surfaces, so that clays may participate in the degradation of organic materials.

FACTORS IN THE LOCATION OF WATER INTAKES

There are a great many factors to be considered regarding intake locations. Raw-water quality may vary greatly from stream to stream; it is assumed that a preliminary selection among various streams has been made on the basis of best available raw-water quality from a public health standpoint, as well as adequate quantity. The problem then becomes one of locating the intake along a given stream.

For the moment, the question of upstream wastewater discharge is set aside, and the other factors are considered. This is done in order to help place the potential hazards of wastewater contaminants in perspective with the risks involved in all water systems, even when there is no pollution from wastewater.

Important items in the location of intakes, which take into account reliability, safety, and cost, include:

- Adequacy of supply
- Channel changes, shoal and bar formation, and silting
- Availability of water to intake ports at all river stages and at all stream flows
- Accessibility to intake for maintenance at all river stages and at all seasons
- Location of the intake with respect to the city to be served
- Navigation requirements
- 100-year flood level
- Need for storage dam, either in-channel or off-channel, and detention provided
- Foundation conditions
- Structural stability and safety of dams
- Protection from rapid currents, wind, ice, boats, floating material, waves, and bottom sediment
- Water-depth, and ability to draw water from different depths
- Distance from service roads and a source of electric power
- Protection from vandalism

Even in the absence of upstream discharges of municipal wastewater, there are public health factors to be considered in intake location. Storm runoff that makes up stream flow may contain almost any of the contaminants included in drinking water standards. The concentrations of these substances can be restricted by proper location and operation of intakes. The protection of raw water supplies is discussed elsewhere in this chapter under ‘‘Watershed Management.’’

In making case-by-case judgments of the minimum distance between the points of wastewater discharge and water supply intake, some of the items to be considered include:

- Stream flow and quality
- Quantity and quality of treated wastewater to be discharged
- Potential water quality improvement by stream purification processes, including dilution, reaeration, adsorption, sedimentation, and biological die-off
- Pollution from sources other than municipal wastewater, including industries, storm runoff, agriculture, and miscellaneous nonpoint sources
- Raw water quality at intake under most adverse conditions
- Water treatment provided
- Relationship between wastewater effluent characteristics and safe drinking water requirements
- Risk assessment of intake siting options

PRETREATMENT OF SURFACE WATER SUPPLIES

Surface water supplies, whether from rivers or reservoirs, often contain impurities or characteristics that may best be removed prior to conventional water treatment. Reservoirs contain algae and other microscopic organisms that can cause taste and odor problems. Anaerobic conditions may develop near the bottom of reservoirs causing solubilization of objectionable metals. Rivers may carry heavy silt loads that would overtax or complicate operation of coagulation or softening unit processes. Debris in either reservoirs or rivers may foul water transport equipment. Pretreatment provisions may be considered for:

- Screening of surface water for removal of general debris
- Treatment of reservoirs for destratification
- Treatment of reservoirs for control of algae and plant growths
- Pretreatment of surface water for control of taste and odors
- Presedimentation of river waters for removal of suspended silt load

Screening

In rivers or lakes where the water transports debris, such as leaves, branches, logs, and similar objects, it is necessary to provide screening facilities before water is withdrawn for treatment. When substantial amounts of debris are present, mechanically cleaned screens should be considered. Most water intakes include protective screens, or bar racks, that are not mechanically cleaned and remove only the largest pieces of debris.

Screens should be located at the intake structure. Typically, the screenings, whether manually or mechanically removed, will be returned to the water source in such a way that they will not return to the screen and accumulate.

Traveling water screens are operated intermittently unless the debris is particularly heavy. These screens are installed in a channel as shown in Figure 8–10, where they move in a slot, upward to the surface. Water sprays clean the screens and sluice the debris to a channel or pipe that carries it back to the river or lake. Screen openings are normally ⅛ to ½ inch (3.18 to 12.7 mm) wide.

Protective bar racks are often placed upstream of traveling water screens as barriers to logs, large fish, and other similar objects that could cause damage or are not suitable for removal by the screen. These bars may be spaces at 3 to 4 inches (75 to 100 mm),

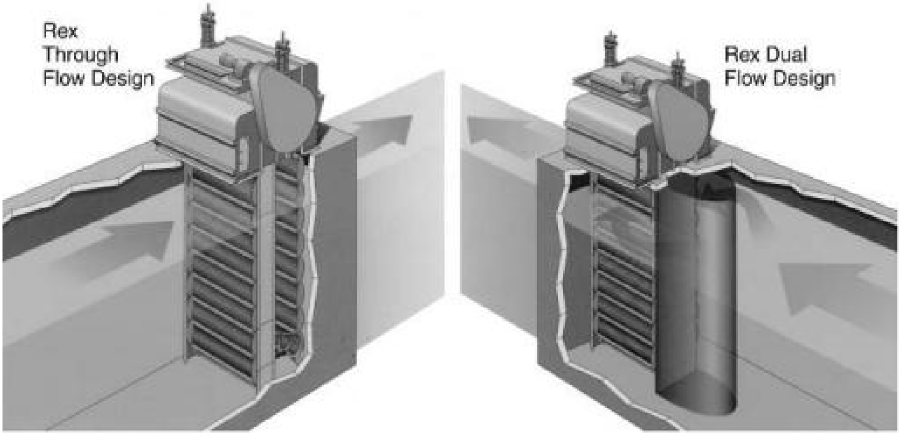


Fig. 8–10. Traveling water screen (Courtesy of USFilter/Johnson Screens)

and inlet velocities are less than 0.25 ft/sec (75 mm/s). They may be fixed in place with no provision for cleaning because they are not intended to collect material, but are merely to prevent entry of large material. Where they will collect debris, provision should be made to allow hoisting them to the surface to allow its removal.

Microscreens, shown in Figure 8–11 are also used as a pretreatment process, to remove finer material, such as filamentous algae. Screening media are generally stainless steel or polyester. Media openings can be as small as I micron but are typically 20 to 30 microns. Microscreens typically are located at the treatment plant, and are used only when the water source is very turbid. Figure 8–12 shows a typical installation.

Destratification of Reservoirs

Thermal stratification in lakes and reservoirs is a seasonal phenomenon. In the spring as the weather warms, heat is transferred to the surface of lakes and reservoirs. As the water is warmed, its density decreases, and the warmer, lighter water rises to the top of the reservoir and floats on a heavier cooler mass of water, causing two distinct water layers. The upper zone, or *epilimnion,* is characteristically well mixed and aerobic. It is separated from the lower zone, or *hypolimnion,* by a transition zone known as the *thermocline,* where the temperature of the water changes rapidly with depth between the two zones. The hypolimnion is characteristically cool and unmixed (Figure 8–13) a schematic diagram of a reservoir showing these two zones.

As the spring weather begins, the surface water warms and the stratification of the reservoir is initiated. The wind action on the water causes a zone of mixing, and a layer of water at the surface increases uniformly in temperature. The stability of the thermocline, or temperature difference, between the epilimnion and hypolimnion increases as the average air temperature increases.

The thermocline may initially be 6 to 10 feet (1.8 to 3.0 m) below the surface of the water. If wind velocity increases, the mixing of the epilimnion increases, and the thermocline is found farther below the surface.

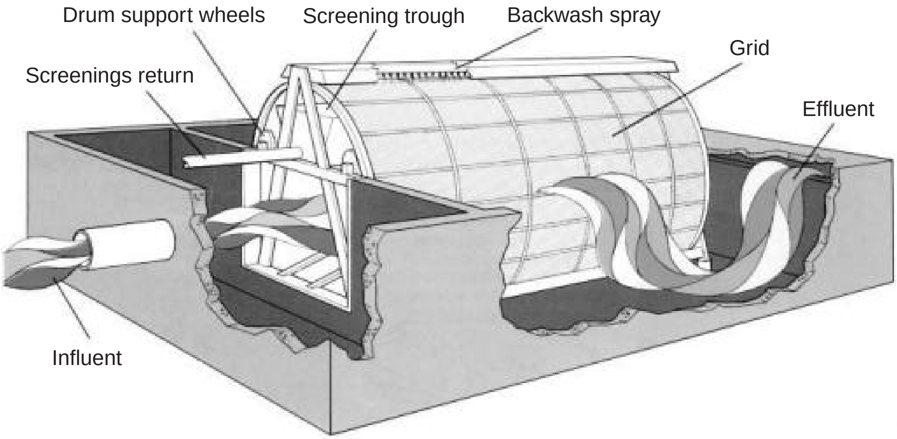


Fig. 8–11. Envirex microscreen diagram (Courtesy of USFilter-Rex and LinkBelt Intake Products)

Fig. 8–12. Microscreen installation (Courtesy of USFilter-Rex and LinkBelt Intake Products)

The hypolimnion is isolated from a source of oxygen. In aged reservoirs, or where a reservoir has an input of organic materials, the demand for oxygen may cause a total depletion of oxygen in all or part of the hypolimnion. The lack of oxygen may cause anaerobic conditions, and chemical constituents in the hypolimnion and benthal deposits will be chemically reduced. Hydrogen sulfide may be formed, and iron and manganese dissolved. Hydrogen sulfide imparts taste and odor to the water, and iron and manganese form precipitates (when water is subsequently aerated) and cause the water to be turbid and colored. Iron and manganese will also stain plumbing fixtures and laundry, and if present in sufficiently large concentrations, will cause taste problems.

In addition, anaerobic conditions in the hypolimnion may cause reduction in pH and an increased concentration of CO_2, which in turn may impair the capability of the treatment facilities.

The problem can be resolved by not drawing water from the hypolimnion, adding more intensive water treatment, or eliminating the problem in the reservoir. In some cases, it may be feasible to eliminate thermal stratification so as to oxygenate the hypolimnion, or to add oxygen to the hypolimnion without thermal destratification. These approaches may not be practical for very large reservoirs.

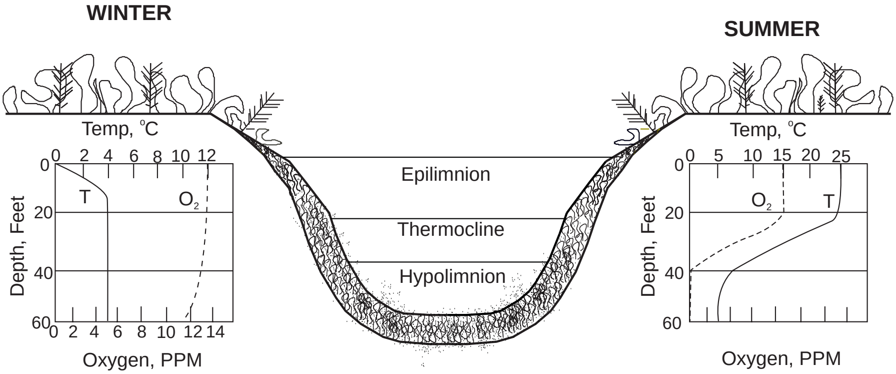


Fig. 8–13. Distribution of oxygen and temperature with lake depth (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

To destratify the reservoir, the cooler, more dense water from the hypolimnion is raised to mix with the less dense, warmer water of the epilimnion. If sufficient energy is added, complete dispersion of the thermocline occurs. If the oxygen resources and natural reservoir reaeration are sufficient, an aerobic reservoir may be maintained.

There are potential problems associated with destratification. In an advanced eutrophic reservoir, the accumulation of sufficiently high organic content in the hypolimnion may, if mixed with the epilimnion, cause insufficient dissolved oxygen throughout the reservoir. In such a case, thermal destratification could do more harm than good.

In eutrophic reservoirs, dissolved nutrients in the hypolimnion, if mixed into the epilimnion, may stimulate aquatic growths, which can trigger taste and odor problems in the water supply. Also, mixing of water from the cool hypolimnion water with that of the warm epilimnion will result in an increase in water temperature at a hypolimnitic intake. The warmer water may not be desirable for a water supply.

Most of these problems appear to be of consequence only in small lakes. The studies done with thermal destratification in large reservoirs have not shown evidence of either oxygen depletion or increased phosphorus concentration in the epilimnion. Methods to aerate the hypolimnion without mixing it with epilimnion have been developed by several investigations.[20,21] The most common method is to install a tube the full depth of the reservoir with the upper lip of the tube above the reservoir water surface. Air is injected near the bottom of the tube, and an airlift effect is produced, raising the water in the tube and concurrently aerating the water in transit. A slot or opening is provided in the tube below the thermocline to provide a means for the water to escape before it reaches the epilimnion, and the air continues up the tube to the atmosphere.

The two methods for in-reservoir oxygenation are:

- Thermal destratification to co-mix the epilimnion and hypolimnion to provide a uniform dissolved oxygen concentration and temperature,
- Artificial oxygenation and mixing of the hypolimnion to maintain a layered reservoir.

The energy levels required to accomplish thermal destratification have been calculated by Symons, et al., who related energy efficiency of destratification to ‘‘oxygenation capacity’’ (OC) and ‘‘destratification efficiency’’ (DE).[22]

The oxygenation capacity is determined by calculating the mass input of dissolved oxygen per unit of time divided by the energy input per unit of time. The dissolved oxygen input per unit of time is the summation of increased concentration in dissolved oxygen per unit of time and the steady-state demand rate for oxygen per unit of time. In reservoirs, the demand rate for oxygen is low and often ignored as insignificant. The OC is expressed in units of pounds of oxygen transferred per kilowatt-hour.

The DE is calculated by the net change of ‘‘stability’’ over a set time frame divided by the total energy input over the same time frame. Stability is defined as the minimum energy needed to mix the lake, and is calculated by multiplying the weight of water in the lake by the vertical distance between the center of gravity of the lake, taking into account the density due to the thermal gradient. The results of this calculation are in foot-pounds, which can be converted to kilowatt-hours over the time frame used. The change in stability divided by the power input yields the DE.

As the depth of the epilimnion increases and/or as the temperature differential between the epilimnion and hypolimnion increases, the location of the center of gravity in the reservoir moves downward. The farther the center of gravity moves from the isothermal center of gravity, the more stable is the stratification, and the more energy is required to overcome the stratification.

The use of either the OC or DE in design of lake destratification is precarious without prior experience. Table 8–3 shows experienced power requirements and time required to acquire destratification, using pumps and diffused air mechanical devices.

CHEMICAL TREATMENT OF RESERVOIRS

When sufficient nutrients are available and warm, sunny conditions prevail, algal growths occur in reservoirs, and can cause taste and odor problems. Copper sulfate and/or potassium permanganate is used as an algicide in reservoirs to reduce the number of organisms. Chlorine has also been used in some situations, but its usage must be carefully controlled because it combines with certain odor-forming compounds to cause a more intense odor and may result in trihalomethane production.

Copper sulfate is considered to be effective in controlling algal growths. There is some difference of opinion over whether copper sulfate is algistatic or algicidal; however, continuing programs in which copper sulfate is added regularly to keep growths from occurring are more effective than programs depending on copper sulfate to destroy an existing growth. In a typical situation, copper sulfate would be added to a reservoir in a concentration of about 0.1 to 0.5 mg/L. Many times it is only necessary to add the copper sulfate around the shoreline where the water is quiescent and the algal growths are heaviest. The copper will eventually precipitate from solution. The alkalinity of the water is important in determining the dosage. If the methyl orange alkalinity is less than 50 mg/L, copper sulfate has been shown to be effective at an application rate of 0.33 mg/L. If the methyl orange alkalinity is greater than 50 mg/L, the dosage rate will be 5.4 pounds/acre (606 kg/Mm^2). Depth is not a factor in high alkalinity water because precipitation of copper will occur rapidly.

Care must be exercised to prevent overdosing of copper sulfate and killing fish. Safe dosages for most fish are about 0.5 mg/L, but for trout the safe dosage is 0.14 mg/L.

TABLE 8–3. Experienced Power Requirements for Destratification

Lake	Volume (acre feet)	Surface Area (acres)	Average Depth (ft)	Mechanical Mixing Device	Time Rq'd for Destratification (hr)	Power (HP)
Steward Hollow	120	8	15	Pump	37.5	12
Caldwell	100	10	10	Pump	8	12
Vesuvius	1,260	105	12	Pump	208	12
Pine	98	14	7	Pump	35	12
Indian Brook	316	18	—	Diffused Air	168	8
Wohl Ford	2,510	130	—	Diffused Air	78	50
Blelham Tarn	575	27	—	Diffused Air	335	1

Source: Adapted from References 20 and 23.
Conversion factors may be found in Appendix A.

Potassium permanganate has proved to be algicidal at dosages of approximately 0.5 to 2.0 mg/L, but it is used to a lesser extent than copper sulfate in controlling algal growths in reservoirs.[24] The higher dosages and cost of potassium permanganate have restricted its application for reservoir algal control.

If blue-green algae are predominant—a condition associated with low oxygen conditions—alternative control methods may be advantageous. The addition of aeration or destratification techniques may prove to be simpler and more direct than use of an algicide. However, blue-greens are more susceptible to copper sulfate than green algae.

Presedimentation

Presedimentation is a pretreatment process for control of silt load on subsequent treatment units. It is unnecessary to use presedimentation for reservoir supplies because the reservoir accumulates the silt. The silt load to the plant should be minimal, if an effective intake structure is provided. Therefore, presedimentation is a unit process almost solely associated with river supplies, and, ore specifically, only for those river supplies that carry heavy silt loads.

Presedimentation basins require additional capital expenditures that are not justifiable at all plants. The silt load on most rivers is controlled by upstream reservoirs, and only during high river flow rates does the silt load become a problem. If presedimentation is not provided, the heavy turbidities will require increased chemical dosages. If the high turbidities are associated with short-duration, seasonal periods, it may not be justifiable to provide presedimentation.

Silt settles rapidly, and surface overflow rates of 1,600 to 2,000 gpd/sq ft (2.71 to 3.39 m/h) have been used. Detention times of 2 to 3 hours at peak flow rates are frequently used. Longer detention times are required where there is no sludge collection equipment.

RESERVOIR OUTLETS

Spillways

All natural streams are subject to periodic flooding. The storage capacity of a reservoir will usually be less than the potential volume of runoff that can come from its watershed during a large flood, and there must be a provision to pass excess floodwaters through a spillway. The spillway is vital to the safety of the structure, and it is also generally one of the more costly parts of the project.

In many cases, there will be two spillways, one known as the service spillway and one as the emergency spillway. The service spillway is usually built of reinforced concrete and passes small and medium flows. The emergency spillway has the necessary additional capacity to handle very high flows, which seldom occur but must be included in the design. The emergency structure often will simply be a channel cut through one of the abutments to discharge into the streambed below the dam. Repair of the emergency channel will, at all times, be required after water flows through it, but this is acceptable because emergency bypass rarely happens. The adequacy of the combined spillway system is usually evaluated in terms of the probable maximum flood from the watershed.

Normal practice is to build the dam high enough to allow some freeboard above the maximum high water level that could occur during the probable maximum flood. This is done to keep waves from breaking over the top of the dam at the height of a storm. The amount of freeboard will vary, depending on reservoir location, depth, size and shape. Typically, it is 3 to 6 feet (0.91 to 1.83 m).[3]

Service Outlet

A requirement for most reservoirs is that they can release water and lower the lake level if desired. This provision, known as the service outlet, uses a conduit, controlled by gates or valves, passing through an abutment or under the dam at a level near the bottom of the reservoir. If the outlet passes beneath the dam, it is desirable to place the control mechanism at the upstream end, so that there is no water pressure in the conduit when it is shut off. A typical configuration involves an intake tower, standing in the water at the upstream toe of the dam, with several gated ports for entry of water from the lake. The service outlet conduit connects the intake tower to an outlet channel on the downstream side of the dam.

GROUNDWATER SUPPLIES

Subsurface Distribution of Water

In the United States, groundwater storage exceeds by many times the capacity of all surface reservoir and lakes, including the Great Lakes. It has been estimated that the total usable groundwater in storage is equivalent to the total precipitation for 10 years, or to the total surface runoff to streams and lakes for 35 years.[25] All water within the groundwater reservoirs, however, is not available for practical use because of such limiting factors as accessibility, dependability, quality, and cost of development.

Not all of the water that infiltrates the soil becomes groundwater. First, it may be pulled back to the surface by capillary force and be evaporated into the atmosphere. Second, it may be absorbed by plant roots growing in the soil and then reenter the atmosphere by the process of transpiration. Third, water that has infiltrated the soil deeply enough may be pulled downward by gravity until it reaches the level of the zone of saturation—the groundwater reservoir that supplied water to wells. The subsurface distribution of water is illustrated in Figure 8–14.

The upper stratum, where the openings are only partly filled with water, is called the *zone of aeration.* Immediately below this, where all the openings are completely filled with water, is the *zone of saturation.*

The zone of aeration is divided into three belts: the belt of soil water, the intermediate belt, and the capillary fringe. The belts vary in depth, and their limits are not sharply defined by physical differences in the earth materials. A gradual transition exists from one belt to another.

The *belt of soil water* is of particular importance to agriculture because it furnishes the water supply for plant growth. Water passing downward from this belt escapes the reach of the roots of most plants. The depth of the belt of soil water varies with the types of soil and vegetation, and may extend from a few feet to as much as 20 feet (6.1 m) or more below the surface.

The roots of some plants reach into the capillary fringe or the water table where this area is relatively close to the surface. This occurs mainly along stream courses.

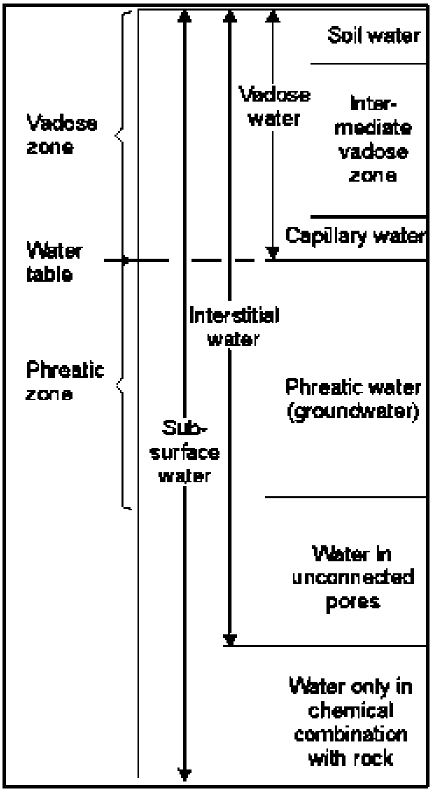


Fig. 8–14. Subsurface distribution of water

Such plants, called *phreatophytes,* grow without dependence upon the belt of soil water.

Water that does pass through the belt of soil water enters the *intermediate belt* and continues its movement downward by gravitational action. The intermediate belt holds suspended water by molecular attraction and capillarity, the latter being the more important of the two forces. The suspended water in this belt is dead storage because it cannot be recovered for use. The thickness of the intermediate belt varies greatly, and has a significant effect on the time it takes water to pass through this belt to recharge the zone of saturation.

The *capillary fringe* lies immediately below the intermediate belt and above the zone of saturation. It holds water above the zone of saturation by capillary force acting against the force of gravity. The thickness and the amount of water held in the capillary fringe depend on the grain size of the material.

The capillary fringe in silt and clay material is sometimes as much as 8 feet (2.43 m) thick. In coarse sand or gravel, it may be a fraction of an inch.

Water in the zone of saturation is the only part of all subsurface water that is properly referred to as groundwater. An exception to the above description of groundwater is ancient seawater found entrapped in some sedimentary formations. Groundwater of this origin is called *connate water.* Fresh water from precipitation percolating downward may slowly replace the salt water, but in many cases displacement of all the original seawater is not yet complete. Therefore, connate water remains in some formations in the zone of saturation.

Zone storage capacity is the total volume of the pores or openings in the rocks that are filled with water. The thickness of the zone of saturation varies from a few feet to many hundreds of feet. Factors that determine its thickness are: the local geology, the availability of pores or openings in the formations, the recharge, and the movement of water within the zone from areas of recharge toward points or areas of discharge.

Formations or strata within the saturated zone from which groundwater can be obtained for beneficial use are called *aquifers.* To qualify as an aquifer, a geologic formation must contain pores or open spaces that are filled with water, and these openings must be large enough to permit water to move through them toward wells and springs at a perceptible rate. Individual pores in a fine-grained material, such as clay, are extremely small, but the combined volume of the pores in such a formation is usually large. While a clay formation has a large water-holding capacity, water cannot move readily through the tiny open spaces. This means that a clay formation will not yield significant quantities of water to wells, and therefore it is not an aquifer even though it may be water-saturated.

A coarser material, such as sand, contains larger open spaces through which water can move fairly easily. A saturated sand formation is an aquifer because it can hold water, and it can transmit water at a perceptible rate when pressure differences occur.

The upper surface of the zone of saturation is called the *water table.* The shape of the water table is controlled partly by the topography of the land and tends, typically, to follow the shape of the land surface.

GROUNDWATER CONDITIONS

The various conditions under which groundwater may be found are shown in Figures 8–15 and 8–16. When the upper limit of the aquifer is defined by the water table, the aquifer is referred to as a *water-table aquifer* or an *unconfined aquifer,* or *free groundwater.* When a well is drilled in a water-table aquifer, the static water level in the well stands at the same elevation as the water table.

In some cases, a local zone of saturation may exist at some level above the main water table. This situation can occur where an impervious stratum within the zone of aeration interrupts percolation and causes groundwater to accumulate in a limited area above that stratum. The upper surface of the groundwater in such a case is called a *perched water table.*

The water table periodically moves up and down—rising when more water is added to the saturated zone by vertical percolation, and dropping during dry periods when previously stored water flows out toward springs, streams, wells, and other points of groundwater discharge.

The zone of saturation may include both permeable and impermeable layers. The permeable layers are aquifers. When an aquifer is found between impermeable layers above and below it, the aquifer and the water it contains are said to be confined. Because of the upper impermeable layers, the water of the aquifer is confined at pressures greater than atmospheric. Groundwater in such a situation is said to occur under artesian conditions, and the aquifer is called an *artesian aquifer.* The terms *confined aquifer* and *confined groundwater* are also used.

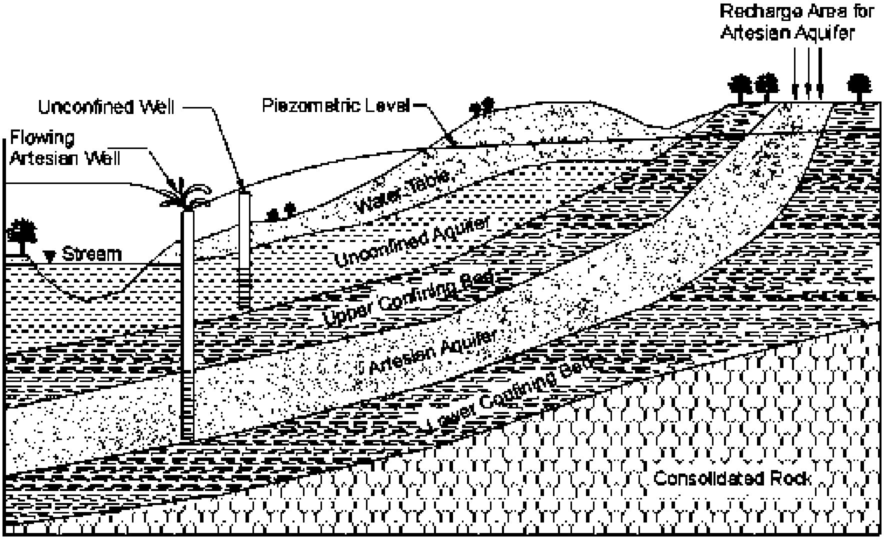


Fig. 8–15. Types of aquifers

When a well is drilled through the upper confining layer and into an artesian aquifer, water rises in the well to a level above the top of the aquifer. The water level in the well represents the artesian pressure in the aquifer. The hydraulic head, expressed in depth of water, at any point within the aquifer equals the vertical distance from this level down to the point in question.

The elevation to which the water level rises in an artesian well is referred to as the *piezometric level.* An imaginary surface representing the artesian pressure throughout all or part of an artesian aquifer is called the *piezometric surface.* This imaginary surface is analogous to the real water surface, the water table, in a water-table aquifer.

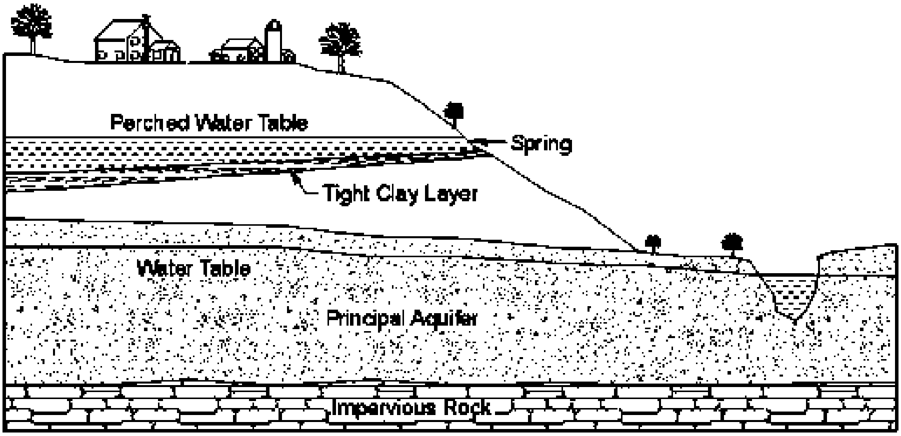


Fig. 8–16. Perched water table (Courtesy of US Filter/Johnson Sceeens)

The hydrostatic pressure within an artesian aquifer is sometimes great enough to cause the water to rise in a well above the land surface. A flowing artesian well results. Nonflowing artesian wells have the advantage that the water is higher in the casing and there is less pumping head required to bring the water to the surface than in an ordinary well.

Aquifer Functions and Properties

An aquifer performs two functions: storage and transmission. Aquifer properties related to the storage functions are porosity and specific yield. The property related to the transmission function is permeability. These properties are discussed below.

The *porosity* of a water-bearing formation is that part of its volume that consists of openings or pores not occupied by solid material. Porosity is a measure of how much groundwater can be stored in the saturated material, and is expressed as a percentage of the bulk volume of the material. For example, if 1 cubic foot (0.28 m^3) of sand contains 0.30 cubic feet (0.0084 m^3) of open spaces or pores, its porosity is 30 percent.

While porosity represents the amount of water an aquifer will hold, it does not indicate how much water the porous material will yield. When water is drained from a saturated material by gravity, only part of the total volume stored in its pores is released. The quantity of water that a unit volume of the material will give up when drained by gravity is called its *specific yield.*

The water that is not removed by gravity drainage is held against the force of gravity by molecular attraction and capillarity. The quantity that a unit volume retains when subjected to gravity drainage is called its *specific retention*. Both specific yield and specific retention are expressed as decimal fractions or percentages. Specific yield equals porosity minus specific retention. These factors can be applied to an aquifer to determine its potential yield, as shown in the following example.[24]

Example to Determine Potential Yield

A water-table aquifer extending over an area of 20 square miles (52 km^2), with an average thickness of 40 feet (12.2 m), occupies a total volume of 22.3 billion cubic feet (624.4 Mm^3). If the porosity were 25 percent, this groundwater reservoir would store 5.6 billion cubic feet (156.8 Mm^3) of groundwater. If the specific yield of the material were 10 percent and the upper 5 feet (1.52 m) of the aquifer were drained by lowering the water table 5 feet (1.52 m), then the total yield would be about 280 million cubic feet (7.84 Mm^3) of water, or about 2.1 billion gallons (7.95 Gm^3).

This quantity would supply four wells pumping 700 gpm (44.2 L/s) continuously, 12 hours each day, for 1,042 days, or almost 3 years. This pumping would be sustained by the groundwater stored in the upper 5 feet (1.52 m) of the aquifer in the absence of any replenishment to the aquifer during the 3-year period.

The above example illustrates how effectively groundwater aquifers can serve as reservoirs. Their enormous capacity often makes them more effective than surface reservoirs.

Permeability The property of a water-bearing formation related to its transmission function is called its *permeability,* which is the capacity of a porous medium for transmitting water. Movement of water from one point to another in the material takes place whenever a difference in pressure or head occurs between two points. Permeability may be measured in the laboratory by noting the amount of water that will flow through a sample of sand in a certain time and under a given difference in head.

This relationship has been quantified in Darcy's law:

$$V = P\,\frac{h_1 - h_2}{1} \tag{8–1}$$

where:

V = velocity of flow, ft/day
P = the coefficient of permeability, a constant that depends on the characteristics of the porous material through which the water flows, gpd/sq ft (m/h)
1 = distance along the flow path between points 1 and 2, ft (cm)
h_1 = hydraulic head measured at point 1, ft (m)
h_2 = hydraulic head measured at point 2, ft (m)
$\frac{h - h_2}{1}$ = hydraulic gradient (headloss per unit of travel) usually denoted as I, ft/ft (m/m)

The quantity of water moving through the aquifer can be determined by multiplying the above velocity by the cross-sectional area of the aquifer through which the water is moving.

The coefficient of permeability, often simply called the permeability, depends on the size and arrangement of the particles in an unconsolidated formation and on the size and character of the surfaces of the crevices, fractures, or solution openings in a consolidated formation. It may change with any variation in these characteristics. The coefficient of permeability is the quantity of water that will flow through a unit cross-sectional area of a porous material per unit of time under a hydraulic gradient of 1.00 (100 percent) at a specified temperature.

For convenient use in well problems, P is expressed as the flow in gallons per day (m^3/d) through a cross section of 1 square foot (1 m^2) of a water-bearing material under a hydraulic gradient of 1.00 and at a temperature of 60°F (15.6°C). The permeability unit with these dimensions is called a *Meinzer unit.*

The slope of the water table or the slope of the piezometric surface is the hydraulic gradient under which groundwater movement takes place. The total flow through any vertical section of an aquifer can be calculated from the thickness of the aquifer, its width, its average coefficient of permeability, and the hydraulic gradient at the section in question. The flow q per unit width of the aquifer is:

$$q = PmIc \tag{8–2}$$

where:

q = flow, gpd/ft of aquifer width (m^3/d/m of aquifer width)
P = average coefficient of permeability of the material from top to bottom of the aquifer, gpd/sq ft (m/d)
m = thickness of the aquifer, ft (m)

I = hydraulic gradient, ft/ft (m/m)
c = 1.0 for English units listed (80.5 for metric units listed)

The product of P and m is often used as a single term to represent the water-transmitting capability of the entire thickness of an aquifer, and is called the *coefficient of transmissibility*. This coefficient is the rate of flow in gallons per day (m^3/d) through a vertical section of an aquifer whose height is the thickness of the aquifer and whose width is 1 foot (1 m), when the hydraulic gradient is 1.00. The temperature assumed in this definition of the coefficient is the temperature of the groundwater in the aquifer.

The coefficient of transmissibility can be determined from aquifer pumping tests. Determination using field test data overcomes the problem of getting reliable values of the average coefficient of permability from laboratory tests. Inaccuracies are always present in the laboratory results because the samples are never entirely representative of the natural state of the formation from which they are taken.

Formations made up entirely of coarse, unconsolidated materials, such as gravel, give relatively high yields. Because of the large particle sizes in such a formation, the pores or voids are large. The large pores offer less resistance (higher permeability) to flow than the smaller pores in finer sands, so more water will flow through a unit area of the coarser material under any given pressure difference.

WELLS

Characteristics of Wells

Before we discuss the types and design, some key characteristics of wells need to be defined (refer to Figures 8–17 and 8–18). The height to which water will rise in a well without pumping is known as the *static level*. When pumping starts, the water level drops, and this decline below the static level is known as the *drawdown*. When a water table is pumped, the water-bearing beds around the well are unwatered for some distance, which is known as the *radius of influence*. This depression in the formerly saturated material is called the *cone of depression*. Under artesian conditions, similar results occur with pumping, but the beds are not unwatered. It is only the pressure in the water-bearing stratum that is reduced outward to the edge of the circle of influence. When pumping from a water table is stopped, it may require days or even weeks for the water to rise to the original static level, because the water must percolate back into the unwatered section. On the other hand, under artesian conditions, only a few minutes may be required for the pressure to be reestablished to, or almost to, the static level. The total lift of the pump is the drawdown, plus the friction head in the pump and pump discharge pipe from the drawdown point to the ground surface, plus any additional head required to lift the water to and above the ground.

The location of wells may have an important effect on their yield. Where a large amount of water from a formation is to be obtained by means of a number of wells, it is usually advantageous to locate them in a line perpendicular to the direction of the underflow.

The spacing of wells is important. If they are too closely spaced, the cones of depression of two or more wells will overlap and the interference between wells will cause mutual loss of head. Thus, the total lift required to raise the water to the surface will be increased, as will the pumping cost. If practical, heavily pumped wells that

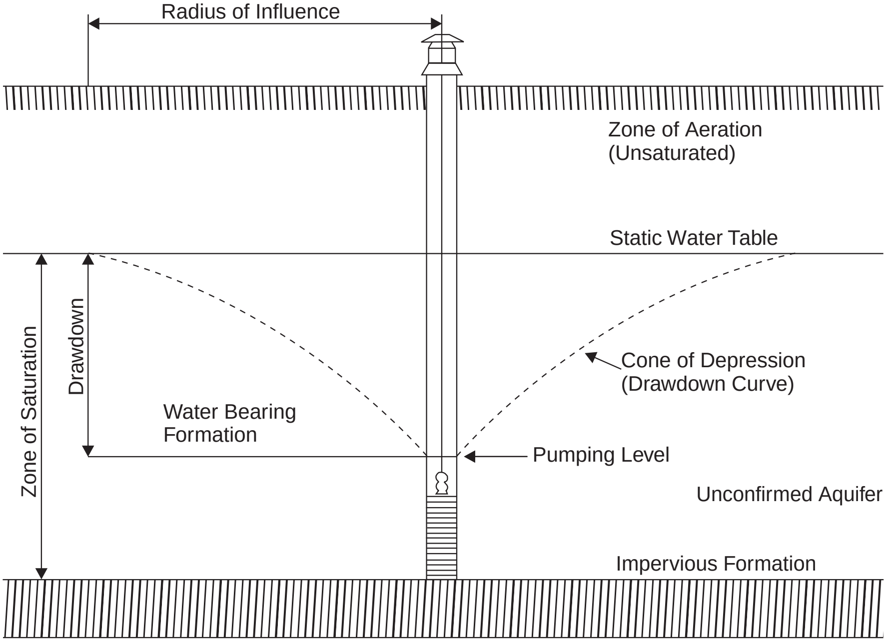


Fig. 8–17. Water-table well characteristics

draw on the same water-bearing strata should be spaced at least one-fourth of a mile apart—one-half mile is better—to avoid excessive interference.[26]

Types of Wells

Dug Wells. Dug wells, usually excavated by hand, are shallow in depth and vary from a few to many feet in diameter. They should be avoided unless geological conditions prevent the use of a drilled well. Because dug wells are especially subject to contamination by surface water, they must be carefully protected. This can be done by the use of metal casing, concrete walls, vitrified tile pipe, concrete pipe, or double brick walls. Large and deep dug wells are often constructed by sinking their liners as excavation proceeds. The casing should be seated securely in an impervious formation whenever possible. Where an impervious formation does not occur, protection of the supply is increased by extending the casing as far as is practical below the water table—if possible, at least 10 feet (3.05 m) below the lowest level of the water table. The casing should extend several inches above the pump room floor, and the floor should be elevated some distance above normal ground level. A concrete apron or ring should extend around the outside of the casing at least 12 inches (0.3 m) in order to divert surface water from the well.

A tight cover over the top of the well is necessary. The pump should be placed on the well cover in such a manner that wastewater from the pump will not return to the well. The pump base should also be sealed so that rodents, insects, and rainwater cannot enter the well. After the well has been completed, it should be disinfected with

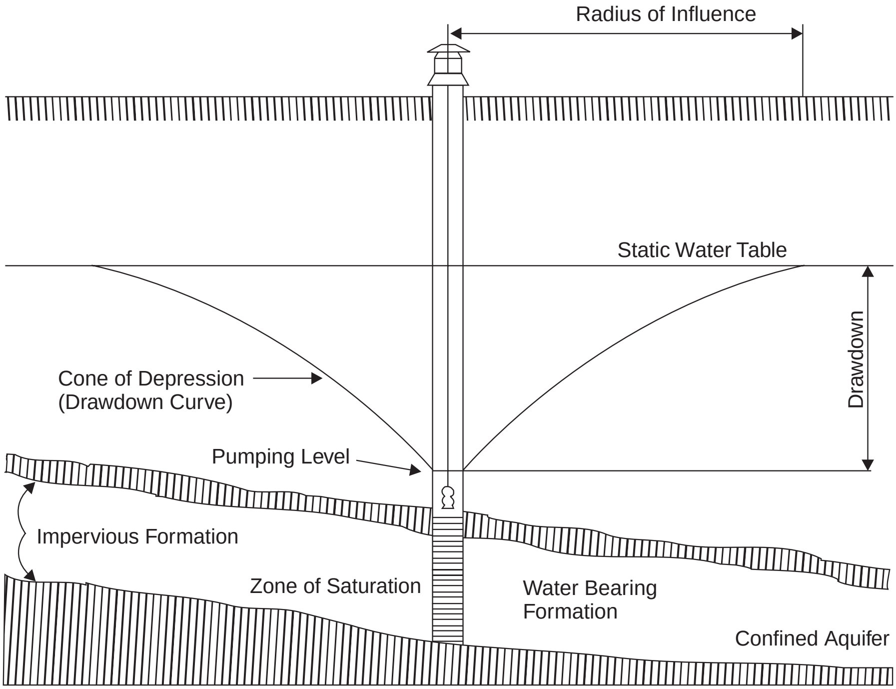


Fig. 8–18. Artesian well characteristics

chlorine. This type of well is not generally used for public supplies except where small quantities of water are required.

Driven Wells Wells are often driven when the formation involved is a relatively shallow sand layer. These wells usually are used when adequate quantities of water are found at depths of 20 to 70 feet (6.1 to 21.3 m). They are made simply by driving a metal pipe into the water-bearing stratum; the pipe acts as the permanent casing. Well-points are usually driven by hand when depths are 30 feet (9.1 m) or less. For greater depths, driving tools are often suspended from a tripod or derricks. With this type of well, special precaution should be taken because the strata penetrated are likely to show contamination. The well should be protected from surface waters by use of a concrete well top with an apron, the well pump should be properly sealed, and the well surroundings should be banked and tamped to divert surface waters. The well should have a solid casing that extends from a minimum of 12 inches (0.31 m) above the ground surface to at least 10 feet (3.05 m) below the groundwater surface.

Drilled Wells There are a large number of methods for drilling wells. Those most commonly used are percussion, rotary, or reverse-circulation drilling. The percussion method consists of lifting and dropping a heavy string of tools in the borehole. The drill bit breaks or crushes hard rock into soft fragments. In soft, unconsolidated rocks, the drill bit loosens the material. Water in the borehole (added if necessary) mixes with the crushed or loosened rock to form a slurry, which is removed with a sand pump or boiler. The drill bit is of a size that will permit the casing to be introduced into the well after drilling is completed.

In rotary drilling, a cutting bit is attached to a hollow drill rod rotated rapidly by an engine-driven rotary table. Either water or a suspension of colloidal clay is pumped down the drill pipe, flows through openings in the bit, and transports the loosened material to the surface. The clay suspensions are designed to reduce loss of drilling fluid into permeable formations, lubricate the rotating drill pipe, bind the wall against caving, and suspend the cuttings. In drilling for water, the thick drilling clay may be forced into the aquifer and reduce the flow into the well, but new methods of reaming and flushing have largely overcome such difficulties.

Reverse-circulation, rotary drilling is done with the flow of drilling fluid reversed with respect to the system used in the conventional rotary method. The drilling fluid and its load of cuttings move upward inside the drill pipe and are discharged by the pump into a settling pit. The fluid returns to the borehole by gravity flow. It moves down the annular space around the drill pipe to the bottom of the hole, picks up cuttings, and reenters the drill pipe through ports in the drill bit.

Boreholes with diameters up to 60 inches (1.53 m) can be drilled. To maintain a low velocity for the descending fluid, the diameter of the hole must be large in relation to the drill pipe. Descending velocities on the order of 1 foot/sec (0.305 m/s) or less are the rule.

Reverse-circulation offers the least expensive method for drilling large-diameter holes in soft, unconsolidated formations. Where geologic conditions are favorable, the cost per foot of borehole increases little with increase in diameter. Drilling cost for a 36- or 40-inch (0.92- or 1.02-m) hole is only moderately greater than for a 24-inch (0.61-m) hole.

Gravel-Packed Wells The effective diameter of a well can be increased by packing gravel between the well screen and the outer limits of the borehole. A well hole is first drilled and reamed to a diameter of 24 inches (0.61 m) or more. An outer casing is then cemented in place, and the aquifer is cleaned before a smaller inner casing carrying the well screen is inserted. Gravel is then packed into the annulus between the two casings. Gravel-packed wells are favored in fine uniform sands and in loosely cemented sandstones. In fine sands, the use of gravel permits the use of larger slot openings in the well screens. In loose sandstone, the gravel can prevent sloughing sandstone from entering the well without the need to resort to very small well screen openings. Another system consists of several standard vertical wells in a circular pattern. The individual wells are generally smaller in diameter than normal production wells, and when pumped simultaneously, all the wells produce as much as an extremely large single well. Ring wells are chosen for groundwater development where thin aquifers of fine sands are encountered that would require the use of very large-diameter well screens to obtain the desired capacity in a single well. Often it is more economical to drill several 6- to 10-inch (0.15- to 0.25-m)-diameter wells than one very large-diameter well.

Horizontal Collection (Ranney) Wells These wells, which are shown in Figure 8–19, are typically used to withdraw water from nearby rivers. They are constructed by lowering a concrete caisson into unconsolidated material, sealing the bottom with a concrete plug, and jacking slotted pipes radially out from the bottom of the caisson. Water enters the main shaft or caisson through these horizontal laterals, and is pumped

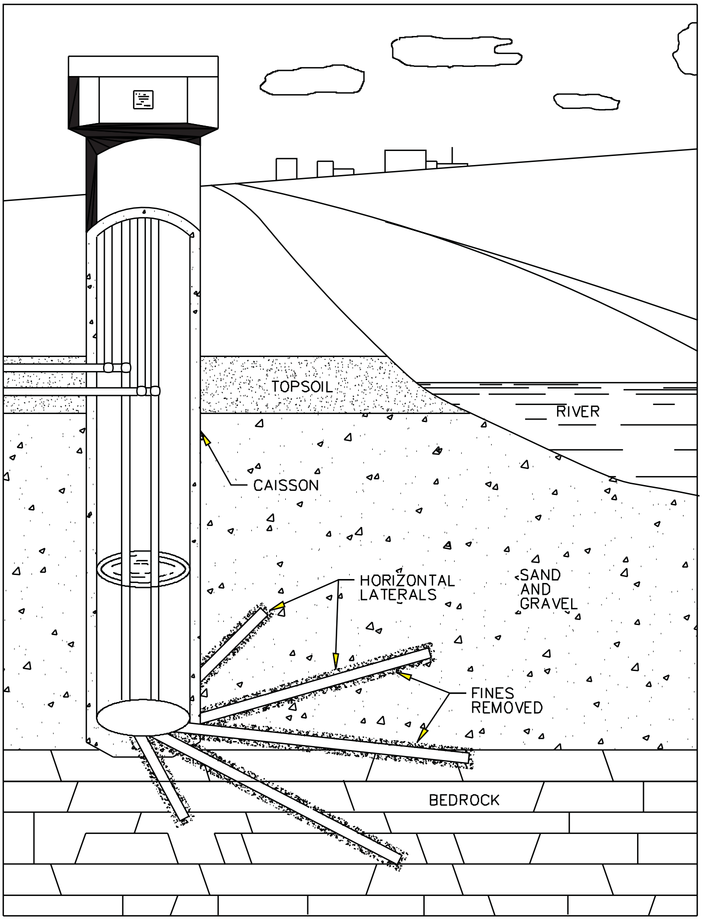


Fig. 8–19. Horizontal collector well (Reprinted from *Journal AWWA,* Vol. 71, No. 10 (October 1979), by permission. Copyright 1979, American Water Works Association)

to the water system by vertical turbine pumps hung in the caisson. The hydraulic head differential created between the river water surface and the water level in the caisson draws water through the riverbed material to the slotted pipes of the collector well. The advantage of this system is that it provides a very large intake area through which to draw water, much more than can be obtained from a single vertical well. However, collector wells are more expensive than a single vertical well.

Well Yield

The specific yield of a well is the discharge per foot of drawdown. The safe yield is the capacity of the aquifer to supply water without a continuous lowering of the water table or piezometric surface.

When water is pumped from a well, the quantity discharged initially is derived from aquifer storage immediately surrounding the well. As pumping continues, more water must be derived from storage at greater and greater distances from the well. The circular-shaped cone of depression must expand so that water can move from greater distances toward the well. The radius of influence of the well increases as the cone continues to expand. The drawdown also increases as the cone deepens to provide the additional head required to move the water from a greater distance. Over time, the cone expands and deepens at a decreasing rate because with each additional foot of horizontal expansion, a larger volume of stored water is available than from the preceding one. The cone will continue to enlarge until aquifer recharge equals the pumpage.

When the cone has stopped expanding for one or more of the above reasons, a condition of equilibrium exists. There is no further increase in drawdown with increase in time of pumping. In some wells, equilibrium occurs within a few hours after pumping begins; in others, it does not occur even though the length of the pumping period maybe extended for years.

Well discharge formulas for equilibrium conditions are well established.[25] There are two basic formulas, one for artesian conditions and the other for water-table conditions. Both assume recharge at the periphery of the cone of depression. Figure 8–20 shows a vertical section of a well constructed in a water-table aquifer. The formula for the water-table well is:

$$Q = \frac{P(H^2 - h^2)}{1055 \log R/r} \tag{8–3}$$

where:

Q = well yield or pumping rate, gpm (m^3/d)
P = permeability of the water-bearing sand, gpd/sq ft (m/d)
H = saturated thickness of the aquifer before pumping ft (m)
h = depth of water in the well while pumping, ft (m)
R = radius of influence, ft (m)
r = radius of the well, ft (m)
1055 = constant for English units listed (3.993 = constant for metric units listed)

Figure 8–21 is a vertical section of a well pumping from an artesian aquifer. The formula for a well operating under artesian conditions is:

$$Q = \frac{Pm(H - h)}{528 \log R/r} \tag{8–4}$$

where:

m = thickness of aquifer, ft (m)
H = static head at bottom of aquifer, ft (m)
528 = constant for English units listed (1.9984 = constant for metric units listed)

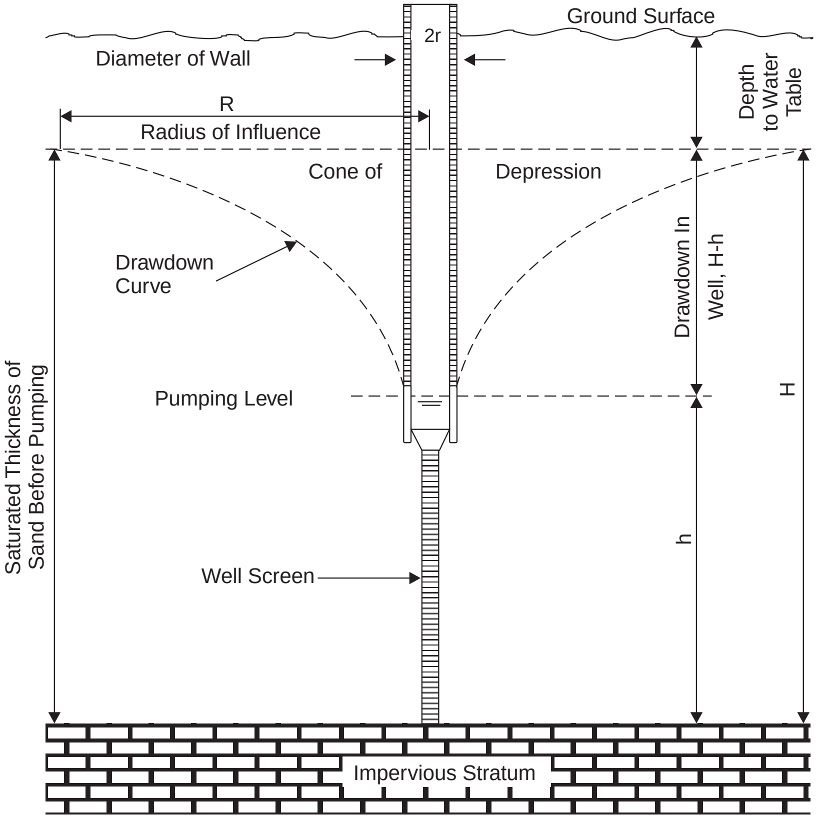


Fig. 8–20. Water-table well (Courtesy of US Filter/Johnson Screens)

All other terms are as defined in Equation (8–3) for a water-table well. The above formulas show how the well diameter affects yield:

$$Q \propto \frac{1}{\log R/r} \tag{8–5}$$

In other words, well yield is not a direct function of well diameter. For example, if a 6-inch (0.15-m) well will yield 100 gpm (6.3 L/s) with a certain drawdown, a 12-inch (0.31-m) well constructed at the same spot will yield 110 gpm (6.94-m) with the same drawdown, and an 18-inch (0.46-m) well will yield 117 gpm (7.38 L/s). A 48-inch (1.22-m) well will yield 137 gpm (8.65 L/s), or 37 percent more water than a 6-inch (0.15-m) well at the same drawdown.

Derivations of the above formulas are based on the following simplifying assumptions:[25]

1. The water-bearing materials are of uniform permeability within the radius of influence of the well.

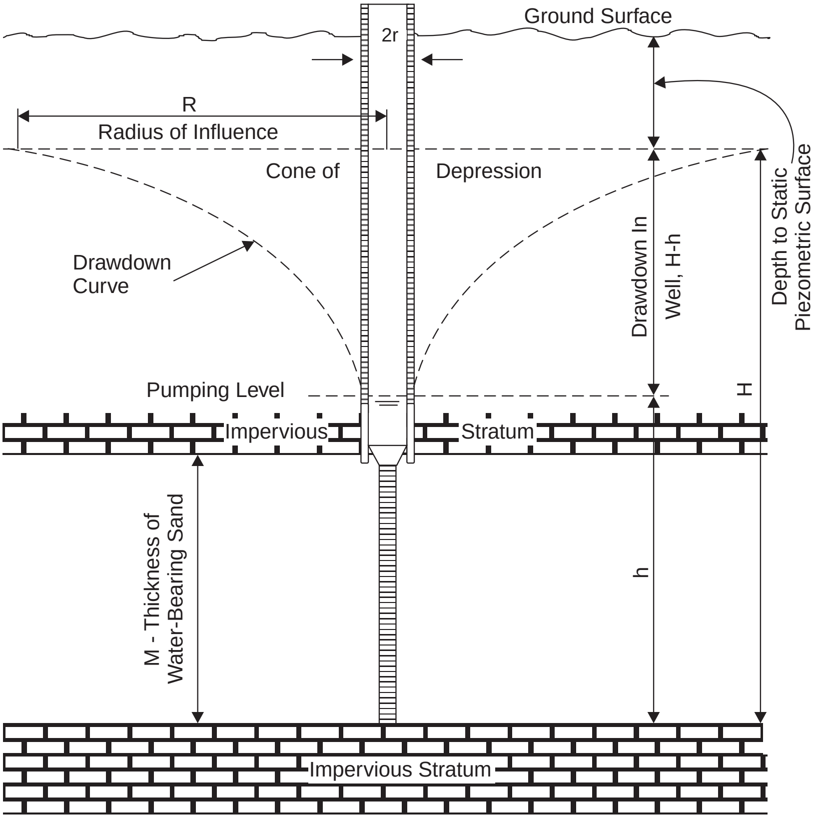


Fig. 8–21. Artesian well (Courtesy of US Filter/Johnson Screens)

2. The aquifer is not stratified.
3. For a water-table aquifer, the saturated thickness is constant before pumping starts; for an artesian aquifer, the aquifer thickness is constant.
4. The pumping well is 100 percent efficient.
5. The pumping well penetrates to the bottom of the aquifer.
6. Neither the water table nor the piezometric surface has any slope; both are horizontal surfaces.
7. Laminar flow exists throughout the aquifer and within the radius of influence of the well.
8. The cone of depression has reached equilibrium so that both the drawdown and the radius of influence of the well do not change with continued time of pumping at a given rate.

Uniform permeability is seldom found in a real aquifer, but the average permeability as determined from aquifer pumping tests has proved reliable for predicting well performance. For artesian wells where most of the aquifer thickness is penetrated and screened, the assumption of no stratification is not an important limitation. For water-table aquifers, where drawdown reduces the saturated thickness considerably, the situation can be handled when the stratification is known and taken into account in applying the formula.

Assumption of constant thickness is not a serious limitation because variation in aquifer thickness within the cone of depression in most situations is relatively small. Where changes in thickness are important, they can be taken into account.

Assumption that the well is 100 percent efficient can cause the calculated well yield to be seriously in error if the real well is inefficient. Therefore, the actual efficiency must be taken into account.

The water table or the piezometric surface is never truly horizontal; however, the slope is usually very flat, and its effect on calculation of well yield is negligible in most cases. The slope of the water table or the piezometric surface does cause distortion of the cone of depression, making it elliptical rather than circular.

The above formulas can be used to calculate well yield if P, H, m, and R are known. The well log provides values of H and m; R is usually estimated; P must be determined from laboratory or field tests. For a water-table aquifer, the formula for calculating P is:

$$P = \frac{1055Q \log r_2/r_1}{(h_2^2 - h_1^2)} \tag{8–6}$$

where:

P = permeability, gpd/sq ft (m/d)
Q = pumping rate, gpm (m^3/d)
r_1 = distance to the nearest observation well, ft (m)
r_2 = distance to the farthest observation well, ft (m)
h_2 = saturated thickness at the site of the farthest observation well, ft (m)
h_1 = saturated thickness at the site of the nearest observation well, ft (m)
1055 = constant for English units listed (2405.6 = constant for metric units listed)

For artesian conditions, the formula for determining the permeability is:

$$P = \frac{528Q \log r_2/r_1}{m(h_2 - h_1)} \tag{8–7}$$

where:

m = thickness of the aquifer, ft (m)
h_2 = head at the site of the farthest observation well, measured from bottom of aquifer, ft (m)
h_1 = head at the site of the nearest observation well, measured from bottom of aquifer ft (m)
528 = constant for English units listed (1203.9 = constant for metric units listed)

The remaining terms are the same as for the water-table aquifer.

Safe Yield

The safe yield of an aquifer is the quantity of water that can be withdrawn without major detrimental effects. ‘‘Significant effects’’ could be physical, such as drying up existing wells or springs, or holistic, such as drawing water across a study area boundary. Withdrawing the safe yield from an aquifer should not cause an overdraft condition, where groundwater levels fall over the long term. As with other water supplies, groundwater development should be sustainable, meaning pumping should not exceed recharge over the long term.

Well Casing Sizing

The selection of the casing diameter is important for two reasons: (1) it should be large enough to accommodate the pump with clearance for easy installation and efficient operations, and (2) it should be sized to provide a vertical water velocity equal to or less than 5 ft/sec.[27] The casing diameter should be two pipe sizes larger than the nominal pump diameter.[27] The recommended casing sizes for various pumping rates are shown in Table 8–4.

Well Screen Sizing

The screen slot openings should be based on the grain size distributions in the aquifer and should be sized to prevent entry of sand into the well. An accurate sample of the native material is needed to perform the sieve analysis. This becomes more challenging if the well is drilled using air or drilling fluids. Care should also be taken to locate the various strata and select the screen to fit. Fine sand (nominal size 0.0 inch, for example) may be followed by a coarse sand and gravel (nominal size 0.05 inch, for example). The finer materials should have a fine screen, which should penetrate some distance into the coarse material before changing to a coarser screen.

Well Screen Length

The length of the screen is based on the thickness of the water-bearing strata. The screen should be located in the strata having the highest hydraulic conductivity. Recommended screen lengths are:[27]

1) Homogeneous unconfined aquifer: For an aquifer that is less than 150 feet thick, the screen length should be one-third to one-half the aquifer depth and placed at the bottom of the aquifer.
2) Nonhomogeneous unconfined aquifer: The screens should be located in the most permeable layers of the lower portion of the aquifer, so that maximum drawdown is available.
3) Homogeneous confined aquifer: Eighty to 90 percent of the water-bearing aquifer should be screened.
4) Nonhomogeneous confined aquifer: Eighty to 90 percent of the most permeable layers should be screened.

TABLE 8–4. Recommended Well Diameters for Various Pumping Rates*

Anticipated Well Yield		Nominal Size of Pump Bowls		Optimum Size of Well Casing†		
gpm	m^3/day	in	mm	in	mm	
Less than 100	Less than 545	4	102	6 ID	152 ID	5 ID
75 to 175	409 to 954	5	127	8 ID	203 ID	6 ID
150 to 350	818 to 1,910	6	152	10 ID	254 ID	8 ID
300 to 700	1,640 to 3,820	8	203	12 ID	305 ID	10 ID
500 to 1,000	2,730 to 5,450	10	254	14 OD	356 OD	12 ID
800 to 1,800	4,360 to 9,810	12	305	16 OD	406 OD	14 OD
1,200 to 3,000	6,540 to 16,400	14	356	20 OD	508 OD	16 OD
2,000 to 3,800	10,900 to 20,700	16	406	24 OD	610 OD	20 OD
3,000 to 6,000	16,400 to 32,700	20	508	30 OD	762 OD	24 OD

(Courtesy of US Filter/Johnson Screens)

* For specific pump information, the well design engineer should contact a pump supplier, providing the anticipated yield, the head conditions, and the required pump efficiency.

† The size of the well casing is based on the outer diameter of the bowls for vertical turbine pumps, and on the diameter of either the pump bowls or the motor for submersible pumps.

Filter Pack Thickness

A filter pack is installed in the annular spaces between the screen and the ground. This filter pack is designed to prevent particles from the ground to enter the well. The filter pack should be more than 3 inches and less than 8 inches thick.[27] The filter pack gradation should be based on the gradation of the water-bearing strata. The finer the strata, the finer the filter pack gradation has to be.

Well Screen Diameter

The diameter is selected so that the entrance velocity of the water is less than 0.1 ft/sec.[27] The screen diameter can be adjusted after the opening size and length have been established. Increasing the diameter has less impact on the well capacity than increasing the screen length. The theoretical increase in yield can be estimated from:

$$Q \approx \frac{C}{\log R/r} \tag{8–6}$$

where:

C = constant
R = radius of influence, ft
r = radius of screen, ft

For typical situations of R = 400 ft, doubling the screen diameter increases the capacity by only 10 percent.

Recharge

Intentional groundwater recharge can be used to enhance the yield of an aquifer. Surface water, imported water, stormwater, or treated wastewater can be captured and percolated or injected into the ground. If soil and aquifer conditions permit, and land costs are reasonable, recharge ponds or spreading basins are generally the most cost-effective way to get large quantities of water into the ground (Figure 8–22). Injection wells can also be used, but costs are relatively high for the amount of water recharged.

One of the most effective modes of groundwater recharge is liberal irrigation in wet years, as long as something other than groundwater is used for irrigation. It is the best way to get a large land area contributing to groundwater recharge. The recent trend, however, has been conversion to high-efficiency irrigation systems, which endeavor to minimize deep percolation. This is highly desirable in dry years, but severely inhibits the ability to recharge in wet years. Making matters worse, many high-efficiency systems require very clean water because of small nozzle and/or emitter openings, operated over long periods of time. Because surface water is often laden with silt, algae, or other debris, and deliveries are scheduled for relatively short durations, growers tend to switch from surface water to private wells for irrigation. So in the conversion, the groundwater supply takes a double hit; new consumptive use plus lost recharge.

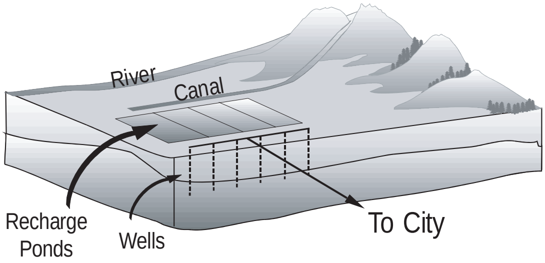


Fig. 8–22. Groundwater recharge using spreading basins

GROUNDWATER QUALITY CONSIDERATIONS

When water seeps downward through overlying material to the water table, particles held in suspension, including microorganisms, may be removed. The extent of removal depends on the depth and character of the overlying material. The bacterial quality of the water also generally improves during storage in the aquifer because time and storage conditions are usually unfavorable for bacterial multiplication or survival. Of course, the clarity of groundwater does not guarantee safe drinking water, and only adequate disinfection can guarantee the absence of pathogenic organisms.

After a well has been completed and is to be tested for yield, or any time after a permanent pump is installed or repaired, the well should be disinfected. The procedure is outlined in detail in AWWA A100-66, American Water Works Association Standard for Deep Wells. This standard recommends the use of hypochlorites or fresh chlorinated lime in a concentration of 50 mg/L of chlorine.

Four alternative procedures are given: (1) preparation of the disinfecting solution at the ground surface to a concentration of 50 mg/L and to a quantity at least twice the volume of the well, after which the solution is discharged rapidly into the well interior; (2) preparation of a stock solution containing 15,000 mg/L of chlorine and diluting it to the 50 mg/L concentration in the well by feeding it into a continuous flow of water; (3) adding the stock solution directly to the well and agitating the well contents with a baler or bit; and (4) placing the calculated amount of dry hypochlorite in a perforated pipe section capped at both ends and moving the pipe up and down within the well casing. The disinfection period should be at least 2 hours, and the procedure should not be attempted until after the well has been thoroughly cleansed of oil, grease, and foreign matter of any kind. After the well has been pumped until all of the chlorine solution has been removed, a sample should be collected for bacteriological examination.

All groundwater withdrawal points should be located a safe distance from sources of pollution. Sources of pollution include septic tanks and other individual or semipublic sewage disposal facilities, sewers, and sewage treatment plants, industrial waste discharges, land drainage, farm animals, fertilizers, and pesticides. Where water resources are severely limited, groundwater aquifers subject to contamination may be used for water supply if adequate treatment is provided.

Because many factors affect the determination of safe distances between groundwater supplies and pollution sources, it is impractical to set fixed distances. Where insufficient information is available to determine the safe distance, the distance should be the maximum that economics, land ownership, geology, and topography will permit. If possible, a well site would be located at an elevation higher than that of any potential source of contamination. The direction of groundwater flow does not always follow the slope of the land surface, so the slope of the water table should be determined from observation wells.

Groundwater quality in aquifer systems varies spatially and may range from good to unacceptable. Water-quality problems vary from high concentrations of dissolved solids to small amounts of trace elements, organics, and pathogens that exceed drinking water standards. In some cases water-quality problems become apparent only after a water well is drilled, constructed, developed, and tested. However, test wells may be used to predict the quality of water that will be obtained from permanent, high-capacity wells. Other chapters of this book address the significance of various contaminants and techniques for their removal. A few contaminants are more frequently of concern in groundwater than in surface water, including iron, manganese, fluoride, and nitrates.

Iron can cause staining of plumbing fixtures and clothes and may encrust well screens and pipes. Concentrations greater than 0.5 mg/L are usually troublesome.

Water may pick up iron from contact with the well casing, pump parts, and piping. The more corrosive the water, the more metal it will dissolve from the iron surfaces with which it comes in contact. Water standing in a well that has been idle will have a higher iron content than the water in a water-bearing formation.

Upon contact with air, dissolved ferrous iron changes to the ferric state and precipitates. The resulting iron hydroxide and iron oxide are commonly called rust.

Well water containing iron in appreciable amounts may be completely clear and colorless when first pumped. If it stands for a time, oxygen from the air oxidizes the dissolved iron. The water grows cloudy and will yield a deposit of a rust-colored material.

Iron-bearing waters also favor the growth of iron bacteria, such as crenothrix. These growths can form so abundantly in water mains, recirculating systems, and other places that they exert a marked clogging action.

Manganese resembles iron in its chemical behavior. Because manganese is less abundant in rock materials than iron, its occurrence in water is less common than that of iron. Manganese occurs in groundwater as soluble manganous bicarbonate, which changes to insoluble manganese hydroxide when it reacts with oxygen of the air. The stains caused by manganese are more annoying and harder to remove than those caused by iron.

Fluoride in groundwater may be derived from fluorite, the principal fluoride mineral of igneous rocks, or from any of a considerable number of complex fluoride-bearing minerals. Volcanic or fumarolic gases may also contain fluoride and may be the source of fluoride in water. Too much fluoride in the water has been shown to be associated with the dental defect known as mottled enamel. This may appear on the teeth of children who drink water containing too much fluoride during the period when permanent teeth are formed. Conversely, small concentrations of fluoride are beneficial and help to prevent tooth decay. The desirable fluoride content varies with air temperature, as shown in Chapter 20.

Nitrate in concentrations greater than 45 mg/L (as NO_3) is undesirable in water used for domestic purposes because of the possible toxic effect that it may have on young infants. Nitrates are transported through soil without significant degradation or adsorption. Groundwater under heavily fertilized areas may have very high (more than

100 mg/L) nitrate concentrations. Other major sources may include liquid wastes and bacterial fixation of atmospheric nitrogen.[28]

EVALUATION OF WATER SUPPLY ALTERNATIVES

Water supply is the most fundamental requirement of an urban community. As such, water can be considered a least cost commodity. It isn't a question of *whether* people will choose to purchase water, only how much it will cost, and the magnitude of the other consequences. Water supply planning must consider both cost and non-cost factors in selection of a preferred plan. Cost factors include capital, operation, maintenance, and replacement costs, and rate impacts. Non-cost factors may include reliability, environmental impacts/benefits, public acceptance, drought resilience, water budget impacts, groundwater quality impacts, responsiveness to future regulatory changes, compatibility with phased construction, institutional complexity, funding complexity, and ease of implementation. A rating system can be employed to evaluate and rank the various water supply alternatives (Table 8–5).

TABLE 8–5. Sample Water Supply Alternative Ranking Criteria

	Alternative		
Criteria	1	2	3
Ability to meet water supply objectives			
Capital cost (ability to fund)			
Ability to phase construction			
Rate impacts			
Ease of implementation			
Balanced water budget			
Reliability			
Drought resilience			
Groundwater quality protection			
Surface water quality protection			
Response to environmental regulation changes			
Response to drinking water regulation changes			
Environmental impacts			
Accommodates land use objectives (accommodates or constrains growth)			
Impacts to other stakeholders			
Totals			

Note: Alternatives are rated based on a 1–3 or 1–5 scale for each criterion, then totaled at the bottom of each column. Weighting factors can also be used to amplify certain criterion, if desired.

Based on this evaluation, water supply alternatives are developed in their rank order, as needed to serve water demands into the future. Associating key water supply milestones with demand levels instead of dates is called event-driven planning (Figure 8–23). Because it is difficult to accurately forecast land use, population, and water demands beyond about 5–10 years, use of EDP enables the water supply planner to identify a plan that is responsive to future demographic changes.

WATERSHED MANAGEMENT

Increasingly, integrated water resources management is being used for long term planning over an entire watershed. The term ‘‘watershed’’ has a strict definition in hydrology as the land area that drains to a given point. However, in the regulatory framework, the use of the term watershed has been broadened to effectively mean ‘‘problem-shed.’’ In this chapter, the term ‘‘study area’’ has been used. Either way, the concept is to manage the water resources of a reasonably large area in order to bracket the principal water resources issues at hand, but not paralyze the planning process. The key drivers for watershed planning could be water needs, river water quality, groundwater contamination, endangered species, land-use changes, flood control, recreation, hydropower, navigation, water rights adjudication, and competing water interests, but more likely a combination of these factors.

Stakeholder involvement is critical to successful water resources management. Although it may be tempting to try to enlarge the study area and solve a broad range of water resources issues within a study, recent history has shown that the difficulty of coming to consensus is geometrically proportional to the number of stakeholders and driving issues. A more appropriate stakeholder involvement standard is consent. Considering that each critical stakeholder effectively holds a veto in the process, the solutions must be acceptable enough to all critical stakeholders to keep them from exercising that veto.

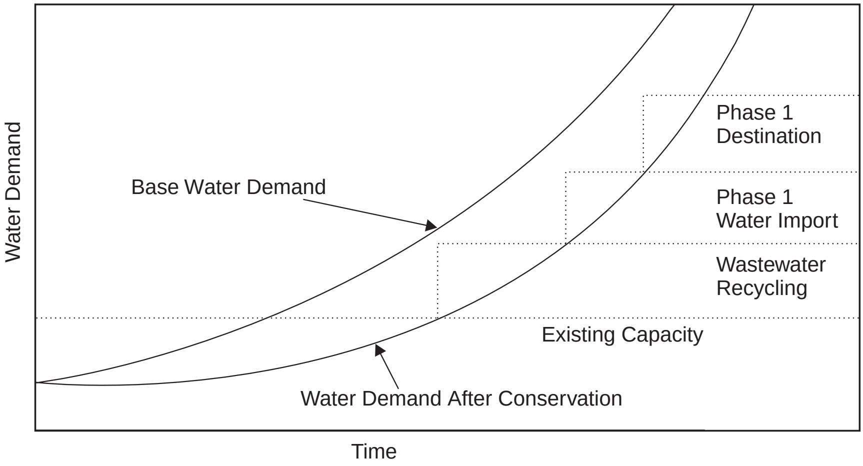


Fig. 8–23. Event-driven water supply planning

The following steps are involved in developing a watershed management plan.[6,29]

- Identify and characterize surface water sources
- Identify and characterize potential surface water impacts
- Determine the vulnerability of intake to contaminants
- Establish protection goals
- Develop protection strategies
- Implement the program
- Monitor and evaluate the program

Identify and Characterize Surface Water Sources

The outer boundary of the watershed with all the relevant water sources should be defined. The outlined area must include all upland streams and water bodies that drain to the treatment plant intake.

Additional information is desirable to have and includes such characteristics as climate, topography, geology, soils, vegetation, wildlife, land use, and ownership. Groundwater recharge zones should be identified to determine the degree to which surface waters and groundwater sources affect each other. Understanding these factors enables the watershed manager to assess natural impacts on water quality.

Identify and Characterize Potential Surface Water Impacts

The next consideration is to determine potential source impacts. Table 8–6 lists illustrative potential contaminant sources. The potential sources need to be ranked according to priority for control. Watershed activities and land uses that may be causing contamination need to be identified along with any monitoring data that may be available. Natural watershed basin features, such as steep slopes, highly erosive and clayey soils, and wildlife riparian areas, need to be identified. An assessment needs to be made as to the future water quality impact from increased land use activities accompanying projected growth of the area.

Determine Vulnerability of Intake to Contaminants

Intake vulnerability to potential contaminants, from the sites inventoried, needs to be determined. Point discharges are easier to assess since there is a definable flow and pollutant load. As one moves to less definable nonpoint sources, the effect on the water supply intake will become more difficult to assess because the frequency and intensity is based on runoff events. Accidental spills occur at random and are even more difficult to assess.

Intake vulnerability is based on the ability of the water treatment processes to remove contaminants and prevent them from passing to the distribution system or resulting in significant disinfection by-product formation. Water quality monitoring, modeling, and on-site assessments are three ways to determine the effect of land use on source water quality.

TABLE 8–6. Potential Watershed Contaminant Sources

- **Agriculture**
  - *Crop-related sources*
    - Irrigated crop production
    - Non-irrigated crop production
    - Specialty crop production (e.g., horticulture, citrus, nuts, fruits)
  - *Grazing-related sources*
  - *Intensive animal feeding operations*
    - Concentrated animal feeding operations
    - Confined animal feeding operations
    - Aquaculture
- **Atmospheric deposition**
- **Collection system failure**
- **Combined sewer overflow**
- **Construction**
  - Highway/road/bridge construction
  - Land development
- **Contaminated sediments**
- **Debris and bottom deposits**
- **Domestic wastewater lagoon**
- **Erosion from derelict land**
- **Graveyards**
- **Groundwater loadings**
- **Groundwater withdrawal**
- **Habitat modification**
  - Removal of riparian vegetation
  - Bank or shoreline modification/destabilization
  - Drainage/filling of wetlands
- **Highway maintenance and runoff**
- **Hydromodification**
  - Channelization
  - Dredging
  - Dam construction
  - Upstream impoundment
  - Flow regulations/modification
- **Industrial Point Sources**
- **Injection wells**
- **Internal nutrient cycling (primarily lakes)**
- **Land disposal**
  - Sludge
  - Wastewater
  - Landfills
  - Inappropriate waste disposal/wildcat dumping
  - Industrial land treatment
  - On-site wastewater systems (septic tanks)
  - Hazardous waste
  - Septage disposal
- **Landfills**
- **Leaking underground storage tanks**
- **Marinas and recreational boating**
  - In-water releases
  - On-land releases
- **Municipal point sources (wastewater discharges)**
- **Natural sources (e.g., arsenic, radon, wildlife)**
- **Open dumps**
  - Pipelines
- **Recreational extraction**
  - Mining
  - Petroleum activities
  - Mill tailings
  - Mine tailings
  - Acid mine drainage
  - Abandoned mining
  - Inactive mining
- **Salt storage sites (e.g., deicing)**
- **Sediment resuspension**
- **Sewer lines (leaking)**
- **Silviculture**
  - Harvesting, restoration, residue management
  - Forest management (e.g., pumped drainage, fertilization, pesticide application)
  - Logging road construction/maintenance
  - Silviculture point sources
- **Sources outside state jurisdiction or borders**
- **Spills (accidental)**
- **Urban runoff/storm sewers**
- **Water storage/storage tank leaks**

Source: Reference 29.

Establish Protection Goals

The primary source protection objective is providing high-quality water to the consumer. Goals that support the primary objective can be based on the water quality parameters of concern and the characteristics of the watershed. Examples may include pollutant load reduction, protection from urban development, avoidance of treatment and disinfection changes, minimizing risks from hazardous chemicals, mitigating effects from natural disasters, land preservation, and enhancing fish and wildlife habitats. When establishing protection goals, public and agency input is essential.

Develop Protection Strategies

Protection strategies include land use controls and best management practices (BMPs) in urban, agricultural, and forest areas. Table 8–7 summarizes typical BMPs. BMPs can be structural and nonstructural. Structural BMPs involve the construction of physical structures that control water quality. A nonstructural BMP involves activities that a landowner may undertake to control the pollutant load.

Implement the Program

Developing a plan to control the impacts of this widely divergent list of factors is a complex task usually involving the coordination of many different agencies, landowners, and the public. Since the water treatment utility rarely has the power to unilaterally impose best management practices throughout the watershed, consensus building among the parties involved in the watershed is usually required to find those management practices that are economically feasible and politically acceptable.

Monitor and Evaluate the Program

Water quality monitoring is essential in determining the effectiveness of a watershed protection program. Monitoring will provide an early warning signal to an impending problem. Special studies and sanitary surveys should be conducted periodically to better understand the watershed dynamics. By comparing the results of the evaluation to the goals and objectives of the program, the watershed manager can make adjustment.

WELLHEAD PROTECTION PROGRAMS

The Safe Drinking Water Act mandates that each state develop a wellhead protection program. The program should generally include the following elements:

- Delineate a wellhead protection area for each well.
- Complete an inventory of potential sources of contamination within the delineated area.
- Develop recommended practices to reduce the potential for contamination.
- Prepare contingency plans for providing alternate sources of drinking water.

TABLE 8–7. Typical Best Management Practices (United States Public Health Service).*

Agricultural	Forestry	Urban
	Nonstructural	
Tillage and cropland erosion control Pesticide and fertilizer application Range and pasture management Contour farming and strip cropping Confined feedlot management Cover cropping Crop residue usage Cropland irrigation management	Forestry preharvest Streamside management areas Forest chemical management Fire management Forest vegetation of disturbed areas	Land use planning and management Public acquisition of watershed land Minimum lot size zoning restrictions Impervious surface restrictions Buffer zones and setbacks Public information and education Citizen advisory committees Watershed sign posting Storm drain stenciling Illegal dumping and illicit connection controls Material exposure controls Material disposal and recycling Household hazardous pickup days Used motor oil collection Wastewater disposal restrictions Septic tank management Community wastewater systems control Sanitary sewer facilities planning and management Catch basin and street cleaning Construction site land stabilization
	Structural	
Animal waste management Terrace systems Diversion systems Sediment basins Filter strip and field borders	Erosion and sediment controls Access roads Skid trails Stream crossings Filter strip sediment controls	Detention/retention facilities Wet detention ponds Extended detention ponds Vegetated swales and strips Constructed wetlands Infiltration ponds and trenches Drainage structure controls Inlet floatable controls Oil water separators Media filtration Erosion and sediment control Stream bank stabilization and riparian buffer restoration

Source: Reference 30.

*Water body BMP: Direct use of the source water for recreational activities can be controlled by river and reservoir management restrictions on body contact recreation, motorboat engine restrictions, bird control, and shoreline restoration.

Wellhead Protection Area

This is the area around a well, wellfield, or spring and through which contaminants could pass to the groundwater that feeds the well. The threat of contamination is reduced the farther from the well a contaminant is spilled onto the ground.

There are a variety of methods used to establish the wellhead protection area. They would typically include:

- A sanitary control area
- A series of expanding circles with the well as the center
- A buffer area

The sanitary control area is the area that should be managed to eliminate the possibility of surface flows reaching the well and traveling down the casing to the groundwater supply. The design of the wellhead should include precautions against incidental contamination through including a surface or sanitary seal in the well. The top of the casing should be above the ground and be protected from interference by fencing it in.

The next zones are based on protecting the well from viral, microbial, and other chemical contaminants that may pass down through the ground to the aquifer. These may be based on theoretical travel times or other methods.

A buffer zone may encompass the entire watershed or supply focus on critical areas, such as recharge zones, artesian areas, or other sensitive items.

Inventory of Potential Contaminant Sources

Many of the potential contaminants previously identified in Table 8–5 must also be considered for a groundwater source.

DEVELOP PROCEDURES TO REDUCE THE POTENTIAL FOR CONTAMINATION

The steps previously described for surface waters describe basic steps that are largely applicable to ground waters.

CONTINGENCY PLANS

Contingency plans should be prepared to handle any unforeseen incidents. Local officials must be able to respond quickly and efficiently to these situations and be able to provide an alternate source of drinking water. The plan should include both short-term and long-term actions in the event of a spill. The plan may include all or some of the following items:

- Impact through the loss of the largest well. Determine capacity needs.
- Evaluate the capacity of the existing system to determine if it can handle added flows.

- Identify potential interties to other wells or the water system of an adjacent water purveyor.
- Consider existing emergency preparedness plans, and update as needed for the water system.
- Possibly identify future sources of water, and evaluate their quality and suitability as a drinking water source.
- Provide current list of emergency personnel and their phone numbers.

REFERENCES

1. Walker, R., *Water Supply, Treatment and Distribution.* Prentice Hall, Englewood Cliffs, NJ, 1978.
2. Culp, Wesner, *Technical Guidelines for Public Water Systems,* USEPA, Contract 68-01-2971, 1975.
3. Gooch, R. S., ‘‘Surface Water Supplies,’’ *Manual of Water Utility Operations,* Texas Water Utilities Association, Distributed through Texas State Dept. of Health, Austin, TX, 1975.
4. Davis, C. V., *Handbook of Applied Hydraulics,* McGraw-Hill, New York 1942.
5. AWWA Committee, ‘‘Water Supply Resources: How Much Storage Space Remains?’’ Committee Report, *JAWWA*, p. 674, 1975.
6. AWWA, ‘‘Watershed Sanitary Survey Guidance Manual,’’ AWWA, California-Nevada Section, December 1993.
7. Hutcheon, B. C., and Smith, D. W., ‘‘Raw Water Studies Determine Siting of New Intake,’’ *JAWWA*, p. 16, 1981.
8. Fair, G. M., Geyer, J. C., and Okun, D. A., *Water and Wastewater Engineering,* John Wiley & Sons, Inc., New York 1966.
9. Sinclair, R. M., ‘‘Clam Pests in Tennessee Water Supplies,’’ *JAWWA,* 56(5):592, 1964.
10. Herbert, P. D. N., II, Muncaster, B. W., and Mackie, G. L., ‘‘Ecological and Genetic Studies on *Dreissena polymorpha* (Pallas): A New Mollusc in the Great Lakes.’’ *Can. Jour. Fisheries and Aquatic Sci.,* 46:1587, 1989.
11. Smith, A. L., et al., ‘‘Clams—A Growing Threat to Implant Water Systems,’’ *Plant Engrg.,* 33:165, 1979.
12. Haney, P. D., ‘‘Water Reuse for Public Supply,’’ *JAWWA,* p. 73, 1969.
13. Culp, Wesner, *Guidance for Planning the Location of Water Supply Intakes Downstream from Municipal Wastewater Treatment Facilities,* USEPA 68-01-4473, April 1978.
14. Manka, J., et al., ‘‘Characterization of Organics in Secondary Effluents,’’ *Environmental Science and Technology,* 8:1017, November 1974.
15. Sachdev, D. R., et al., ‘‘Apparent Molecular Weights of Organics in Secondary Effluents,’’ *JWPCF,* p. 570, 1976.
16. Katz, S., et al., ‘‘The Determination of Stable Organic Compounds in Waste Effluents at Microgram per Liter Levels by Automatic High Resolution Ion Exchange Chromatography,’’ *Water Research* 6:1029–1037, 1972.
17. Helfgott, T., et al., ‘‘Analytic and Process Classification of Effluents,’’ *Proc. ASCE SA.* 3: 779–803, June 1970.
18. Hoehn, R. C., et al., ‘‘Algae as Sources of Trihalomethane Precursors, *JAWWA,* p. 344, June 1980.
19. Craun, G. F., et al., ‘‘Water Borne Disease Outbreaks in the U.S., 1971–74,’’ *JAWWA,* pp. 420–424, August 1976.

20. Irwin, W. H., et al., ‘‘Impoundment Destratification by Mechanical Pumping, *JSED, ASCE 92:SA6:21,* December 1966.
21. Symons, J. M., et al., ‘‘Management and Measurement of DO in Impoundments,’’ *JSED, ASCE 93:SA6:181,* December 1967b.
22. Symons, J. M. et al., ‘‘Impoundment Water Quality Changes Caused by Mixing,’’ *JSED, ASCE* 93:SA2:1, April 1967a.
23. Thackston, E. L., ‘‘Discussion of ‘Impoundment Destratification by Mechanical Pumping,’ ’’ *JSED, ASCE,* 93:SA2:45, 1967.
24. Ficek, K. J., and Emanual, A. G., ‘‘Surface and Well Water Treatment with Potassium Permanganate,’’ *J. MWSC,* p. 25, 1965.
25. Universal Oil Products Co., *Ground Water and Wells,* Johnson Division, St. Paul, MI, 1974.
26. Harvill, C. R., and Billings, C. H., ‘‘Ground Water Supplies,’’ *Manual of Water Utility Operations,* Texas Water Utilities Association, Distributed through Texas State Dept. of Health, Austin, TX, 1975.
27. Driscol, F. G., *Groundwater and Wells,* 2nd edition, Johnson Screens, St. Paul, MN, 1986.
28. Kaufman, W. J., ‘‘Chemical Pollution of Ground Waters,’’ *JAWWA*, p. 152, 1974.
29. Pontius, F. W. (ed.), *Water Quality and Treatment,* AWWA, 5th edition, 1999.
30. United States Public Health Service, *Manual of Recommended Water Sanitation Practice,* 1946.

CHAPTER 9

Aeration and Air Stripping

INTRODUCTION

Water treatment facilities have come to employ a wide variety of gas transfer processes, although the basic theory of these processes has remained relatively unchanged since the 1920s. Historically, aeration has been used for taste and odor control and to remove excess carbon dioxide from water. Since the 1970s, it has also been used for the removal of radon and volatile organic compounds (VOCs). In December 1997, the United States Environmental Protection Agency (USEPA) released a drinking water advisory for methyl tertiary butyl ether (MTBE), a volatile gasoline oxygenate, after this compound was detected in numerous water supplies throughout the United States. In October 1999, the USEPA released the proposed drinking water standard for radon, which cited high-performance aeration as the proposed best available technology for radon removal. These recent developments will likely spur additional interest in the use of aeration for drinking water treatment.

This chapter is organized into three parts: The first section discusses the fundamentals that govern all aeration processes; the second part covers the different types of unit processes used for aeration, as well as the associated design considerations; and the third part includes specific applications of aeration in water treatment.

GAS TRANSFER FUNDAMENTALS

Aeration in water treatment depends upon two basic principles: equilibrium and mass transfer kinetics. In the case of removing VOCs, the water is initially supersaturated with the contaminant gas of concern, and the gas will tend to diffuse from the water. In other cases, such as the addition of oxygen to groundwater, the water is initially undersaturated with a desired gas and the goal is to increase the concentration of that gas in the finished water. In both cases, the limit to how much gas can be transferred is defined by the equilibrium between the gas in solution and in the vapor phase. The rate at which this transfer occurs is governed by kinetics.

Equilibrium

Equilibrium conditions define the limits of the gas transfer process. Aeration drives a system toward equilibrium regardless of whether water is undersaturated or oversaturated with a gas. Equilibrium is affected by the properties of the gas, the temperature, dissolved solids, and the partial pressure of the gas. For a fixed set of conditions, the equilibrium concentration of a gas in water is proportional to the partial pressure of the substance in the gaseous phase. This relationship is linear at low partial pressures and is commonly known as Henry's law (discussed next). It is valid for most applications of aeration in water treatment.

Henry's Law

Where the total pressure of the system is 1 atm, the concentration of a gas in water is defined by Henry's law, which can be expressed as follows:

$$C = \frac{H}{P}$$

where

C = concentration of the gas, mol gas/mol water
H = Henry's constant, atm
P = partial pressure of the gas, atm

Another common way of expressing Henry's constant is in units of atmosphere-liters per milligram. The conversion from one form of Henry's constant to another is as shown below:

$$H* = \frac{H}{(55.56 \text{ mol/L}) \times (1{,}000 \text{ mg/g}) \times \text{MW}}$$

where

$H*$ = Henry's constant, atm-L/mg
H = Henry's constant, atm
MW = molecular weight, g/mol

As previously noted, the solubility of a gas in water is affected by, among other things, the temperature of the system. Henry's constant is most often calculated at 68°F (20°C). In most cases, increasing the temperature of the system decreases the solubility of the gas. The change in Henry's constant with temperature can be calculated based on a van't Hoff type of relationship:

$$\log H = \frac{-\Delta H}{RT} + b$$

where

H = Henry's constant, atm
ΔH = heat of absorption, kcal/kmol
R = gas constant = 1.987 kcal/(K-kmol)
T = temperature, K
b = empirical constant, dimensionless

Values for Henry's constant at 68°F (20°C) and the associated temperature correction factors are provided in Table 9–1 for various gases. Examples 9–1 and 9–2 illustrate the use of Henry's law to calculate the equilibrium concentration of gases in water and an enclosed reservoir. The lower the value of Henry's constant, the more soluble the gas. The polarity and molecular weight of a gas strongly affect its solubility, with more polar and higher-molecular-weight gases being more soluble.

Example 9–1: Solubility of Oxygen in Water *The partial pressure of oxygen in the atmosphere is approximately 0.21 atm. Determine the concentration of oxygen in water at 68°F (20°C) and at 41°F (5°C).*

Table 9–1 shows that, at 20°C, H = 4.3 × 10^4 atm for oxygen. Therefore,

$$C = (0.21\ \text{atm})/(4.3 \times 10^4\ \text{atm}) = 4.9 \times 10^{-6}\ \text{mol O}_2/\text{mol H}_2\text{O}$$

To convert to milligrams O_2 per liter,

$$\begin{aligned}C &= (4.9 \times 10^{-6}\ \text{mol O}_2/\text{mol H}_2\text{O})(55.56\ \text{mol H}_2\text{O/L})(1{,}000\ \text{mg/g})(32\ \text{g/mol})\\ &= 8.7\ \text{mg O}_2/\text{L}\end{aligned}$$

By using the van't Hoff equation, determine the Henry's constant at 5°C. From Table 9–1, $\Delta H = 1.45 \times 10^3$ kcal/kmol and $b = 7.11$.

$$\begin{aligned}\log H &= \frac{-1.45 \times 103\ \text{kcal/kmol}}{(1.987\ \text{kcal/K} - \text{kmol})(278\ \text{K})} + 7.11\\ &= 4.49\end{aligned}$$

$$H = 3.06 \times 10^4\ \text{atm}$$

$$\begin{aligned}C &= (0.21\ \text{atm})/(3.06 \times 10^4\ \text{atm})(55.56\ \text{mol H}_2\text{O/L})(1{,}000\ \text{mg/g})(32\ \text{g/mol})\\ &= 12.2\ \text{mg O}_2/\text{L}\end{aligned}$$

Example 9–2: Hydrogen Sulfide in an Enclosed Reservoir *Hydrogen sulfide exists in groundwater at concentrations as great as 2 mg/L. Determine the concentration of H_2S in the headspace of an enclosed reservoir at 68°F (20°C).*

Table 9–1 shows that at 20°C, $H = 5.15 \times 10^2$ atm for H_2S. First, convert the concentration in milligrams per liter to a mole fraction:

$$\begin{aligned}C &= (2\ \text{mg/L})/[(55.56\ \text{mol H}_2\text{O/L})(1{,}000\ \text{mg/g})(34\ \text{g/mol H}_2\text{S})]\\ &= 1.1 \times 10^{-6}\ \text{mol H}_2\text{S/mol H}_2\text{O}\end{aligned}$$

$$\begin{aligned}P &= (1.1 \times 10^{-6}\ \text{mol H}_2\text{S/mol H}_2\text{O})(5.15 \times 10^2\ \text{atm})\\ &= 5.5 \times 10^{-4}\ \text{atm}\\ &= 550\ \text{ppm}_v\ \text{(parts per million on a volume basis)}\end{aligned}$$

Note: The Occupational Safety and Health Administration's peak 10-min exposure limit is 50 ppm_v.

TABLE 9–1. Henry’s Law Constants for Various Gases

					Temperature Correction Factors	
Gas	Reference	Molecular Weight (g/mol)	H (atm at 20°C)	H^* ([atm-L]/mg at 20°C)	ΔH ([kcal/kmol] $\times 10^{-3}$)	b
Volatile organic compounds						
1,1,1-Trichloroethane	1	133.4	2.00×10^{2}	2.70×10^{-5}		
1,1,2-Trichloroethane	1	133.4	4.27×10^{1}	5.76×10^{-6}		
1,1-Dichloroethylene	1	96.9	2.83×10^{1}	5.26×10^{-6}		
1,2-Dichloroethane	1	99.0	6.33×10^{1}	1.15×10^{-5}		
1,2-Dichloropropane	1	113.0	1.53×10^{2}	2.43×10^{-5}		
1,2,4-Trichlorobenzene	2	181.4	7.89×10^{1}	7.80×10^{-6}		
Benzene	2	78.1	2.40×10^{2}	5.52×10^{-5}	3.68	8.68
Carbon tetrachloride	2	153.8	1.29×10^{3}	1.51×10^{-5}	4.05	10.06
Chlorobenzene	1	112.6	2.06×10^{2}	3.29×10^{-5}		
cis-1,2-dichloroethylene	1	96.9	2.27×10^{2}	4.21×10^{-5}		
Dichloromethane	1	84.9	1.69×10^{2}	3.58×10^{-5}		
Ethylbenzene	1	106.2	4.68×10^{2}	7.94×10^{-5}		
Ortho-dichlorobenzene	1	147.0	9.45×10^{1}	1.16×10^{-5}		
Perchloroethylene	2	165.8	1.10×10^{3}	1.19×10^{-4}	4.29	10.38
Toluene	1	92.1	3.58×10^{2}	6.99×10^{-5}		

trans-1,2-dichloroethylene	1	96.9	2.25×10^{2}	4.18×10^{-5}		
Trichloroethylene	2	131.5	5.50×10^{2}	7.46×10^{-5}	3.41	8.59
Vinyl chloride	2	62.5	3.55×10^{5}	1.02×10^{-1}		
Methyl tertiary butyl ether	3	88.1	3.3×10^{1}	6.8×10^{-6}		
Other compounds						
Oxygen	2	32.0	4.3×10^{4}	2.42×10^{-2}	1.45	7.11
Nitrogen	2	28.0	8.6×10^{4}	5.53×10^{-2}	1.12	6.85
Ozone	2	48.0	8.6×10^{4}	1.87×10^{-3}	2.52	8.05
Carbon dioxide	2	44.0	1.51×10^{3}	6.17×10^{-5}	2.07	6.73
Chlorine	2	70.9	5.85×10^{2}	1.49×10^{-4}	1.74	5.75
Chlorine dioxide	2	86.9	5.4×10^{1}	1.44×10^{-5}	2.93	6.76
Sulfur dioxide	2	64.0	3.8×10^{1}	1.07×10^{-5}	2.40	5.68
Ammonia	2	17.0	7.6×10^{-1}	8.03×10^{-7}	3.75	6.31
Methane	2	16.0	3.8×10^{4}	4.26×10^{-2}	1.54	7.22
Hydrogen sulfide	2	34.0	5.15×10^{2}	2.72×10^{-4}	1.85	5.88
Chloroform	2	119.4	1.70×10^{2}	2.55×10^{-5}	4.00	9.10
Bromoform	2	252.7	3.50×10^{1}	2.40×10^{-6}		

* Henry's constant at 25°C.

Kinetics

Over the past century, several models have been proposed to explain gas transfer theory. Of these models, the two-film model proposed by Lewis and Whitman in 1924 is the simplest and is referred to the most frequently.[4] Two other models, the penetration model by Higbie and the surface renewal model by Danckwertz, have also been used to explain gas transfer theory.[5,6]

The rate of mass transfer of a volatile substance from water to air is generally proportional to the difference between the concentration of the contaminant in solution at the system temperature, as defined by Henry's law. The relationship is expressed as follows:

$$M = K_L a(C^* - C)$$

where

M = rate of mass transfer, lb/hr/ft^3 (kg/hr/m^3)
$K_L a$ = overall mass transfer coefficient, hr^{-1}
C^* = equilibrium concentration of the gas in the liquid, lb/ft^3 (kg/m^3)
C = bulk liquid-phase concentration, lb/ft^3 (kg/m^3)

The driving force for the mass transfer is the difference between the equilibrium and bulk liquid-phase concentrations for the gas. As mentioned earlier, the equilibrium conditions are defined by Henry's law for most drinking water applications. The overall mass transfer coefficient, $K_L a$, is a function of the gas, the process used for gas transfer, and physical parameters, such as temperature and dissolved solids. For most drinking water applications, $K_L a$ is controlled by the liquid-phase resistance. Therefore, gas transfer processes should be designed to maximize the liquid film mass transfer rate.

UNIT PROCESSES

The design and operation of aeration processes are well established. Aeration—or air stripping, as it is sometimes called—can provide treatment at low costs and with few operation and maintenance (O&M) requirements; hence, it is particularly attractive to small communities. The most common application of aeration in water treatment is the removal of gases—including hydrogen sulfide, methane, volatile organic compounds, carbon dioxide, and radon—from groundwater. It is also used for the oxidation of iron and manganese, enabling their subsequent removal by filtration. Aeration provides taste-and-odor removal and is gaining recognition as an effective corrosion control strategy.

In all aeration or air-stripping processes, an air-water interface is created across which mass transfer of a compound can occur. There are three general categories of aeration:

- Diffused aeration, which involves the injection of air into water
- Spray aeration, which involves the injection of water into air

- Waterfall aeration, which involves the cascading of water over media, forming droplets or thin films of water to contact with air

Overview of Design Considerations

Key factors affecting the removal of a compound from water by aeration include:

1. Compound and physical system factors
    - Physical and chemical characteristics of the compound
    - Temperature of the water and the air
2. Aeration process factors
    - Air-to-water ratio
    - Contact time
    - Available area for mass transfer

The ability of a compound to be removed is governed by the compound's Henry's law constant. In general, the higher the Henry's law constant, the less soluble the compound and the more easily the compound is removed via air stripping. (Recall that Table 9–1 lists the Henry's constants for the most common compounds removed via air stripping.)

An aeration system should be designed for the coldest water and air temperatures expected to occur during the course of treatment. Most groundwater supplies exhibit a water temperature of about 55°F (13°C). However, some northern-latitude groundwaters may be as cold as 45°F (6°C), and others may be as warm as 75°F (24°C).

General water parameters give an indication of operational constraints that may be encountered with the aeration of a particular water, as well as the suitability of a water for the air-stripping process. Water quality parameters that may influence the aeration process include hardness, iron and manganese, CO_2, and dissolved oxygen (DO) levels. The most frequent operational constraint associated with aeration is the buildup of scale and slimes. Iron, manganese, and calcium carbonate can oxidize and subsequently precipitate on packing material and can plug diffusers and air jets. Scaling from calcium carbonate precipitation is particularly a problem with hard waters that have moderate to high alkalinity. High iron levels may lead to biological fouling as a result of iron bacteria growth.

Waters with very low dissolved inorganic carbon (DIC) levels (below 2–5 mg/L) may not be suitable for aeration. Aeration will remove CO_2, thereby reducing the DIC levels below the recommended minimum of 2 mg/L for adequate buffering capacity.[7]

Packed Tower Aeration

General Description In packed tower aeration, influent water enters the top of a tower, then flows downward by gravity through packing material. Air is blown upward through the tower in a forced or induced draft (countercurrent flow). The off-gas is either treated or vented to the atmosphere.

The most common application of packed tower aeration in water treatment is VOC removal from groundwater. The process provides a high air:water ratio and the highest percent removal of the air-stripping techniques (in some cases greater than 99 percent).

Packed tower aeration allows treatment of the off-gas. Typical packed tower installations for VOC removal may be as tall as 35 ft (10.7 m), which may be an aesthetic concern in some areas.

Configuration and Components Figure 9–1 is a schematic of typical packed tower facility. The major process elements of packed tower aeration include the tower, blower, and chemical cleaning package.

Tower. The tower shell is commonly fabricated of fiberglass-reinforced plastic, although aluminum, stainless steel, and concrete may also be used. Within the column are support plates for the packing material, a mist eliminator, and liquid distributors. The tower design should include provisions (access ports and possibly ladders and platforms) for physical inspection of tower internals and packing media.

The liquid distributors separate the influent into many smaller streams. A uniform distribution of the water through the column is fundamental to ensure good performance. Inadequate distribution of the influent leads to short-circuiting and lowered removal efficiency. Common types of liquid distributors are trough-type distributors, spray nozzles, and orifice plates (Fig. 9–2).

Trough-type distributors have troughs with V-shaped notches along the sides for distributing the water. They have a low fouling potential and can convey a relatively high flow rate, but they are very level sensitive, requiring perfectly level columns for optimum operation.

Spray nozzles avoid the leveling problems associated with trough-type distributors. However, they are more susceptible to fouling and experience higher head loss through the nozzles.

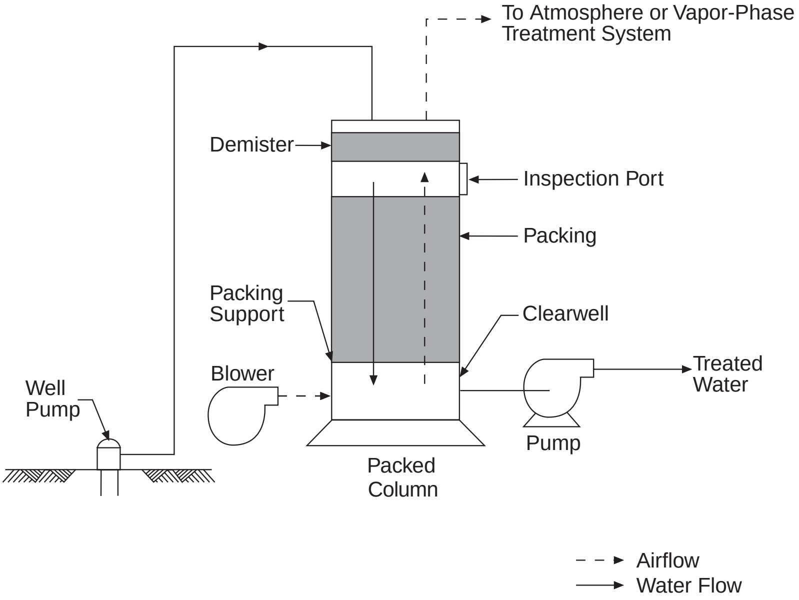
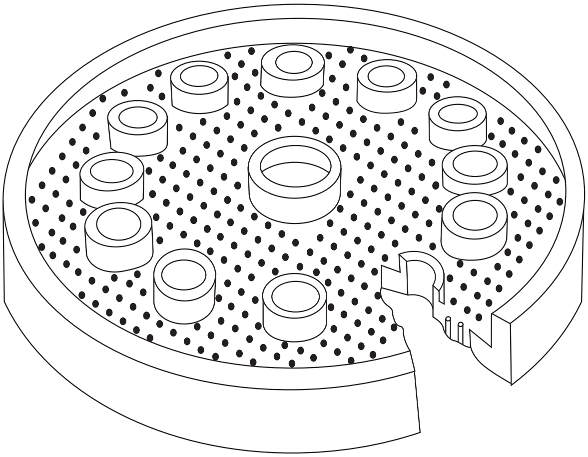
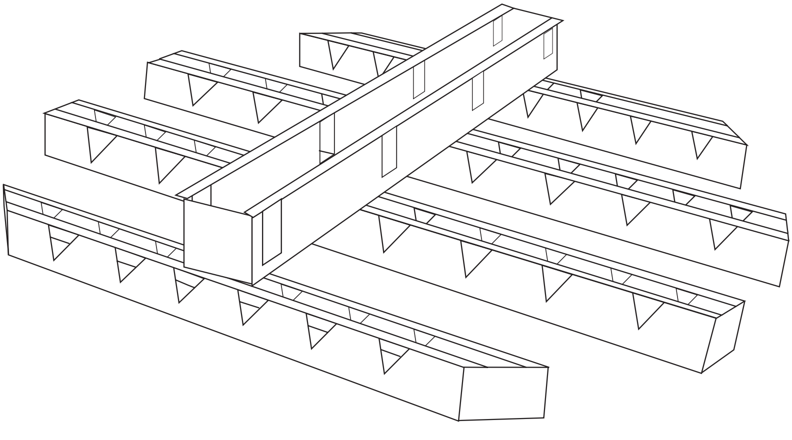


Fig. 9–1. A packed tower aeration system (Adapted from *Proceedings of 1995 AWWA Annual Conference,* by permission. Copyright © 1995, American Water Works Association.)



Orifice-type Distributor



Trough-type Distributor

Fig. 9–2. Examples of liquid distributors

Orifice plate distributors typically have round or rectangular chimneys with a flat floor sealed to the column support ring. The distributors can confine the bed, thereby eliminating the need for bed limiters. The distributors are more susceptible to fouling than trough-type distributors, given the small orifices through which the liquid must pass.

Two types of packing materials are used: randomly placed material and prefabricated sheets. Random packing material may be plastic, stainless steel, or ceramic and is available in various geometrical shapes, as shown in Figure 9–3. Prefabricated sheets

Image Not Available

Fig. 9-3. Examples of packing material for air stripping towers (From Perry, R. H., and Chilton, C. H. (eds.), *Chemical Engineer's Handbook,* 5th ed., McGraw-Hill, New York, 1973. Reproduced with permission of The McGraw-Hill Companies.)

are generally more expensive but are claimed to have a higher transfer efficiency. The packing material should be resistant to corrosion, fouling, and fracturing.

Blower. The air-to-water ratio determines the required size of the blower. Air supplied to the tower should be filtered to remove dust, insects, and any other unwanted materials, which could contribute to tower fouling or biological growth. Air filters should be checked and replaced regularly to avoid a reduction in process efficiency.

Fouling. Fouling of the media and other tower parts can be caused by scale buildup from the oxidation and subsequent precipitation of minerals and by biological slime growth. A cleaning system to periodically recirculate a cleaning solution over the packing material helps to prevent this problem. Generally, a dilute sodium hypochlorite solution is used to prevent fouling of the column due to slime (biological) growth. A dilute hydrochloric acid solution is used to remove scale buildup. In very hard waters, a chemical pretreatment system may be used to inject polyphosphate into the influent raw-water line. The polyphosphate sequesters cations, such as Ca^{+2} and Mg^{+2}, in the water to prevent their precipitation.

Other Considerations. Other features that may need to be considered in an aeration process include:

- A building to accommodate the blower, chemical feed system, piping, valves, ventilation, and dehumidification equipment
- Insulation and heat tracing of the tower sump and piping to prevent freezing in very cold climates
- Off-gas treatment, depending on air emission requirements
- Post-treatment of the effluent to reduce the corrosivity resulting from the increased dissolved oxygen acquired from the aeration process
- Miscellaneous components, including a vented clearwell, raw-water holding tank, noise abatement equipment, and booster pumps

Design Process The design of countercurrent packed towers has been well developed in the chemical process industry.[8,9] A number of researchers have developed design procedures for using packed towers to remove VOCs from groundwater.[2,10,11,12]

More recent research has applied these procedures for the removal of radon.[13,14] This section discusses considerations associated with packed towers. For a more detailed, step-by-step discussion of design procedures, the reader is referred to the literature just cited.

A process schematic for a countercurrent tower with cross-sectional area A and depth of packing Z is shown in Figure 9–4. Water containing a high level of contaminant (concentration C_{in}) enters the top of the tower and flows downward at a superficial velocity $V_L = Q_L/A$ (where Q_L = volumetric liquid flow rate), exiting at the bottom with a low concentration (C_{out}). Correspondingly, forced air containing little or no contaminant (partial pressure P_{in}) enters the bottom of the tower and travels upward at a superficial velocity $V_G = Q_G/A$ (where Q_G = volumetric gas flow rate), exiting the top of the tower with a higher level of contaminant or partial pressure (P_{out}).

The steady-state mass transfer equation can be solved for the case of dilute solutions (for which Henry's law is valid). This solution leads to a packed tower design relation stating that the depth of packing, Z, required to achieve a desired removal performance is the product of the number of transfer units (NTUs) and the height of a transfer unit (HTU).[2] The HTU reflects the rate of mass transfer for a particular packing material and contaminant, whereas the NTU is a measure of the overall mass transfer driving force.

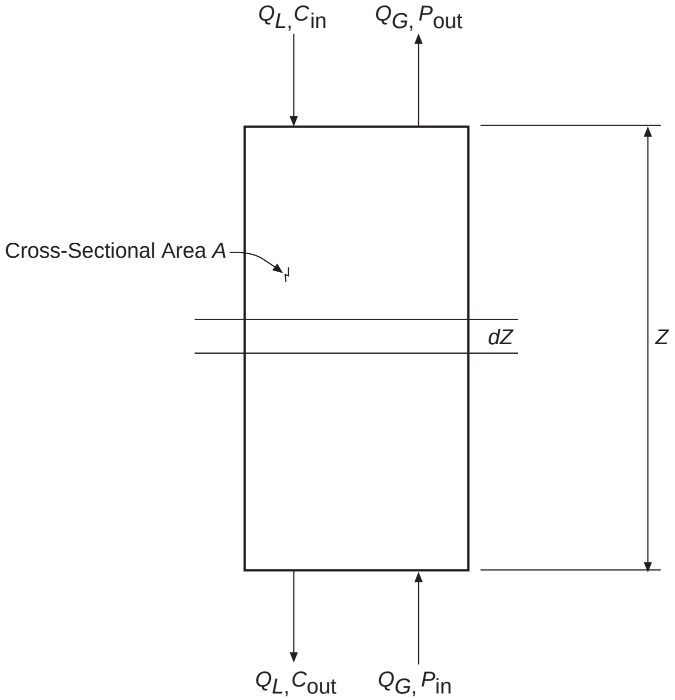


Fig. 9-4. Process schematic for countercurrent packed tower (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

$$Z = \text{NTU} \times \text{HTU}$$

The NTU term characterizes the difficulty in stripping a compound to a desired level; it is given by the following equation:

$$\text{NTU} = \frac{R}{R - 1} \times \ln \left[\frac{(C_{\text{in}}/C_{\text{out}})(R - 1) + 1}{R} \right]$$

where

NTU = number of transfer units, dimensionless
R = stripping factor, dimensionless = HG/L
H = Henry's constant, atm
G = superficial molar air flow rate, $\text{m}^3/\text{m}^2 \cdot \text{h} \cdot \text{atm}$
L = superficial molar liquid flow rate, $\text{m}^3/\text{m}^2 \cdot \text{h}$

NTU is dependent on the desired removal efficiency, the air-to-water ratio, and Henry's constant. Given a specific Henry's constant and a desired removal efficiency, NTU can be computed for a packed column for a given stripping factor or air-to-water ratio. Such a relationship is shown in Figure 9–5.

Optimum column designs are typically based on stripping factors between 1.2 and 5. Figure 9–6 shows the effect on NTU of varying the stripping factor for several removal efficiencies. For 90 percent removal, NTU decreases rapidly for R values slightly greater than 1. Diminishing returns set in as R is increased beyond 2. For very high removals (>90 percent), a stripping factor between 2 and 5 may provide the most economical design.[2]

HTU characterizes the efficiency of mass transfer from water to air and is a function of the liquid loading rate and K_La, the overall mass transfer coefficient. It is defined as follows:

$$\text{HTU} = \frac{L}{K_L a C_0}$$

where

HTU = height of transfer unit, ft (m)
C_0 = molar density of water, lb-mol/ft^3 (kg-mol/L)
L = superficial molar liquid flow rate, $\text{lb-mol/(ft}^2\text{–hr)}$ $\text{(kg-mol/(m}^2\text{–hr))}$
K_La = overall mass transfer coefficient, hr^{-1}

Computing HTU requires data on the mass transfer coefficients for the system under consideration. In some cases, packing manufacturers will supply mass transfer data for air–water systems as a function of temperature and liquid flow rates. It is preferable for values of K_La to be determined from pilot studies utilizing the contaminated water. However, in the absence of such data, mass transfer correlation data from the literature

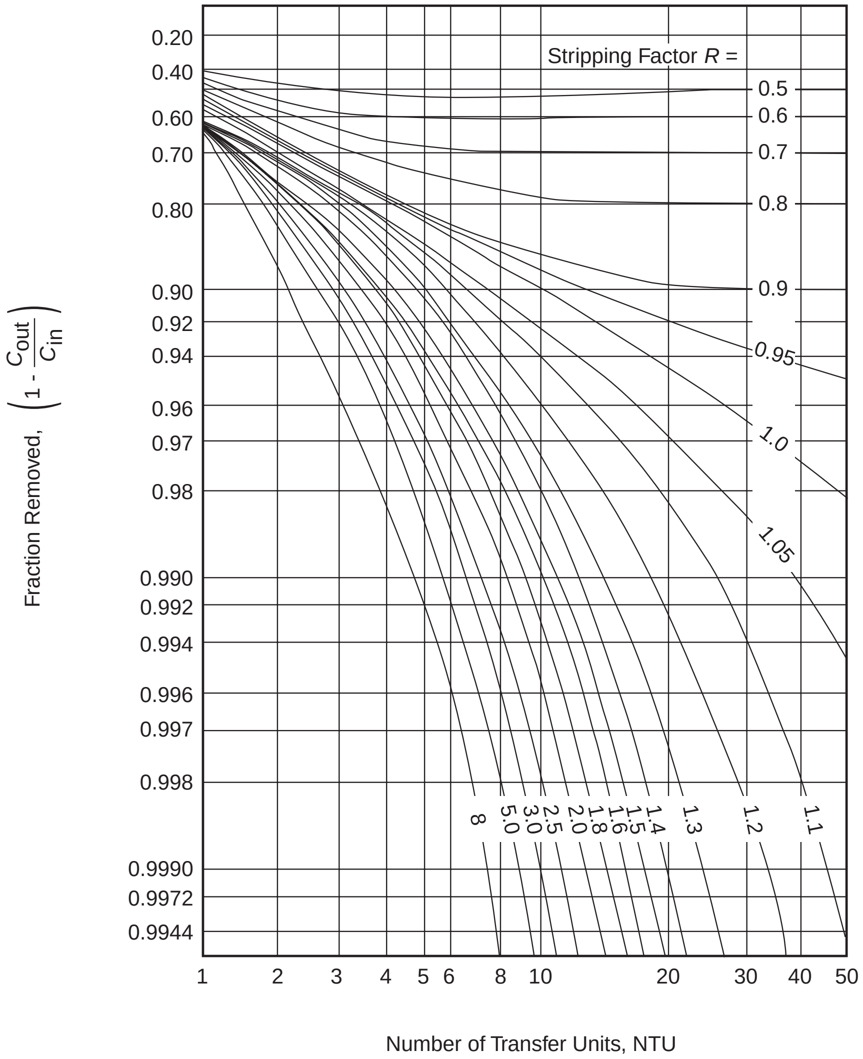


Fig. 9-5. Dependence of number of transfer units on removal efficiency and stripping factor (Reprinted from *Journal AWWA*, Vol. 73, No. 12 (December 1980), by permission. Copyright © 1980, American Water Works Association.)

can be used for preliminary design estimates. A typical empirical correlation used for liquid-phase mass transfer coefficients in towers containing randomly packed materials is the Sherwood-Hollaway correlation:[9]

$$\frac{K_L a}{D_A} = \alpha \left(\frac{L'}{\mu_L}\right)^{1-n} \left(\frac{\mu_L}{\rho_L D_A}\right)^{0.5}$$

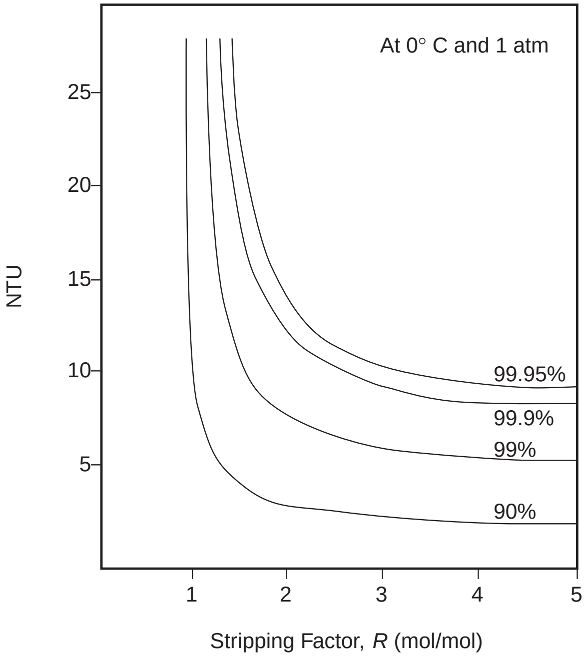


Fig. 9-6. Effect of increasing stripping factor on NTU for various removal efficiencies (Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association.)

where

D_A = molecular diffusion coefficient of the compound to be removed (solute A) in water, ft^2/hr or m^2/h
α = constant
n = constant
μ_L = liquid viscosity, lb/(ft-hr) or kg/(m-h)
ρ_L = liquid density, lb/ft^3 or kg/m^3
L' = liquid mass flux rate, $lb/(ft^2\text{–}hr)$ or $kg/(m^2\text{–}h)$
K_La = overall mass transfer coefficient, hr^{-1}

The empirical constants n and α depend on the type and size of the packing. The constant n ranges from 0.2 to 0.5, and α ranges from 20 to 200. Other correlations may be found in the mass transfer literature.

Key Design Parameters Typical design values for packed tower aeration systems are listed in Table 9–2. The key design parameters that will affect removal of a particular compound include the height and number of transfer units (packing material depth) and the air-to-water ratio. Again, these factors will depend on packing material characteristics, volatility of the compound, air and water temperatures, and the gas and liquid flow rates.

TABLE 9–2. Typical Design Values for Packed Tower Aeration

Design Parameter	Value
Hydraulic loading rate	25 to 30 gpm/ft^2 (61 to 73 m/h)
Tower diameter	3 to 10 ft (0.9 to 3.0 m)
Tower height	15 to 30 ft (4.6 to 9.1 m)
Packing depth	10 to 20 ft
Air-to-water ratio	Typically 30:1 to 40:1, but may range up to 100:1
Gas pressure drop	0.25 to 0.5 in. H_2O per foot of packing depth (200 to 400 N/m^2 per meter of packing depth)

Source: Reference 15. (Adapted from *Proceedings of 1995 AWWA Conference,* by permission. Copyright © 1995, American Water Works Association.)

Figure 9–7 demonstrates the effect of the type of VOC on the packing depth and air-to-water ratio. A ratio of about 10:1 is required to achieve 95 percent removal of trichloroethylene (TCE) with 15 ft (4.6 m) of 1-in. (2.5-cm) packing medium. For 95 percent removal of 1,2-dichloroethane, a less volatile compound, an air-to-water ratio of about 120:1 is required for the same column design.

Figure 9–8 illustrates the relationship between the air-to-water ratio and packing depth to achieve various efficiencies for removal of VOCs (in this case TCE). As illustrated, to achieve 80 percent removal of TCE with an air-to-water ratio of 20:1 would require a column roughly 6 ft (1.8 m) high when 1-in. (2.5-cm) packing medium is used. By contrast, attaining 99 percent removal of TCE at the same air-to-water ratio requires a roughly 20-ft high (6.1-m)-high column.

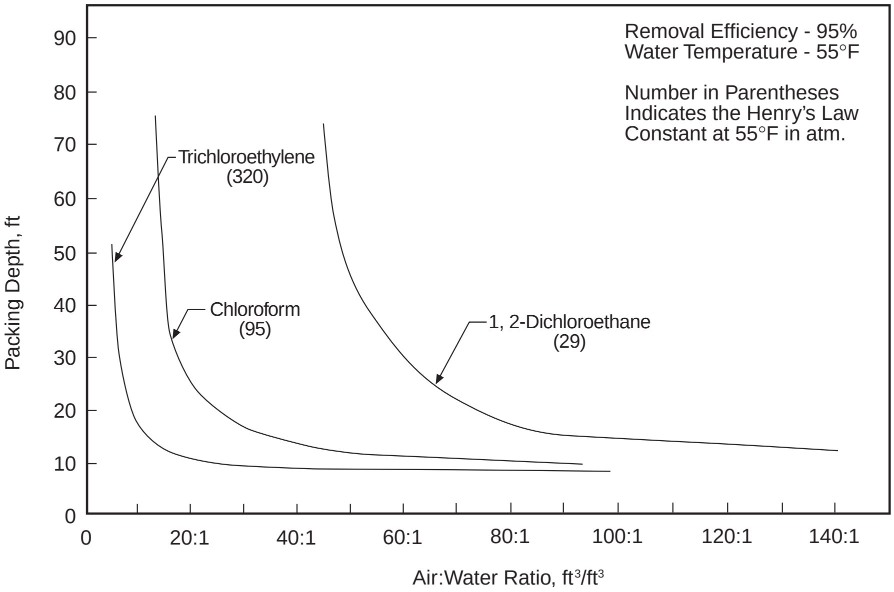


Fig. 9–7. Effect of type of VOC on packed column design (Reprinted from *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* by permission. Copyright © 1983, American Water Works Association and the American Water Works Research Foundation.)

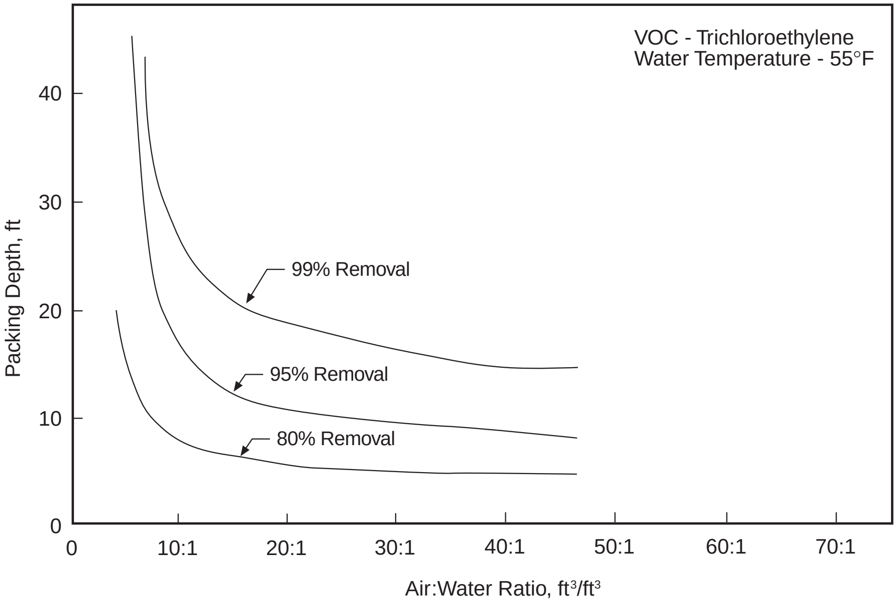


Fig. 9-8. Effect of removal efficiency on packed column design (Reprinted from *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* by permission. Copyright © 1983, American Water Works Association and the American Water Works Research Foundation.)

Figure 9–9 illustrates the relationship of packing depth and air-to-water ratio to achieve 95 percent removal of a VOC (again, TCE) for various water temperatures. At a 20:1 air-to-water ratio, a packing depth of about 8 ft (2.4 m) is required for 75°F (24°C), whereas a packing depth of about 16 ft (4.9 m) is required for 40°F (4°C). In practice, an air-to-water ratio of 30:1 to 40:1 is generally considered sufficient for 95 percent or greater removal of TCE.

It is important not to utilize an overly high air-to-water ratio. A ratio greater than is necessary to achieve the requisite treatment can lead to high operating costs from an oversized blower, flooding due to a sudden increase in gas pressure drop, and mist carryover in the off-gas.

The packing height will dictate the height of the tower. Structural considerations in tower design include the seismic zone, wind speed, and hydraulic load on the tower. The hydraulic load also determines the tower diameter. Overestimating the column diameter results in higher capital and operating costs. Underestimating the diameter results in inadequate handling of the required flow or flooding.

The packing material is designed to simultaneously minimize the pressure drop across the column and maximize the air-to-water contact area through high void volumes, high surface area, and even distribution of liquid and vapor over the cross section of the tower. The shape, material of construction, and nominal packing size are characteristics of concern in selecting the appropriate packing material. Plastic is the most common material for water treatment applications because of its low density, low cost, and high durability. Media with smaller nominal size offer a higher mass transfer coefficient but a higher resistance to air flow.

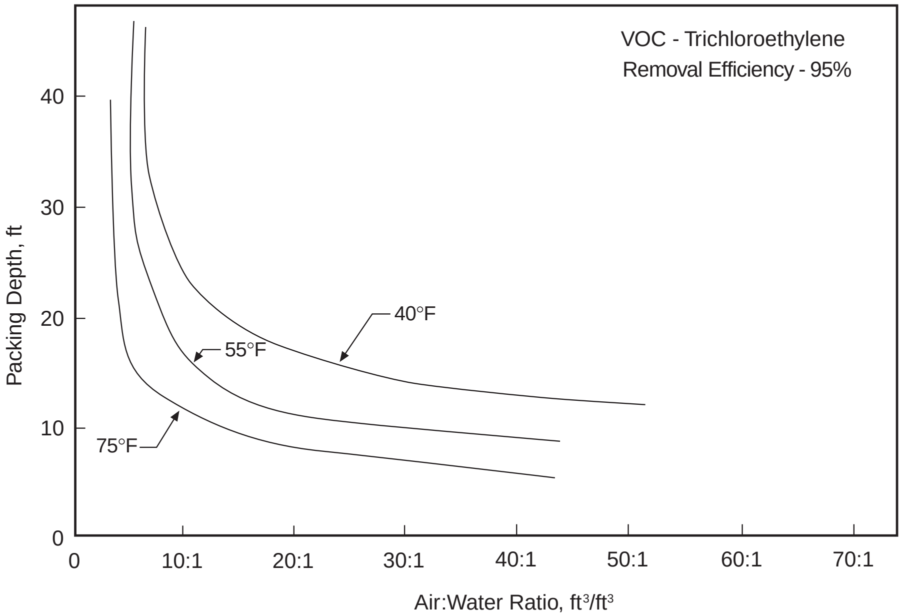


Fig. 9–9. Effect of water temperature on packed column design (Reprinted from *Prediction of GAC Performance Using Rapid Small-Scale Column Tests,* by permission. Copyright © 1989, American Water Works Association and the American Water Works Research Foundation.)

Operation and Maintenance By and large, packed towers operate automatically, with minimal maintenance. However, daily visits are necessary to verify that equipment is running properly. Maintenance includes servicing of pumps and blower motors and the replacement of air filters on the blowers. Even with rigorous cleaning, packed material eventually needs replacement.

Operational difficulties that may be encountered include biological fouling and the precipitation of iron and calcium carbonate compounds on the packing material. These problems can be minimized by pretreatment of the influent or by use of a cleaning package. The frequency of cleaning depends on the influent water quality. A polyphosphate agent may be added to the tower influent to minimize deposition on the packing material.

Multiple-Tray Aeration

General Description Multiple-tray aerators consist of a series of trays with slatted, perforated, or wire-mesh bottoms. The water is distributed evenly over the top, flows from tray to tray, and is collected in a basin at the bottom. Coarse media can be placed in the trays to increase gas transfer effectiveness. Applications for multiple-tray aeration include removal of CO_2, hydrogen sulfide, and (to a lesser extent) taste and odor, as well as the addition of oxygen for iron and manganese oxidation. Figure 9–10 shows a diagram of a typical multiple-tray aeration system.

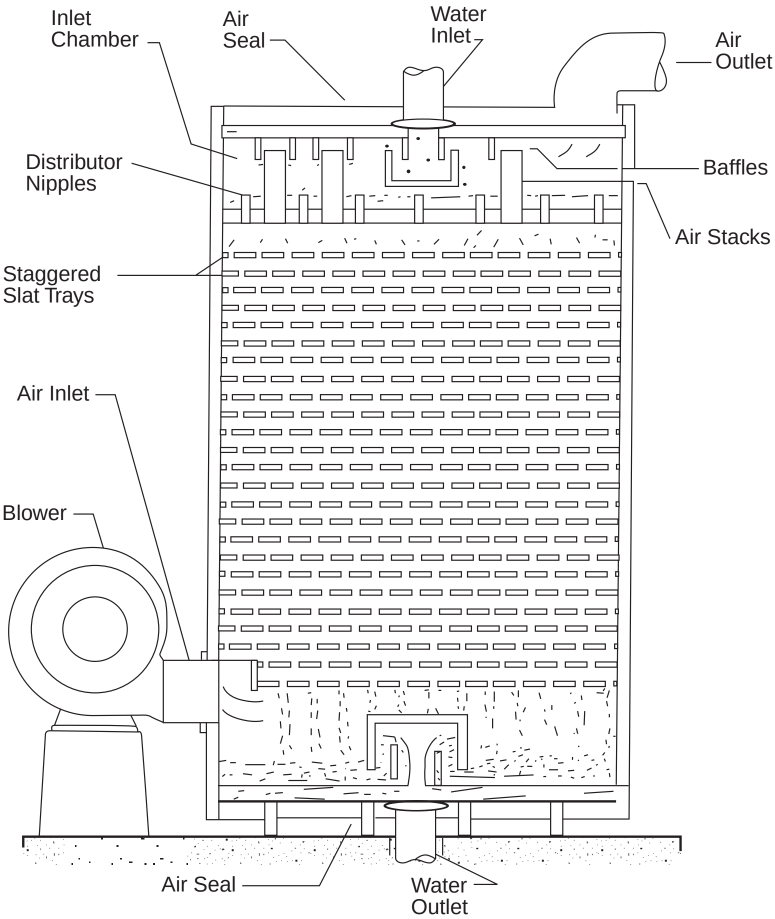


Fig. 9–10. Multiple tray aeration system

Design of Multiple-Tray Aerators A multiple-tray aeration system is often constructed in the form of a packaged unit. The components of the system include trays, housing, and ventilation.

Designs of multiple-tray aeration systems are commonly based on the experiences of similar utilities and on manufacturers' recommendations. Typical design values for tray aeration are listed in Table 9–3. Multiple-tray aerators are generally housed, especially in colder climates.

Removal of a contaminant is increased by increasing the number of trays and by increasing the airflow through the system. Ventilation is an important design consideration. Either sufficient natural ventilation or a supplemental air source is required. Blowers are required for housed systems with inadequate natural ventilation. A blower introduces air from underneath the trays; the air flow is countercurrent to the water flow.

Other design considerations include algae and slime growth problems and corrosion of the trays. Slime and algae growths are controlled with chlorine or copper sulfate.

TABLE 9–3. Typical Design Values for Tray Aeration

Design Parameter	Value
Hydraulic loading rate	14 to 28 gpm/ft^2 (34 to 68 m/h)
Number of trays	3 to 9
Tray spacing	12 to 30 in. (0.30 to 0.76 m)
Tray area	25 to 50 ft^2 per mgd of capacity (0.61 to 1.2 m^2 per ML/d of capacity)
Total height	12 to 16 ft (3.6 to 4.8 m)

The trays must be constructed of corrosion-resistant materials, such as stainless steel, aluminum, rot-resistant wood, concrete, or plastic. An advantage of multiple-tray aeration over packed tower aeration is that it is less susceptible to clogging from iron and manganese precipitation.

Performance Tray aeration has lower removal efficiencies compared with packed tower aeration. For tray aeration, percent reductions for H_2S and CO_2 are on the order of 50 to 80 percent.[16] The removal of CO_2 by multiple-tray aerators has been approximated by the following equation:[17]

$$C_n = C_{\text{in}} \times 10^{-kn}$$

where

C_n = concentration of CO_2 in water after water has passed through n trays, mg/L
C_{in} = initial concentration of CO_2 in water, mg/L
n = number of trays, including the distribution tray
k = experimental coefficient, which is dependent on ventilation, temperature, turbulence, and other characteristics; varies between 0.12 to 0.16

Cascade Aeration

General Description In cascade aeration (also known as *step aeration*), water flows in a thin sheet over a series of steps or baffles, dropping from one layer to another. A common configuration makes use of a concrete step structure.

Cascade aeration is commonly used for oxygenation and for the removal of carbon dioxide and hydrogen sulfide. For groundwater systems, cascade aeration equipment is often built into the storage reservoir. This approach is common in Florida for H_2S removal.

Design of Cascade Aeration Typical design values for cascade aeration are listed in Table 9–4. Increasing the number of steps increases the exposure time. Baffles may be added to produce turbulence, which increases the air-to-water ratio. In cold climates, housing of the aeration equipment is required.

Performance Cascade aeration is less efficient compared with other aeration technologies. It has been reported to reduce CO_2 concentrations by an amount on the order of 20 to 45 percent.[18]

TABLE 9-4. Typical Design Values for Cascade Aeration

Design Parameter	Value
Head requirements	3 to 10 ft (0.9 to 3.0 m)
Surface area	40 to 50 ft^2 per mgd of capacity (0.98 to 1.2 m^2 per ML/d of capacity)
Total height	12 to 16 ft (3.6 to 4.8 m)

Diffused Aeration

General Description In diffused aeration, air is injected into water in a contact chamber though perforated pipes, porous diffusers, or other impingement devices to produce a multitude of fine bubbles. As the bubbles rise, mass transfer of the contaminant takes place across the water–air interface until the bubbles reach the surface or become saturated with the contaminant. In water treatment, diffused aeration is generally used to aerate source water supply reservoirs. It is also effective for VOC removal.

Diffused aeration offers several advantages. First, it requires no housing. In addition, it has few or no problems with cold-weather operation. Diffused aeration can be added to existing structures, such as storage tanks. If seasonal aeration is needed (e.g., trihalomethane [THM] removal in the summer), economic concerns may favor adding diffused aeration to an existing clearwell. The process has negligible headloss, so no additional pumping would be required.

Disadvantages of diffused aeration include higher power costs and lower removal efficiencies compared with packed tower aeration. Diffusers are susceptible to fouling by iron and hardness. Precipitation on the diffusers may require frequent replacement of the diffusers, routine cleaning, or pretreatment to sequester or remove the iron before it enters the aeration unit. Figure 9–11 shows a schematic of a typical diffused aeration system.

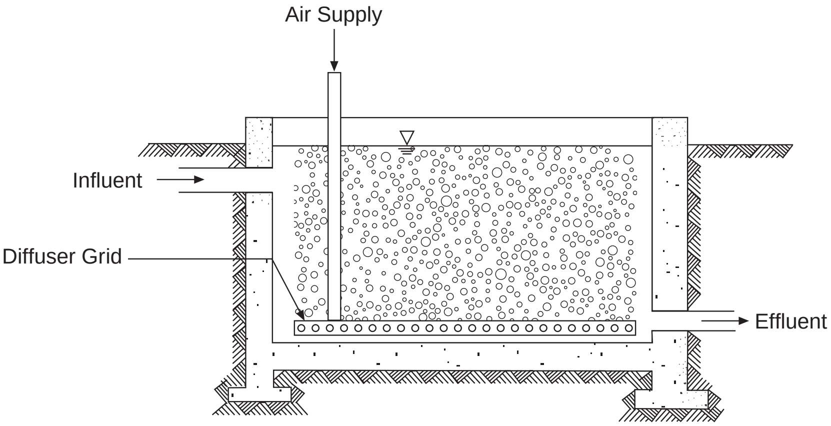


Fig. 9-11. Schematic of diffused aeration

Design Contact chambers are usually rectangular concrete tanks. Most often, the system is operated with countercurrent flow, with water entering the top and exiting at the bottom while exhausted air exits at the top.

Typical diffusers include perforated pipes, porous plates or tubes, and patented impingement or sparger devices. The diffusers are located along the bottom of the tank or are placed along one side of the tank to impart a spiral flow. A spiral flow allows higher velocities to promote gas transfer. Porous tubes or plates are often located mid-depth in the tank to reduce air compression head. Filtration of the air may be necessary to avoid clogging.

Typical design ranges for diffused aeration systems are listed in Table 9–5. The aim of the diffused aeration system is to distribute air uniformly through the water cross section and to produce the desired air bubble size. The removal efficiency of diffused aeration for stripping compounds can be improved by any of the following actions:

- Decreasing the bubble size. Finer bubbles are more efficient than coarse bubbles because of their greater interfacial area; however, finer bubbles are more expensive to produce and may coalesce into large bubbles, which would eliminate any advantage.
- Increasing the water depth
- Increasing detention time
- Increasing the volumetric air-to-water ratio
- Improving chamber hydraulics to prevent short-circuiting. Lateral baffles may be used to prevent short-circuiting.

After aeration, the water is saturated with dissolved oxygen. The water is therefore sent to a wetwell or inlet chamber configured to allow for the air bubbles to be released.

Performance Table 9–6 presents removal efficiencies data for five VOCs and nine synthetic organic chemicals (SOCs). Removal rates are based on diffused aeration units with air-to-water ratios of 5:1 to 15:1 and contact times of 10 to 15 minutes. The removal rates range from 11 to 95 percent for the 14 compounds.

TABLE 9-5. Typical Design Values for Diffused Aeration

Design Parameter	Value
Tank depth	9 to 15 ft (2.7 to 4.6 m)
Tank width-to-depth ratio for effective mixing	2
Tank length	10 to 30 ft (3.0 to 9.1 m)
Detention time	10 to 30 min
Air required	0.01 to 0.15 ft^3 per gal treated (74.8 to 112.2 m^3 per ML treated)
Power requirements	0.5 to 2.0 kW (typically 1 kW) per mgd of plant capacity (0.13 to 0.52 kW [typically 0.26 kW] per ML/d of plant capacity)

TABLE 9-6. Typical Removal Performance of Diffused Aeration for Various Compounds

Compound	Removal (%)
VOCs	
Trichloroethylene	53–95
Perchloroethylene	73–95
1,2-Dichloroethane	42–77
1,1-Dichloroethylene	97
1,1,1-Trichloroethane	58–90
SOCs	
Carbofuran	11–20
1,2-Dichloropropane	12–79
cis-1,2-Dichlorethylene	32–85
trans-1,2-Dichloroethylene	37–96
o-Dichlorobenzene	14–72
Ethylbenzene	24–89
Monochlorobenzene	14–85
Toluene	22–89
Xylenes	18–89

Source: Reference 19.

Multistaged Bubble Aeration System The multistage bubble aeration system (MSBAS) is a type of diffused aeration system. It is a packaged system constructed of polyethylene or stainless steel units, which are matched with regenerative air blowers.

A schematic of a typical system is shown in Figure 9–12. The system is constructed as a series of stages, each of which is separated by a partition. The water flows by gravity through the units. An air blower is attached to a manifold that spans the length of the vessel. Air is blown into each stage. Individual modules are designed to treat water at flow rates ranging from 1 to 1,500 gpm (0.063 to 94.6 L/s).

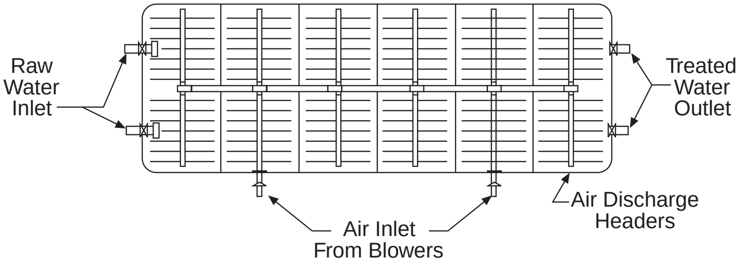
The MSBAS offers advantages over conventional aeration systems. It is more compact; the dimensions of a series of units with a capacity of 1,500 gpm (94.6 L/s) are approximately 12.5 ft (3.8 m) length, 6.5 ft (2.0 m) width, and 6.7 ft (2.1 m) height. In an MSBAS, a much shallower column of water—approximately 1.5 to 3.0 ft (0.46 to 0.91 m)—is aerated, requiring less energy and resulting in a more efficient mass transfer process.[13,20] The shallow depth allows easier accessibility for maintenance.

MSBAS units have been used primarily for VOCs, CO_2, and radon removal. These systems have been found capable of providing greater than 99 percent removal of these constituents.[13,21]

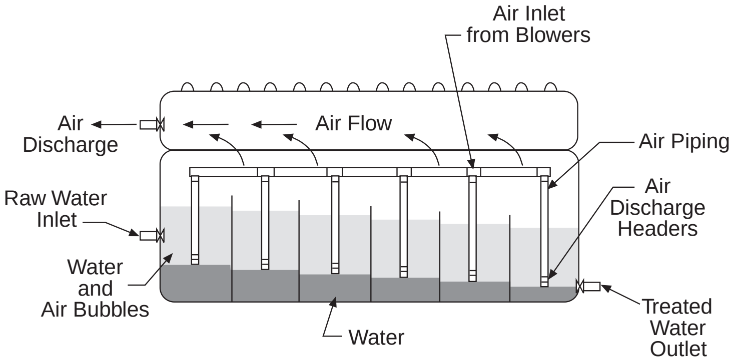
Mechanical Aeration

Mechanical aeration uses a surface or subsurface mechanical stirring mechanism to create turbulence to mix air with the water. Mechanical aeration is extensively used in wastewater treatment. It has limited use for VOC removal in water treatment and is occasionally used as a pretreatment step (for oxygenation) in water treatment.

Mechanical surface aeration is a simple process to build and operate. Some disadvantages of mechanical aeration include long detention time for effective treatment



Top View (Without Lid)



Cross Section (During Aeration)

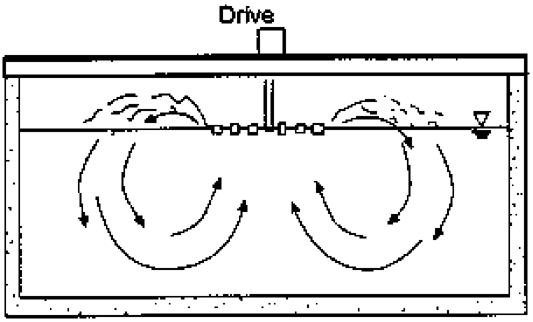
Fig. 9–12. Multistage bubble aeration system (Courtesy of Lowry Systems)

(which means large space requirements), potential freezing problems in cold climates and an inability to control off-gas with the open basins, and high energy requirements. However, a study by Roberts and Levy comparing the energy requirements of mechanical surface aerators, diffused aeration, and packed tower aeration concluded that mechanical aeration and packed tower aeration required approximately the same amount energy to achieve the same efficiency (generally less than 0.0028 kW-h/ft^3 [0.1 kW-h/m^3]).[20]

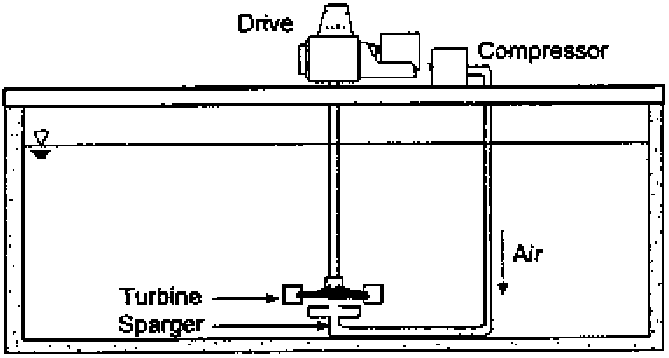
Figure 9–13 shows schematics of two typical mechanical aeration processes. The major components of such a system are the aerator and the basin. Surface aerators are either mounted on fixed platforms or float on the basin surface.

Spray Aeration (Nozzles)

General Description In spray aeration, water is forced through fixed nozzles to form fine droplets. The small droplets that are produced expose a large interstitial surface area through which a compound can migrate from the liquid phase to the gaseous phase. Applications for spray aeration include oxygenation, as well as removal of carbon dioxide, hydrogen sulfide, and taste and odor.



Mechanical Surface Aerator



Submerged Turbine Aerator

Fig. 9–13. Schematic of a mechanical aeration process

The main advantages of spray aeration are that gas transfer between water drops and air is rapid, and the system can be aesthetically pleasing. Disadvantages include potential operating difficulties from icing problems during freezing weather, large space requirements, high discharge head requirements, and potential clogging of nozzle orifices, particularly with hard waters.

Spray aeration typically involves spraying water into the open atmosphere, often into a reservoir or a concrete or earthen basin. Nozzles are often fixed on a pipe grid. They may also be enclosed in a tower, allowing additional mass transfer as droplets descend through a countercurrent airflow.

Two common types of spray nozzles are the hollow cone and the full cone. In a hollow-cone nozzle, a rim of spray is concentrated around the nozzle circumference, with a "hollow" area in the center of the spray jet's cross section. This type of nozzle produces smaller droplets but requires more pressure for the same amount of flow through the nozzle. With full-cone nozzles, the water is delivered uniformly, generally in a round cross-sectional pattern.

Design of Spray Aeration Systems Typical design values for spray aeration are shown in Table 9–7. The size, number, and spacing of spray nozzles depend on the

TABLE 9–7. Typical Design Values for Spray Aeration

Design Parameter	Value
Pressure ranges	1 to 10 psi (6.6 to 70 kPa)
Nozzle diameter	1.0 to 1.5 in. (2.5 to 3.8 cm)
Discharge rates	≈75–150 gpm at ≈10 psi (≈4.7–9.5 L/s at ≈69 kPa)
Nozzle spacing	2 to 12 ft (0.6 to 3.7 m)
Droplet diameter	2 to 10,000 μm
Area required	50 to 150 ft^2 per mgd of capacity

head to be expended, the area allocated to aeration, and the degree of interference between adjacent sprays. Key design parameters for the nozzles include

- K_La (mass transfer values available from manufacturers)
- Droplet diameter
- Time of exposure

The droplet diameter determines the area:volume ratio and is a major controlling factor in the mass transfer rate. It is a function of the dispersing action of the nozzle and generally ranges from 2 to 10,000 μm. A smaller diameter will allow a higher air-to-water ratio. However, the smaller nozzles are more susceptible to clogging, resulting in higher maintenance requirements.

The time of exposure is dependent on the droplet's initial velocity and trajectory. The initial velocity V of a drop emerging from an orifice or nozzle is given by

$$V = C_v\sqrt{2gh}$$

where

V = initial velocity, ft/sec (m/s)
C_v = coefficient of velocity
g = acceleration of gravity, ft/sec^2 (m/sec^2)
h = total head on nozzle, ft (m)

The total discharge Q from the nozzle is given by

$$Q = C_dA\sqrt{2gh}$$

where

Q = nozzle discharge, ft^3/sec (L/sec)
C_d = coefficient of discharge
A = area of nozzle opening, ft^2 (m^2)
g = acceleration of gravity, ft/sec^2 (m/sec^2)
h = total head on nozzle, ft (m)

The theoretical exposure time t of a water drop is given by

$$t = 2C_v \sin \theta \sqrt{\frac{2h}{g}}$$

where

t = time of exposure, sec
C_v = coefficient of velocity
θ = angle between the initial velocity vector and horizontal
h = total head on nozzle, ft (m)
g = acceleration of gravity, ft/sec^2 (m/sec^2)

For the preceding three equations, note that $C_d = C_v C_c$, where C_c is the coefficient of contraction. C_d, C_v, and C_c vary with the shape and other characteristics of the orifice or nozzle.

Performance Spray aerators typically remove in the range of 70 percent of the dissolved CO_2, although removals up to 90 percent have been reported.[18] One experiment observed up to 75 percent radon removal by the use of spray jet aeration; variations in air-to-water ratio had a negligible effect on radon removal efficiency.[13]

Emerging Technology: Gas-Degas Treatment (GDT) Process

The GDT™ (''gas-degas treatment'') process is proprietary and consists of a high-efficiency venturi injector, a reaction vessel, a degassing separator, an auxiliary off-gas discharge valve, and a backpressure control valve (Fig. 9–14). This process has been utilized both for gas transfer into solution and for contaminant stripping.

In the case of gas transfer into solution, the system is held at a high pressure, and a low volumetric gas-to-liquid ratio (V_g/V_l) is then used to achieve high transfer efficiencies. For oxygen-fed ozone systems, typical operating values include injector outlet pressures in the range of 25–34 psig (170–230 kPa gauge) and V_g/V_l ratios of 0.1.[22] Transfer efficiencies of 88 to 95 percent are possible under these conditions.

For contaminant stripping, high V_g/V_l ratios are used in combination with low-pressure systems, as well as with recirculation of the liquid stream in certain cases, to achieve the desired contaminant removal efficiencies. Single-pass systems are used for more easily stripped gases, such as radon, whereas recirculating systems are used for VOCs that are not as readily removed. Typical operating values include V_g/V_l ratios of 1 to 4 and injector pressures of 0 to 2 psig (0 to 14 kPa gauge).[23] The systems themselves are designed for nominal flow rates of 10 to 3,000 gpm (0.63 to 190 l/s).

The primary advantage of the GDT process is the relatively small footprint of the equipment. This feature enables the systems to fit into existing structures or other places with site constraints. For example, a 250–400-gpm (16–25-l/s) unit is approximately 2 ft (0.6 m) in diameter and 6.5 ft (2.0 m) high. A packed tower for a similar application would be approximately 3.5 ft (1.1 m) in diameter and 20 ft (6.1 m) high.

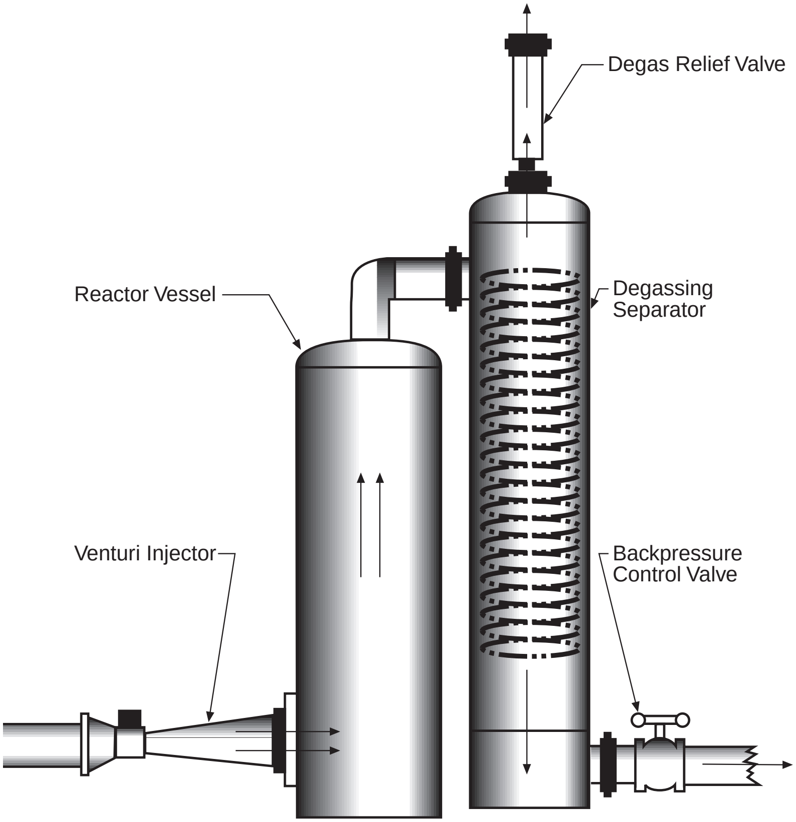


Fig. 9-14. GDT™ process equipment (Courtesy of GDT Process)

Off-Gas Treatment

For some aeration systems, the off-gas may need to be treated before being released to the atmosphere. A recent survey of packed tower installations found that of the seven respondents, none were required to employ off-gas destruction. However, two respondents were planning to install off-gas treatment in the near future—one to comply with regulatory requirements and the other because of public perception.[15] Such treatment can add significant capital costs and operation and maintenance costs.

For a known airflow rate, the mass balance on the emissions rate of a compound (in pounds per hour) is calculated based on the following formula:

$$\text{emissions rate (lb/hr)} = (C_{\text{in}} - C_{\text{out}}) \times Q_L \times K$$

where

C_{in} = influent concentration of the compound, μg/L
C_{out} = effluent concentration of the compound, μg/L

Q_L = volumetric flow rate, gpm (L/sec)
$K = 5.0 \times 10^{-7}$, constant to convert emissions rate to lb/hr
$K = 3.6 \times 10^{-6}$, constant to convert emissions rate to kg/hr

Emissions of the compound are compared with local air quality standards to determine if treatment of the off-gas is necessary. Site-specific factors that may impact the evaluation include:

- Projected ground-level concentrations
- Proximity to human habitation
- Treatment plant worker exposure
- Local air quality
- Local meteorological conditions

The methods for off-gas treatment include:

- Vapor-phase carbon adsorption
- Thermal destruction
- Catalytic incineration
- Ozone destruction with ultraviolet (UV) radiation
- UV photooxidation

Carbon Adsorption The most common method to treat the off-gas is by vapor-phase carbon adsorption. The off-gas is usually processed by a heating step to remove humidity before being applied to the carbon units. Parameters that influence the design of the carbon adsorption system include the airflow rate, influent concentrations of VOCs, and degree of VOC removal by the packed tower unit. The carbon usage rate for vapor-phase treatment of VOCs is generally less than half that for removing VOCs in liquid-phase granular activated carbon (GAC) units. During vapor-phase carbon adsorption, the contaminants are transferred from one phase to another. As with liquid-phase carbon absorption, the exhausted carbon is either regenerated on-site or replaced (see Chapter 17).

Photocatalytic Oxidation Photocatalytic oxidation, which is still an experimental technology for treating off-gas, utilizes ultraviolet (UV) light to oxidize VOCs. Studies to date have utilized the process to treat the off-gas downstream of a packed tower and in a closed-loop air-stripping process (CLASP). Chlorinated ethylenes, such as trichloroethylene (TCE), are more susceptible to photocatalytic oxidation than are chlorinated ethanes, such as trichloroethane (TCA). Water vapor was found to significantly inhibit the VOC destruction rate for TCE.[24] For this reason, photocatalytic oxidation units should be placed downstream of a dehumidifier. The main benefit of operating gas-phase photocatalytic oxidation in a closed-loop process is absorption of the end products into the water flow, which eliminates the release of off-gases to the atmosphere and need for an absorbent, such as granular activated carbon.[25]

Process Selection

All of the aeration techniques discussed thus far have been successfully used to reduce contaminant concentrations in either full-scale or pilot-scale applications. Selection of the appropriate air-stripping option depends on site considerations, the characteristics of the contaminant, the desired removal efficiency, and cost.

Figure 9–15 sets out the circumstances under which the various air-stripping processes are economically feasible. The two principal parameters controlling the selection are Henry's constant and the desired removal percentage. Where removal efficiencies do not have to exceed 90 percent, both diffused aeration and spray towers may be cost-effective alternatives. Assuming that equal power is required for both of these types of systems, the deciding factor is whether Henry's constant is greater or less than about 1,000 atm.

For removals greater than 90 percent, a packed tower is probably the only alternative. The lines in Figure 9–15 that delineate the viable zone for packed towers assume that the height of a transfer unit (i.e., HTU) is approximately 3 ft (1 m) and that the maximum economical depth of packing is approximately 30 ft (9.1 m). For a ratio of local mass transfer coefficients of $k_l/k_g = 0.01$ (where k_l is the liquid-phase resistance and k_g is the gas-phase resistance) and a stripping factor greater than 5, it can be shown that the maximum removal efficiency over a wide range of Henry's law

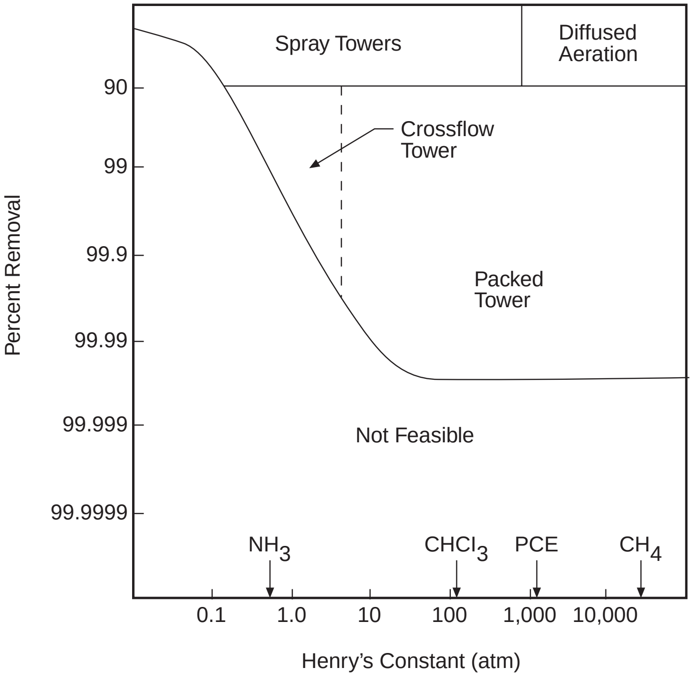


Fig. 9–15. Diagram for selection of feasible air stripping process (Reprinted from *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* by permission. Copyright © 1983, American Water Works Association and the American Water Works Association Research Foundation.)

constants is approximately 99.5 percent.[26] As the Henry's constant decreases, however, the gas-phase resistance (k_g) becomes important and high gas ratios are required for system design. Under these conditions, packed towers become less economical. At some point, only a cross-flow tower—a type of packed column where the gas and water flow at right angles—can handle the high gas flow rates. More detailed descriptions of cross-flow towers can be found elsewhere (e.g., see reference 27).

Others factors that may play a role in aeration technology selection include operational considerations, the water flow rate, and various site conditions (such as space limitations, noise, aesthetics, and air quality requirements).

APPLICATIONS

This section gives more detail concerning specific applications of aeration in the removal of contaminants, including VOCs, radon, H_2S, tastes and odors, and CO_2. It also discusses the use of aeration for iron and manganese oxidation, which is covered in greater detail in Chapter 14.

VOC Removal

Air stripping is the most widely used and usually the most economical technology for removing VOCs from groundwater. Several of the aeration processes discussed thus far have been successfully used to reduce VOC concentrations in either full-scale or pilot-scale applications. Reported results from a few of these applications are presented in Table 9–8.

A 1991 survey of 68 treatment installations that remove VOCs from water cited numerous locations for which removals of 95 percent were achieved by using packed tower aeration.[15] Higher removal efficiency (greater than 99 percent) was achieved through optimum design of the packed column system. The majority of the surveyed facilities removed perchloroethylene (PCE) and TCE and used similar design values. The most common design values found in the survey were as follows:

- Packing height: 15–25 ft (4.6–7.6 m)
- Air-to-water ratio: 25:1 to 50:1
- Column diameter: 5–10 ft (1.5–3.0 m)
- Hydraulic loading rate (HLR): 25–30 gpm/ft^2 (61–73 m/h)

Radon Removal

Aeration has been gaining recognition for its ability to remove radon from groundwater. Air stripping of radon has a distinct advantage over other treatment technologies (such as activated carbon) because it does not generate a waste product requiring specialized disposal, and in only a few instances is it likely that off-gas treatment would be required. Radon can easily be stripped to low levels, as evident in radon's high Henry's constant of 2.26×10^3 atm at 68°F (20°C) and 1 atm—about four times the value for TCE. Most radon-stripping treatment systems reported in the literature (diffused aeration and packed towers) have concentrated on achieving upward of 99 percent removal efficiency. This treatment efficiency is excessive. In most cases, re-

TABLE 9-8. Summary of Water Systems' Experiences with Using Aeration for VOC Removal

Aeration Type and Water System Location	VOC	Air:Water Ratio ([ft^3/min]/[ft^3/min])	Contact Time (min)	Influent Concentration (μg/L)	Removal (%)
Diffused Aeration					
Cincinnati, Ohio	TCE*	4:1	10	241	78
	TCE	8:1	10	241	97
	TCE	16:1	10	241	99
Glen Cove, New York	TCE	5:1 to 20:1	5 to 20	132 to 313	73
Port Charlotte, Florida					
6-ft-deep (1.8-m) basin	THM	7.5:1	6	—	58
	THM	15:1	6	—	73
12-ft-deep (3.7-m) basin	THM	7.5:1	6	—	73
	THM	15:1	6	—	84
	THM	22.5:1	6	—	85
Ft. Lauderdale, Florida	THM	8:1 to 36:1	5 to 10	200 to 250	18 to 39
Multiple-Tray Aeration					
Ft. Lauderdale, Florida	THM	—	—	250 to 280	8 to 12
Camden, New Jersey	TCE	30:1	—	15 to 35	23 to 93
	PCE	30:1	—	6 to 20	18 to 80
Norwalk, Connecticut	TCE	—	—	—	52
	PCE	—	—	—	40
Smyrna, Delaware	TCE	0	—	5 to 70	40 to 60

TABLE 9-8. (*Continued*)

Aeration Type and Water System Location	VOC	Air:Water Ratio ([ft^3/min]/[ft^3/min])	Contact Time (min)	Influent Concentration (μg/L)	Removal (%)
Packed Tower Aeration					
Pelham, Massachusetts	TCEA	5:1 to 60:1	—	42 to 100	88 to 92
	TCEA	3:1 to 114:1	—	630 to 1,200	50 to 70
	TCEA	11:1 to 27:1	—	680	50 to 82
Wausau, Wisconsin	*cis*-1,2-DCE	61:1	—	82.3	96.8
	TCE	61:1	—	72.0	98.0
	PCE	61:1	—	59.6	98.4
	Toluene	61:1	—	30.9	96.9
	Ethylbenzene	61:1	—	5.1	100
	Xylenes	61:1	—	16.6	96.4
	Vinyl chloride	61:1	—	8.8	100
New Haven, Connecticut	TCE	15:1	—	17 to 23	82 to 87
	TCE	25:1	—	19 to 140	85 to 90
	TCE	30:1 to 50:1	—	5 to 135	89 to 94
	TCE	8:1 to 10:1	—	8 to 10	67 to 78
	TCE	19:1 to 26:1	—	126 to 133	76 to 92
	TCE	13:1 to 19:1	—	23 to 34	75 to 87
	PCE	13:1 to 19:1	—	1.7	70+
	PCE	7:1 to 26:1	—	10 to 20	0.5 to 0.7
	PCE	8:1 to 10:1	—	2.4 to 2.7	3 to 5

Source: Reference 26. (Adapted from *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* by permission. Copyright © 1983, American Water Works Association and the American Water Works Association Research Foundation.)

cis-1,2-DCE = *cis*-1,2-dichlorothylene; PCE = perchloroethylene; TCE = trichloroethylene; TCA = 1,1,1-trichloroethane.

*Spiked.

moval efficiencies in the range of 50 to 80 percent will be adequate to achieve even the more stringent of the proposed radon maximum contaminant levels (MCLs). This removal efficiency can likely be achieved by using a naturally ventilated tray aeration system or a sparging device placed in a water storage tank.

H_2S Removal

Aeration is a practical method for H_2S removal for total hydrogen sulfide concentrations of less than 3–4 mg/L. Higher concentrations may require prolonged aeration or an initial aeration period in the presence of an atmosphere with a high partial pressure of CO_2.

Only the H_2S form of sulfur is removable via aeration; the hydrolyzed forms are not affected. Hydrogen sulfide hydrolyzes as follows:

$$H_2S \leftrightarrow H^+ + HS^- \qquad k_1 = 1 \times 10^{-7}\ (25°\mathrm{C})$$

HS^- disassociates as follows:

$$HS^- \leftrightarrow H^+ + S_2^- \qquad k_2 = 1 \times 10^{-14}\ (25°\mathrm{C})$$

The amount of H_2S in water may be calculated by using the following equation:

$$\%H_2S = \frac{100[H^+]^2}{[H^+]^2 + k_1[H^+] + k_1k_2}$$

where

k_1, k_2 = equilibrium constants

The existence of H_2S is pH dependent, as indicated in Figure 9–16. The pH must be below approximately 7.5 to ensure that a high percentage of the sulfur is in the H_2S form and, therefore, available for removal via aeration.

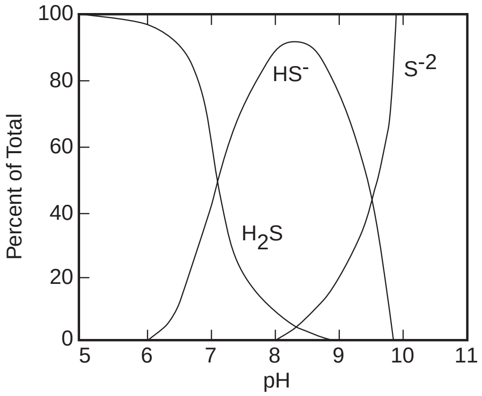


Fig. 9–16. Effect of pH on H_2S-HS^--S^{-2} equilibrium

When water containing both CO_2 and hydrogen sulfide is aerated, the CO_2 is easily removed. Stripping of the CO_2 causes an increase in the pH, which decreases the percentage of total sulfide available as H_2S, thereby lowering the potential for H_2S removal.

During aeration, the hydrogen sulfide may be oxidized to form free sulfur:

$$2H_2S + O_2 = 2H_2O + 2S(s)$$

In this case, the hydrogen sulfide concentration is reduced as the result of a chemical reaction, not by gas transfer. Unless this free sulfur is removed, it may be further oxidized to sulfate and reduced back to sulfide in the distribution system.

Taste and Odor Removal

With the exception of hydrogen sulfide, odor-producing compounds typically are not volatile enough to be effectively removed by aeration. Aeration is of questionable value when used solely for taste and odor control; it is usually used only when oxidation of iron and manganese or removal of CO_2 is also required. However, exceptions have been reported in some of the older literature. During warm months, the use of spray aeration achieved a 44 percent reduction (i.e., reduction of 25 units) in threshold odor number at Appleton, Wisconsin; aeration was less effective during colder months.[28] At Nitro, West Virginia, high-pressure spray aeration was used to effectively reduce initial threshold odor numbers of 5,000–6,000 in Kanwha River water affected by industrial waste discharge.[29] The full-scale operation treated a flow of 4,000 gpm (250 L/s). The system included 30 nozzles (at 55 psi [380 kPa]) arranged on pipes decreasing in diameter from 18 in. to 8 in. High nozzle abrasion rates and poor cold-weather operation were reported.

CO_2 Removal

Aeration can be an effective corrosion control strategy. Aeration reduces CO_2 concentrations, which in turn increases the pH, which subsequently can reduce lead and copper corrosion rates. Aeration has achieved up to 80–90 percent removal of CO_2 for some waters.

Aeration is commonly used for removing CO_2 from groundwater. The suitability of aeration as a corrosion control strategy is highly dependent on the raw-water quality. Aeration is suitable for waters with a low pH (below 7), low hardness, and significant CO_2 concentrations (greater than 10 mg/L). There are many groundwater supplies that match these criteria, and some may in fact have extremely high CO_2 concentrations (greater than 80 mg/L) for which the driving force for CO_2 removal is very high.

Other water quality considerations for selecting aeration as a corrosion control method include the Ca^{2+} concentration and dissolved inorganic carbon (DIC) concentrations. $CaCO_3$ precipitation will lower the final pH that is attainable through aeration, as well as reduce the alkalinity and buffering capacity of the water, both of which are important parameters in corrosion control. A water with a DIC of less than 2 mg/L should not be considered for aeration in order to provide adequate buffering capacity. Lytle et al. provide additional guidance on selecting aeration as a corrosion control strategy.[21]

The change in pH resulting from aeration under a best-case scenario may be approximated from

$$pH_f = pH_i - \log [CO_{2f}/CO_{2i}]$$

where

pH_i = initial pH
pH_f = final pH
CO_{2i} = CO_2 in raw water
CO_{2f} = CO_2 in aerated water, represented by the CO_2 concentration at equilibrium

Depending on the water quality, in some cases aeration may exacerbate corrosion problems. Aerating water that has little or no initial dissolved oxygen will increase the dissolved oxygen content of the water, which in turn may increase corrosion rates. Corrosion control chemistry and other methods of corrosion control are presented in Chapter 21.

Iron and Manganese Oxidation

Aeration followed by precipitation and filtration is a common method for removing iron and manganese from water supplies. This application is addressed in Chapter 14.

REFERENCES

1. Lang, R., “Trace Organic Constituents in Landfill Gas,” Department of Civil Engineering, University of California, Parks and California Waste Management Board, Sacramento, CA, 1987.
2. Kavanaugh, M. C., and Trussell, R. R., “Design of Aeration Towers to Strip Volatile Contaminants from Drinking Water,” *JAWWA* 73(12):684, 1980.
3. USEPA, “Oxygenates in Water: Critical Information and Research Needs,” EPA/600/R-98/048, 1998.
4. Lewis, W. K., and Whitman, W. C., “Principles in Gas Adsorption,” *Ind. Eng. Chem.* 16:1215, 1924.
5. Higbie, R., “The Rate of Absorption of Pure Gas Transfer into a Still Liquid during Short Periods of Exposure,” *Transactions of the AIChE* 31:365, 1935.
6. Danckwertz, P. V., “Significance of Liquid Film Coefficients in Gas Absorption,” *Transactions of the AIME* 55(3):1460, 1951.
7. EES and ISWA (Economic Engineering Services, Inc., and Illinois State Water Survey), *Lead Control Strategies,* American Water Works Association Research Foundation and American Water Works Association, Denver, CO, 1990.
8. Perry, R. H., and Chilton, C. H. (eds.), *Chemical Engineer’s Handbook,* 5th ed., McGraw-Hill, New York, 1973.
9. Treybal, R. E., *Mass Transfer Operations,* 3rd ed., McGraw-Hill, New York, 1980.
10. Kavanaugh, M. C., and Trussell, R. R., “Air Stripping As a Treatment Process,” in *Proceedings AWWA Annual Conference,* American Water Works Association, Denver, CO, 1981.
11. Singley, J. E., Ervin, A. L., Mangone, M. A., Allan, J. M., and Land, H. H., *Trace Organics Removal by Air Stripping,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1980.
12. Termaath, S. G., “Packed Tower Aeration for Volatile Organic Carbon Removal,” paper presented at the Seminar on Small Water Systems Technology, at Cincinnati, OH (USEPA), April 1982.

13. Dixon, K. L., Lee, R. G., Smith, J., and Zielinski, P., ‘‘Evaluating Aeration Technology for Radon Removal,’’ *JAWWA* 83(4):141–148, 1991.
14. Kinner, N. E., Malley J. P., Jr., Clement, J. A., and Fox, K. R., ‘‘Using POE Techniques to Remove Radon,’’ *JAWWA* 85(6):75–86, 1993.
15. Dyksen, J. F., Raczko, R. F., and Cline, G. C., ‘‘Operating Experience at VOC Treatment Facilities,’’ in *Proc. 1995 AWWA Annual Conference,* American Water Works Association, Denver, CO, 1995.
16. Pontius, Frederick W., *Water Quality and Treatment: A Handbook of Community Water Supplies,* 4th ed., McGraw-Hill, New York, 1990.
17. Scott, G. R., ‘‘Committee Report: Aeration of Water,’’ *JAWWA* 44(4):873, 1955.
18. ASCE and AWWA (American Society of Civil Engineers and American Water Works Association), *Water Treatment Plant Design,* 3rd ed., McGraw-Hill, New York, 1997.
19. Milter, R. J., and Frank, C. A., ‘‘Treatment of Synthetic Organic Contaminants for Phase II Regulations,’’ Internal report DWRD, USEPA, 1985.
20. Roberts, P., and Levy, J., ‘‘Energy Requirements for Air Stripping Trihalomethanes,’’ *JAWWA* 77(4):138, 1985.
21. Lytle, D. A., Schock, M. R., Clement, J. A., and Spencer, C. M., ‘‘Using Aeration for Corrosion Control,’’ *JAWWA* 90(3):74–88, 1998.
22. Mazzei, A. L., Meyer, R. M., and Bollyky, L. J., ‘‘Mass Transfer of High Concentration Ozone with High Efficiency Injectors and Degassing Separators,’’ in *Proceedings of International Ozone Association Pan American Group Conference,* 1995.
23. Overbeck, P., Mazzei, A., Meyer, M., and Mullin. J., ‘‘Advanced Water Processing with the Cost Effective GDT™ Process,’’ in *Proceedings of the AWWA Annual Conference,* American Water Works Association, Denver, CO, 1996.
24. Chaugrani, R., Raupp, G. B., and Turchi, C., ‘‘Integration of Photocatalytic Oxidation with Air Stripping of Contaminated Aquifers,’’ EPA/540/SR-98/504, USEPA, 1999.
25. Bhowmick, M., and Semmens, F., ‘‘Laboratory-Scale Testing of a Continuous CLAS Process,’’ *JAWWA* 86(8):86–96, 1994.
26. Hess, A. F., Dyksen, J. E., and Dunn, H. J., ‘‘Control Strategy—Aeration Treatment Technique,’’ in *Occurrence and Removal of Volatile Organic Chemicals from Drinking Water,* American Water Works Association Research Foundation and American Water Works Association, Denver, CO, 1983.
27. Thibodeaux, L. J., et al., ‘‘Mass Transfer Units in Single- and Multiple-Stage Packed Towers,’’ *Industrial Engineering Chemical Process Design Development* 16(3):325–330, 1977.
28. Gallagher, W. V., ‘‘Control of Algae at Appleton, Wisconsin,’’ *JAWWA* 32:1165, 1940.
29. Haynes, L., and Grant, W., ‘‘Reduction of Chemical Odors at Nitro, West Virginia,’’ *JAWWA* 37:1013, 1945.

CHAPTER 10

Coagulation and Flocculation

INTRODUCTION

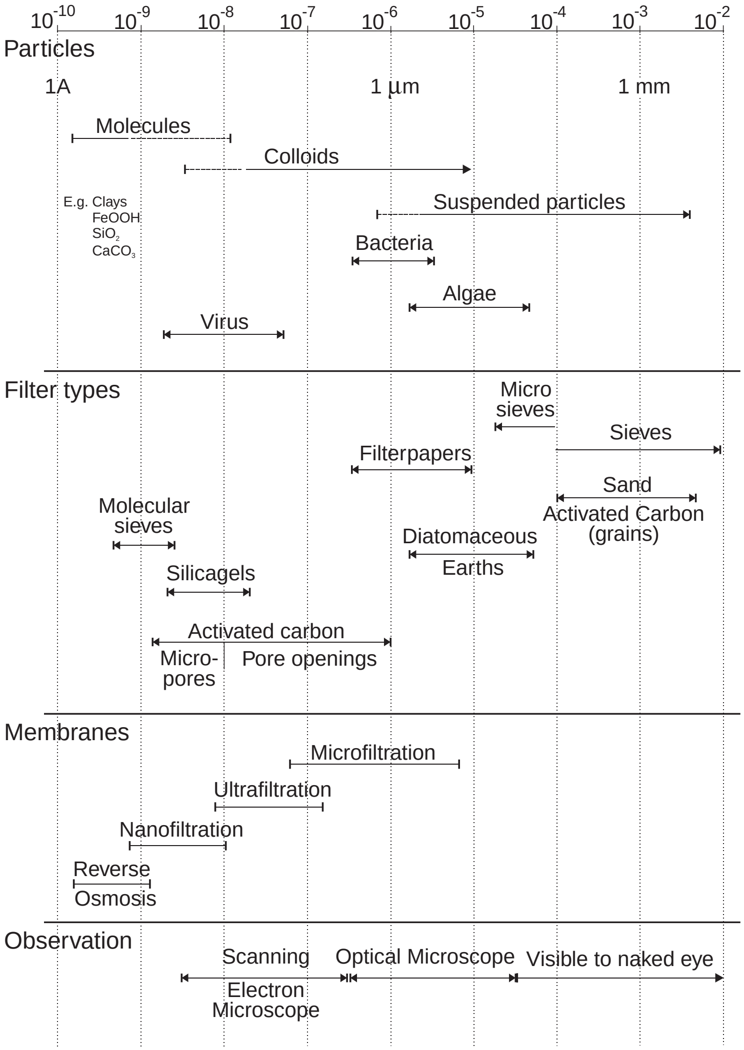
Coagulation/flocculation followed by sedimentation and/or filtration has been used for particulate and color removal for hundreds of years. Although other emerging technologies, such as low-pressure membranes, are proven to be promising alternatives for particle removal, conventional coagulation/filtration remains the most commonly used process for turbidity control. However, in response to rapid changes in regulations and the need to increase treatment capacity of existing facilities, the role of coagulation and its application has significantly expanded since 1990. For example, since metal-based coagulants (e.g., aluminum and ferric salts) are also excellent adsorbents for some soluble species (such as natural organic matter and arsenic) that cause health concerns, using coagulation as a multipurpose process has become a new trend in water industry. Also, the coagulation/flocculation process can be combined with other downstream processes, such as dissolved air flotation (DAF) and membranes, to achieve better treated-water quality and overall system performance.

Although the chemistry of coagulation is the same no matter how it is applied, the purpose and approach of using coagulation have been diversified so much that sometimes the term ‘‘coagulation’’ itself may cause confusion. For example, since the original purpose of coagulation was to improve the removal of natural suspended solids, the term ‘‘coagulation’’ usually implicitly includes ‘‘flocculation’’ and implies the formation of large, dense flocs that can settle rapidly. However, this criterion may not be applicable in the DAF[1] and immersed membranes,[2] since the desirable flocs may be different in these processes.

This chapter will cover the fundamental chemistry and design criteria for coagulation and flocculation processes. The development of new coagulation technologies, such as ballasted flocculation, will also be discussed.

WHAT ARE COAGULATION AND FLOCCULATION?

The presence of particulate materials (e.g., algae clays, silts, and organic particles) and soluble substances (e.g., natural organic matter, [NOM]) in water often cause the water to appear turbid or colored. The settleability of the particulate depends on the density of the material and the size of the particles. Particle size ranges of common particulate found in natural waters are shown in Figure 10–1.[3] The settling velocity of a particle is inversely proportional to the square root of its diameter. For example, a 4-μm



* 1 mm = 10^{-3}m; 1 µm = 10^{-6}m.

Fig. 10-1. Size spectrum of waterborne particles and corresponding filtration processes

particle will settle twice as fast as a 1-μm particle. Conceptually, particles that have density higher (even slightly) than the water should eventually settle due to gravitational force. However, since particles are also experiencing other forces (e.g., electrostatic repulsion between particles) that tend to stabilize them (i.e., keep them suspended), small particles, especially those with density close to water, may never settle and remain suspended in the water unless other ‘‘destabilization’’ mechanisms are introduced.

Therefore, agglomeration or aggregation of particles into a larger floc is a necessary step for their removal by sedimentation. The process that destabilizes the particles (i.e., modifying water chemistry so that particles can attach to other particles) and the process that enhances the aggregation of particles into larger flocs are referred to coagulation and flocculation, respectively. Since coagulation is always followed by flocculation, coagulation is often mistakenly used to represent both coagulation and flocculation.

Impurities that can be removed by coagulation, flocculation, and sedimentation include organic particulate (e.g., algae, bacteria, and viruses), inorganic particulate (e.g., oxidized iron and manganese, calcium carbonate, and clay particles), and soluble organic compounds that impart color (e.g., natural organic matter, NOM). Clays contribute a major portion of natural turbidity in raw waters but are not directly responsible for harmful effects to humans. However, there is some evidence that clays affect human health indirectly through adsorption, transport, and release of inorganic and organic toxic constituents, viruses, and bacteria. Removal efficiencies of clay particles in water treatment are not normally monitored, although several laboratory studies have demonstrated the effectiveness of alum coagulation on clay suspensions.[4,5]

In addition to the removal of particulate matter, coagulation by metal-based coagulant, such as alum and ferric salts, has also demonstrated the capability of removing soluble contaminants, such as NOM and arsenic, via coprecipitation and adsorption. Both laboratory studies and full-scale operation have demonstrated the removal of these constituents using iron and aluminum salts.[6–13] While humic acids were readily removed using these salts, a large fraction of the fulvic acids was not removed. Because humic acids react with chlorine in the formation of halomethanes, haloacetic acids, and other chlorinated compounds, their removal in the coagulation process is an important step in limiting the production of potential carcinogens.

THEORIES OF COAGULATION

Virtually all natural colloids acquire negative charges on their surfaces either by ionization of surface functional groups or by adsorbing negatively charged organic molecules. The aggregation of these suspended colloidal particles takes place in two separate and distinct phases.[14,15] First, the repulsion force between particles must be overcome, a step that requires that the particles be destabilized; and, second, contact between the destabilized particles must be induced so that aggregation can occur. The destabilization step typically is achieved through the addition of chemicals to modify the electrochemistry properties on the particle surfaces, followed by thorough blending in rapid mix tanks. The aggregation step is accomplished through gentle stirring (slow mixing) in flocculation tanks. A representation of the coagulation process is shown in Figure 10–2.[15]

It is useful to define the terms used in connection with the coagulation process.[16] Coagulation is defined as the process that causes a reduction of repulsion forces between particles or the neutralization of the charges on particles. Flocculation is defined as the aggregation of particles into larger elements. The coagulation (destabilization) step is virtually instantaneous following addition of the coagulant, while the flocculation (transport) step requires more time for development of large flocs.

Historically, two theories have been advanced to explain the coagulation process of colloidal systems:

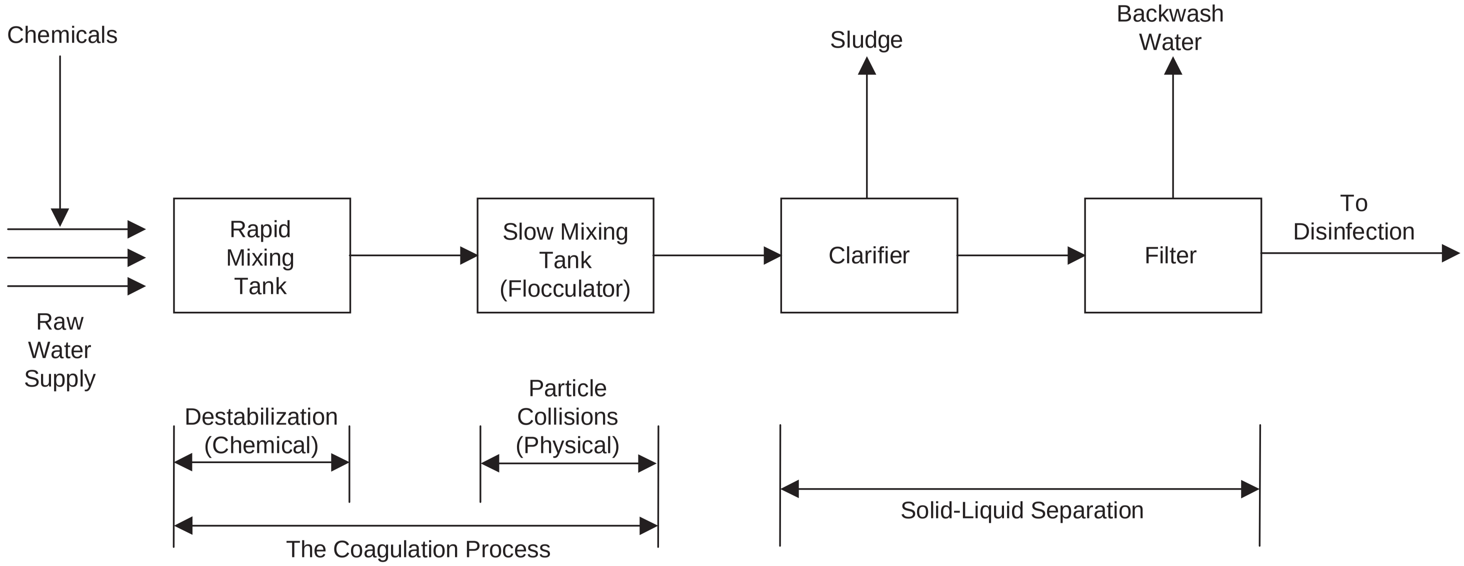


Fig. 10–2. Schematic diagram of conventional coagulation/flocculation/sedimentation process

- Chemical theory
- Physical or double-layer theory

The former theory presumes that destabilization (coagulation) of colloids is a result of the coprecipitation of insoluble complexes that are formed by chemical reactions between the colloids and the coagulants. The second theory is based on the presence of physical factors, such as electrical double layers surrounding the colloidal particles in the solution and counterion adsorption. Destabilization requires a reduction in the electric potential between the fixed layer of counterions and the bulk of the liquid. This electric potential (or zeta potential) can be estimated by observing the movement of microscopically visible particles in an electric field. This theory is presented in more detail below. These two mechanisms are not mutually exclusive, and both theories are used to explain the process of coagulation in treatment systems containing a heterogeneous mixture of colloids.

The Electric Double Layer

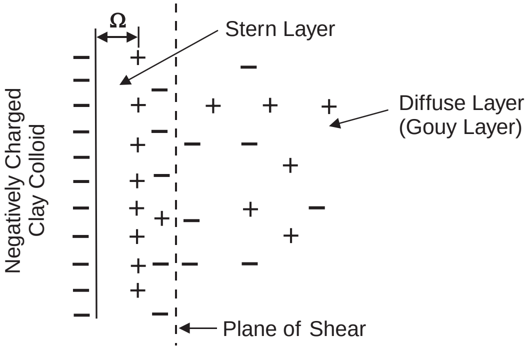
When a colloidal particle is immersed in a solution, electrical charges develop at the particle–water interface. However, a colloidal dispersion does not have a net electrical charge. For electroneutrality to exist, the charges on the colloids must be counterbalanced by ions of opposite charge (counterions) in the solution. The ions involved in establishing the electroneutrality are arranged in an electrical double layer. The concept of the electrical double layer was proposed initially by Helmholtz and later modified and improved by Gouy, Chapman, and Stern.[17–19]

The Stern-Gouy diffuse, double-layer model, which is illustrated in Figure 10–3, can be used to describe the electrical potential in the vicinity of a colloid particle. A portion of the counterions remain in a compact (“Stern”) layer on the colloid surface. The remainder of the counterions extend into the bulk of the solution, and constitute the diffuse (“Gouy-Chapman”) layer. The effective thickness of the double layer is influenced significantly by the ionic strength of the solution, but relatively little by the size of the colloid.

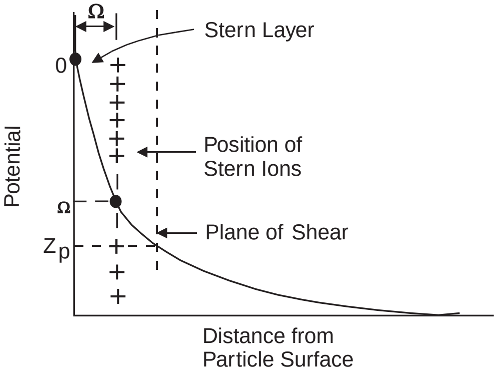
The electrical potential created by the surface charges will attract counterions toward the colloidal particles. The closest approach of the counterions to the particle is limited by the size of the ions. Stern proposed that the center of the closest counterions is separated from the surface charge by a layer of thickness, which represents the Stern layer. The electrical potential drops linearly across this layer. Beyond the Stern layer, in the diffuse layer, the electrical potential decreases exponentially with distance from the particle.

The magnitude of the charge on a colloid cannot be measured directly, but the value of the potential at some distance from the colloid can be computed. This potential, also referred to as *zeta potential*, can be computed by several techniques, such as electrophoresis, electroosmosis, and streaming potential. Most often, the electrophoretic mobility of the colloidal particles is used to compute the zeta potential, by observing the particle mobility through a microscope. The zeta potential is:

$$Zp = \frac{4\pi\nu\mu}{D} \tag{10–1}$$



(a) Distribution of charges in the vicinity of a colloidal particle



(b) Distribution of potential in the electrical double-layer

Fig. 10–3. Stern's model for the electrical double layer

where:

Zp = zeta potential, millivolts
μ = electrophoretic velocity, cm/s
ν = viscosity, poise
π = constant = 3.14159
D = dielectric constant of the liquid

The magnitude of the zeta potential is an approximate measure of colloidal particle stability. Low zeta potentials indicate relatively unstable systems (particles tend to coagulate), while a high zeta potential represents strong forces of separation (via electrostatic repulsion) and a stable system (particles tend to suspend).

The computed value for the zeta potential may vary significantly from the true value because of the uncertainty of values for the various constants in the equation. As a result, many researchers report experimental results in terms of electrophoretic mobility, instead of zeta potential. The electrophoretic mobility may be computed from the equation:

$$\Omega = \frac{dx}{tIR_s} \tag{10–2}$$

where:

Ω = electrophoretic mobility, $\mu/s/V/\text{cm}$
d = distance traveled μ/s
x = cross-sectional area of cell, cm^2
t = time, seconds
I = current density, amperes
R_s = specific resistance of suspension, ohm-cm

Mobilities are positive (+) for those particles that migrate to the negative pole of the cell, and are negative (−) for those that migrate toward the positive pole. The point at which there is no particular migration (zero mobility) is considered to be the isoelectric point (IEP).

Coagulation Mechanisms

Coagulation can be accomplished through any of four different mechanisms:[20,21]

- Double-layer compression.
- Adsorption and charge neutralization.
- Enmeshment by a precipitate (sweep-floc coagulation).
- Adsorption and interparticle bridging.

Double-Layer Compression This mechanism relies on compressing the diffuse layer surrounding a colloid. This is accomplished by increasing the ionic strength of the solution through the addition of an indifferent electrolyte (neutral salt). The explanation for this phenomenon lies in the Schulze-Hardy rule for anions, which was based on Schulze's work on the coagulating power of cations. He noted that coagulating power increases in the ration of 1:10:1000 as the valency increases from 1 to 2 to 3. The Schulze-Hardy rule for anions is:

> *The coagulating power of a salt is determined by the valency of one of its ions. The prepotent ion is either the negative or positive ion, according to whether the colloidal particles move down or up the potential gradient. The coagulating ion is always of the opposite electrical sign to the particle.*

This rule is valid for indifferent electrolytes, which are those that do not react with the solution. If such an electrolyte is added to a colloidal dispersion, the particle's surface charge will remain the same, but the added electrolyte will increase the charge density in the diffuse layer. This results in a smaller diffuse-layer volume being required to neutralize the surface charge. In other words, the diffuse layer is ‘‘compressed’’ toward the particle surface, reducing the thickness of the layer. At high electrolyte concentrations, particle aggregation can occur rapidly.

Adsorption and Charge Neutralization The energy involved in an electrostatic interaction having a 100-millivolt potential difference across the diffuse layer between a colloidal particle and monovalent coagulant ion is only about 2.3 kcal/mole. This compares with covalent bond energies in the range of 50 to 100 kcal/mole. Based on these facts, it is apparent that some coagulants can overwhelm the electrostatic effects and can be adsorbed on the surface of the colloid. If the coagulant carries a charge opposite to that of the colloid, a reduction in the zeta potential will occur, resulting in destabilization of the colloid. This process is quite different from the double-layer compression mechanism described above.

The hydrolyzed species of Al(III) and Fe(III) can adsorb onto the colloids and destabilize the particles. However, at higher doses of Al(III) or Fe(III) coagulation is caused by enmeshment of the colloidal particles in the precipitated metal hydroxide. This aspect is discussed in the next section. Destabilization by adsorption is stoichiometric. Therefore, the required coagulant dosage increases with increasing concentrations of colloids in the solution.

Enmeshment by a Precipitate (Sweep-Floc Coagulation) The addition of certain metal salts, oxides, or hydroxides to water in high dosages could result in the rapid formation of precipitates. These precipitates enmesh the suspended colloidal particles as they settle.[5] Coagulants such as aluminum sulfate ($Al_2(SO_4)_3$), ferric chloride ($FeCl_3$), and lime CaO or $Ca(OH)_2$) are frequently used as coagulants to form the precipitates of $Al(OH)_3(s)$, $Fe(OH)_3(s)$ and $CaCO_3(s)$. The removal of colloids by this method has been termed *sweep-floc coagulation*.

This process can be enhanced when the colloidal particles themselves serve as nuclei for the formation of the precipitate. Therefore, the rate of precipitation increases with an increasing concentration of colloidal particles (turbidity) in the solution. Sometimes additional turbidity (e.g., bentonite particles) is artificially added to the raw water to enhance the sweep-floc coagulation. Packham reported the inverse relationship between the optimum coagulant dose and the concentration of the colloids to be removed.[5] Benefield explained this phenomenon as follows:[20]

> *At low colloidal concentrations, a large excess of coagulant is required to produce a large amount of precipitate that will enmesh the relatively few colloidal particles as it settles. At high colloidal concentrations, coagulation will occur at a lower chemical dosage because the colloids serve as nuclei to enhance precipitate formation.*

This method of coagulation does not depend upon charge neutralization, so an optimum coagulant dose does not necessarily correspond to minimum zeta potential. However, an optimum pH does exist for each coagulant.

Destabilization by Interparticle Bridging Synthetic polymeric compounds have been shown to be effective coagulants for the destabilization of colloids in water. These coagulants can be characterized as having large molecular sizes, and multiple electrical charges along a molecular chain of carbon atoms. Polymers with positive charge (cationic), negative charged (anionic), or even uncharged (nonionic) are capable of destabilizing negatively charged colloidal particles. Surprisingly, the most economical destabilization process is often obtained using anionic polymers.[21] The mechanisms already described cannot be used to describe this phenomenon, although a generally accepted chemical bridging theory/model has been developed that can explain the unusual reaction associated with synthetic polymer compounds.[16,22] However, it should be noted that although a particle exhibits an overall negatively charged characteristic, from the microscopic scale, there could be positively charged sites across the particle surfaces. A segment of the anionic polymer molecule can attach to these positive sites, while the rest of the molecule could extend (stretch) out due to the repulsion from the negatively charged particle. It is possible that this repulsion-induced molecule expansion makes anionic polymers better bridging agents than cationic polymers.

The chemical bridging theory, shown schematically in Figure 10–4, may be explained as follows. The simplest form of bridging, shown in Figure 10–4a, proposes that a polymer molecule will attach to a colloidal particle at one or more sites. Colloidal attachment is postulated to occur as a result of coulombic attraction if the charges are of opposite charge or from ion exchange, hydrogen bonding, or van der Waal's forces.[23] The second reaction is shown in Figure 10–4b, where the remaining length of the polymer molecule from the first reaction extends out into the bulk of the solution. If a second particle having some vacant adsorption sites contacts the extended polymer, attachment can occur to form a chemical bridge. The polymer then serves as the bridge. However, if the extended polymer molecule does not contact another particle, it can fold back on itself and adsorb on the remaining sites of the original particle, as shown in Figure 10–4c. In this event, the polymer is no longer capable of serving as a bridge, and in fact it restabilizes the original particle.

Colloidal restabilization can occur from an overdose of polymer to the sol or from extended or intense agitation. If polymer is added in excess quantities, the polymer segments may saturate the colloidal surfaces to the extent that no sites are available for interparticle bridging. This reaction, shown in Figure 10–4d, results in restabilization of the particles. Excess organic polymer may also increase TOC in the treated water or foul the downstream filters. Intense or extended agitation can result in restabilization due to the destruction of previously formed polymer-surface bonds or bridges. These reactions are depicted in Figures 10–4e and 10–4f.

COAGULATION IN WATER TREATMENT

The coagulation process in water treatment is normally accomplished by using the adsorption or enmeshment mechanisms. The most commonly used coagulants are alum ($Al_2(SO_4)_3 \cdot 14H_2O$), polyaluminum chloride (PACL, $Al(OH)_x(Cl)_y$), ferric sulfate ($Fe_2(SO_4)_3 \cdot 9H_2O$), and ferric chloride ($FeCl_3 \cdot 6H_2O$). Sometimes cationic polymer can also be used as the primary coagulant, although this is not a common practice.

Synthetic organic polymers or activated silica are used as the coagulants when adsorptive-type coagulation is desirable. The coagulants used in the enmeshment principle are either metal hydroxide [Al(III) or Fe(III)] or carbonate precipitates. The selection of the most appropriate coagulant is site specific, and depends on water quality, chemical and sludge handling concerns, and costs. Specific issues that should be considered include turbidity removal efficiency (especially at low temperature),

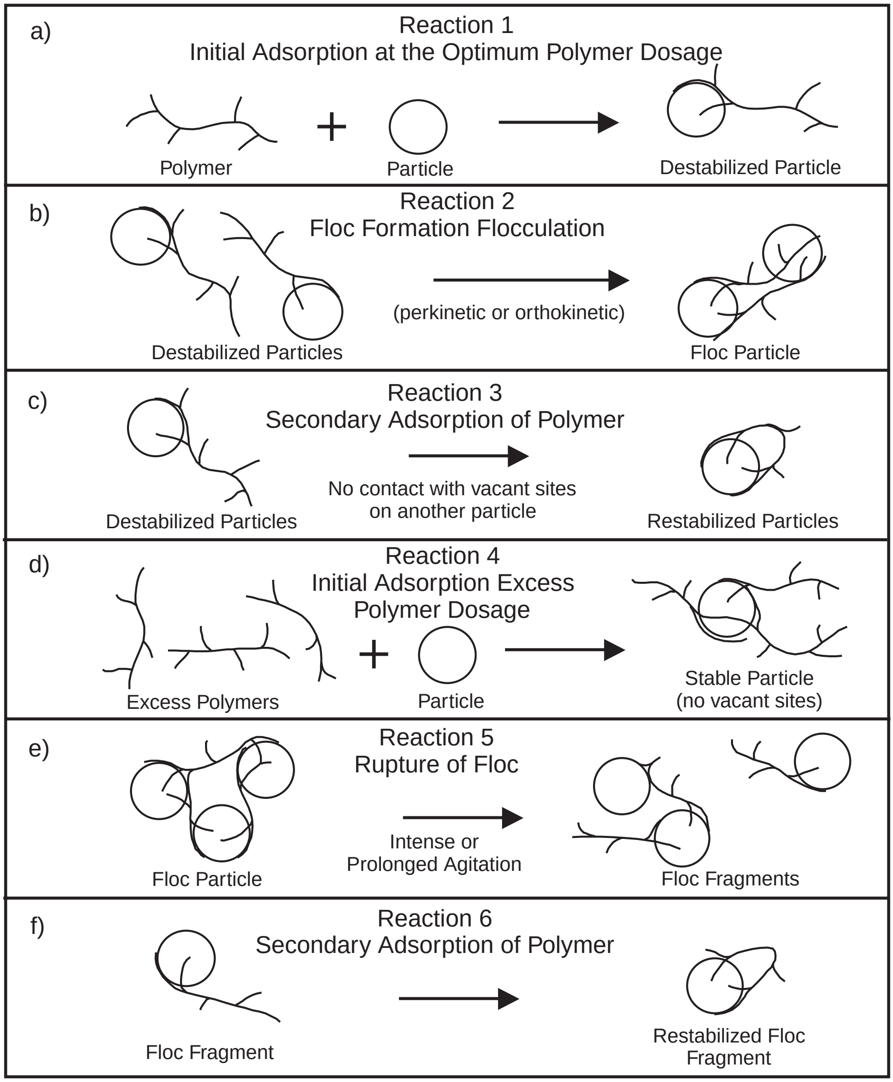


Fig. 10-4. Schematic representation of the bridging model for the destabilization of colloids by polymers

required hydraulic retention time (settling velocity), chemical storage and handling, quantity of sludge generated, dewaterability of the sludge, the compatibility with downstream processes (e.g., membranes), the ability to remove other contaminants (such as DBP precursors), and the cost of the coagulants.

Although both ferric- and aluminum-based coagulants are all effective in forming settleable flocs to assist particulate removal, a particular coagulant may be preferred over the others under specific conditions. For example, compared to alum, PACL has been reported to be more efficient in removing smaller particles ($1 \sim 2\ \mu m$), and the destabilization/flocculation process is much faster.[24] However, the cost of PACL could be up to double that of alum. Hanson and Cleasby[25] reported that alum flocs are much weaker than iron flocs. Also, ferric coagulants can remove more TOC than alum. Since the molecular weights of these coagulants are very different, it is important to note that when comparing the performance between different coagulants, molar concentration (mole/litter) should be used to quantify the coagulant dosage.

Coagulation Using Al(III) and Fe(III)

Al(III) and Fe(III) are the salts used most frequently to coagulate colloidal material in water treatment. These salts are hydrolyzed metal ions, and a brief discussion of the chemistry of these ions will be provided to assist the understanding of ability in destabilizing and coagulating colloidal particles. More detailed discussions of the chemistry of these salts are presented elsewhere.[21,26,27]

When added to water, Al(III) and Fe(III) salts dissociate to their respective trivalent ions, Al^{+3} and Fe^{+3}, and then associate with water molecules to form hydroxy complexes, ($Al(H_2O)_6^{+3}$ and $Fe(H_2O)_6^{+3}$). These complexes then react with the water by replacing the H_2O molecules in the aquometal complex with OH^- ions. These subsequent reactions are called *hydrolytic reactions.* Hydrolytic reactions result in the formation of several soluble species, including monomeric, dimeric, and polymeric hydroxometal complexes. Typical reactions include:[15]

$$Fe(H_2O)_6^{+3} + H_2O = Fe(H_2O)_5(OH)^{+2} + H_3O^+ \tag{10–3}$$

$$Al(H_2O)_6^{+3} + H_2O = Al(H_2O)_5(OH)^{+2} + H_3O^+ \tag{10–4}$$

$$Al^{+3} + 4H_2O = Al(OH)_4^- + 4H^+ \tag{10–5}$$

$$Fe(OH)^{+2} + H_2O = Fe(OH)_2^+ + H^+ \tag{10–6}$$

$$2Fe^{+3} + 2H_2O = Fe(OH)_2^{+4} + 2H^+ \tag{10–7}$$

The last three equations shown above are presented without the H_2O ligands for simplicity.

The speciation of metal complexes or hydroxides greatly depends on the pH of the solution. When Al(III) or Fe(III) salts are added to water in quantities less than the solubility limit of the hydroxide, the hydrolysis products will form and will adsorb on the colloidal particles. Adsorption of the hydrolysis products will cause destabilization by charge neutralization. However, when the amount of Al(III) or Fe(III) added to the water exceeds the solubility limit of the hydroxide, the hydrolysis products will form as kinetic intermediates in the eventual precipitation of metal hydroxides. In this case, charge neutralization and enmeshment in the precipitate both act to destabilize and coagulate the colloids.

Since the solubility of both iron and alumina is very low (less than 10^{-8} M) at the pH commonly used in water treatment (pH $6 \sim 8$), the amount of Al(III) or Fe(III) added in a conventional water coagulation process is sufficient to exceed the solubility limit of the respective metal hydroxides. The solubility of $Al(OH)_3(s)$ and $Fe(OH)_3(s)$ is a minimum at a specific pH (around pH 8.2 and pH 5.5 for Fe and Al, respectively) and increases as the pH increases or decreases from that point. Precipitation of amorphous metal hydroxides is necessary for sweep-floc coagulation.

The pH must be controlled to establish optimum conditions for coagulation. For pHs below the isoelectric point of the metal hydroxide, positively charged polymers (kinetic intermediates) will be formed. Adsorption of these positive polymers can destabilize negatively charged colloids by charge neutralization. Above the isoelectric point, negative polymers will predominate and destabilization is achieved by bridge formation. Control of the coagulation process is complicated by the release of hydrogen ions as shown by Equations 10–3 through 10–7. The hydrogen ions liberated will react with the alkalinity in the water to yield:

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Ca(OH)_2 = 2Al(OH)_3 + 3CaSO_4 + 14H_2O + 6CO_2 \qquad (10\text{–}8)$$

Equation 10–8 predicts that each mg/L of alum will consume 0.50 mg/L (as $CaCO_3$) of alkalinity. If the alkalinity is not sufficient to react with the alum and buffer the pH, then it is necessary to add alkalinity to the water in the form of lime, sodium bicarbonate, soda ash, or some other similar chemical. The following are the stoichiometric reactions:

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Ca(OH)_2 = 2Al(OH)_3 + 3CaSO_4 + 14H_2O \qquad (10\text{–}9)$$

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Na_2CO_3 + 3H_2O = 2Al(OH)_3 + 3Na_2SO_4 + 3CO_2 + 14H_2O \qquad (10\text{–}10)$$

As the Al(III) or Fe(III) is gradually added to water, low coagulant doses may not be sufficient to destabilize the colloidal particles. As the coagulant dosage increases, particles are destabilized and then rapid aggregation occurs. Increasing the coagulant dose further can cause restabilization of the dispersion at some pHs. Finally, if a sufficient quantity of coagulant is added, large amounts of metal hydroxide are precipitated that enmesh the colloidal particles and sweep-floc coagulation occurs.

A knowledge of the interrelationships between optimum coagulant dosage, pH, and colloid concentrations, combined with an understanding of the two modes of destabilization that are caused through the addition of Al(III) or Fe(III) salts, is useful in the operation of a coagulation process. O'Melia describes four types of suspension, as follows:[15]

- *High colloid concentration, low alkalinity*. This is the easiest system to treat, in that only one chemical parameter must be determined—the optimum coagulant dosage. Destabilization is achieved by adsorption of positively charged hydroxometal polymers; these are produced at acidic pH levels (pH 4 to 6, depending on the coagulant).
- *High colloid concentration, high alkalinity*. In this case, destabilization is again achieved by adsorption and charge neutralization at neutral and acidic pH levels. Because of the high alkalinity, the pH will generally remain in the neutral region where the hydroxometal polymers are not highly charged so that charge neutralization is more difficult. The engineer can elect to use a high coagulant dosage. Alternatively, it is possible to reduce alkalinity by adding acids so that particles can be destabilized with a lower coagulant dosage at a lower pH.

- *Low colloid concentration, high alkalinity*. Coagulation is readily accomplished here with a relatively high coagulant dosage by enmeshment of colloidal particles in a sweep floc. Alternatively, a coagulant aid (such as bentonite or clay particles) may be added to increase the colloid concentration and increase the rate of interparticle collision. Destabilization by adsorption and charge neutralization may then be effective at a lower primary coagulant dosage.
- *Low colloid concentration, low alkalinity*. Coagulation is most difficult in such systems. Al(III) and Fe(III) salts will be ineffective if used alone, because the pH will be depressed too low to permit the rapid formation of a sweep floc and the rate of interparticle contacts is presumably too slow to utilize destabilization by charge neutralization. Additional alkalinity, colloidal particles, or both must be added to provide effective coagulation.

Coagulation with Polymers

Synthetic organic *polymers* have been shown to be effective coagulants or coagulant aids. Polymers are long-chain molecules composed of many subunits called *monomers*. A polymer that is composed of only one type of monomer is termed a *homopolymer* and those comprised of different monomers are termed *copolymers*. The number and type of subunits or monomers can be varied to yield a wide range of polymers having different chemical characteristics (such as charge polarity and charge density) and molecular weights.

A polymer is called a *polyelectrolyte* if its monomers consist of ionizable groups. Polyelectrolytes having a positive charge upon ionization are referred to as *cationic polymers*. Negatively charged polyelectrolytes are termed *anionic polymers*. Finally, polymers that do not contain ionizable groups are called *nonionic polymers*.

Cationic polymers can be effective in coagulating negatively charged clay particles.[22] It has been hypothesized that electrostatic forces or ion exchange is the process by which the polymers become attached to the clay particles, which is then followed by bridging. Cationic polymers do not require a large molecular weight to be effective in destabilization.

Anionic particles generally are ineffective coagulants for negatively charged particles,[22] and there is strong evidence that the presence of divalent metal ions (such as Mg^{2+}) is necessary for anionic polymers to flocculate negative colloids. However, anionic polymers of large molecular weight or size are able to bridge the energy barrier between two negatively charged particles, thereby effectively enhancing the coagulation efficiency. The minimum polymer size depends on several factors, but limited data indicate that the minimum size is on the order of a molecular weight of one million.[15] When anionic polymers are used in conjunction with an electrolyte such as NaCl or $CaCl_2$ or another coagulant such as alum, their coagulation efficiency is increased.

Low dosages of cationic polymer (0.1 to 1.5 mg/L) are usually sufficient to achieve coagulation. In contrast, 5 to 150 mg/L of alum is often needed to obtain similar results. Other important differences between the use of polymers and metal ions are sludge quantities and dosage control. The use of alum or ferric chloride can result in copious volumes of sludge that must be handled, whereas the additional sludge quantity is negligible when a polymer is used. A narrow bank exists for optimum polymer dosage. Overdosing or underdosing from this optimum will result in restabilization of the colloids. The control method for polymer feed systems must be precise and reliable to give satisfactory performance.

Because polymers do not affect the pH of water, their use offers a clear advantage for treating low-alkalinity waters. This is particularly true of the low-alkalinity waters that are high in turbidity. Such waters would require considerable qualities of alum, which would require the addition of soda ash or lime to replenish the buffering capacity of the water and maintain desirable pH.

Coagulation with Magnesium

Magnesium precipitated as magnesium hydroxide ($Mg(OH)_2$) has been shown to be an effective coagulant for the removal of color and turbidity from waters.[28,29] Coagulation is achieved by enmeshing the colloidal particles in the gelatinous hydroxide precipitate. Similar findings have been shown in lime treatment of wastewaters.

Waters that are high in naturally occurring magnesium can be coagulated by raising the pH to the point at which $Mg(OH)_2$ is precipitated. Normally, pH elevation is accomplished through the addition of lime. For waters that are low in magnesium, it is necessary to add a suitable magnesium salt. In using lime to raise the pH, one advantage is that both lime and magnesium can be recovered from the sludge and reused. The recovery process takes the form of recalcination in a furnace, typically a multiple hearth furnace. The sludge disposal problems are reduced by the lime recovery. However, the sludge-handling processes are complex. The economics of treatment must be carefully evaluated before recalcination is implemented.

The chemistry of magnesium coagulation with lime addition is based on water softening and coagulation theories. The reactions involved in $Mg(OH)_2$ precipitation are:

$$CO_2 + Ca(OH)_2 \rightarrow \underline{CaCO_3} + H_2O \tag{10–11}$$

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow \underline{2CaCO_3} + 2H_2O \tag{10–12}$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow MgCO_3 + \underline{CaCO_3} + 2H_2O \tag{10–13}$$

$$MgCO_3 + Ca(OH)_2 \rightarrow \underline{Mg(OH)_2} + \underline{CaCO_3} \tag{10–14}$$

The solubility diagram for magnesium (Fig. 10–5) shows that precipitation is enhanced at high pH values. In practice, sufficient lime is added to raise and maintain pH at least 10.7 through the clarification process.

Recovery of magnesium is achieved by recarbonation to lower the pH to the point at which $Mg(HCO_3)_2$ is formed and removed. The magnesium can then be returned to the coagulation process. Alternatively, the magnesium can be held with the lime sludge for recalcination for lime recovery. The recovered lime is then used to raise the pH as previously explained. Lime–soda softening and recarbonation are described in Chapter 21.

Coagulant Aids

Ideally, flocculated colloidal particles should settle rapidly and be strong enough to resist shearing forces. Often, the flocs do not possess these characteristics, and a co-

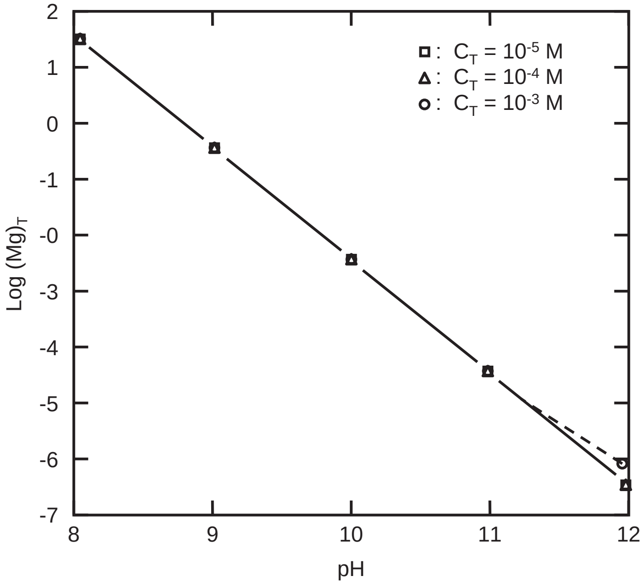


Fig. 10-5. Relationship between total soluble magnesium, pH, and the final equilibrium total carbonic species concentration (*Source:* Reference 20; courtesy Larry Benefield)

agulant aid is then added to improve floc properties. Coagulant aids that have been used include clays, activated silica, and polymers.

Bentonite clays have been used as coagulant aids for low-turbidity waters. The use of clay may reduce the amount of coagulant and improve the floc settleability. The reduction in required coagulant dose is achieved by providing greater particle contact opportunities (increased colloid concentration) with subsequent charge neutralization.

The other advantage of using clays is that the floc particles are weighted (ballasted) by the clay particles, which cause the floc to settle more rapidly than regular alum flocs. Bentonite doses in the range of 10 to 50 mg/L are generally sufficient for improved coagulation efficiency. However, the optimal dosage should be determined by laboratory testing.

Activated silica has been used as a coagulant aid in water treatment plants. When used in conjunction with alum, activated silica, with a mechanism[30] similar to that of bentonite particles, increases the rate of flocculation, improves floc toughness, and increases settleability.

Activated silica normally is used with alum, with the dose typically in the range of 7 to 11 percent of the alum dose. However, use of excess silica can be detrimental to coagulation. Activated silica has been successfully added both before and after alum addition, although the latter approach is the more widely used. Jar tests should be used to identify the optimum combination of chemicals to use. Activated silica has also been found to be an effective filter aid because it strengthens the flocs.[31]

The use of polymers has been discussed elsewhere, and only its use as a coagulant aid is presented here. Both anionic and nonionic polymers have proved effective as coagulant aids. The polymers help to promote large floc particles by a bridging mechanism, after the colloidal particles have been destabilized by a coagulant such as alum. Nonionic polymers are more effective with increasing concentrations of divalent cations (Ca^{+2}, Mg^{+2}, etc.).

Anionic polymer doses in the range of 0.1 to 0.5 mg/L in association with primary metal-based coagulants improve floc settleability and toughness compared to primary coagulants alone. However, overdosing the solution can inhibit coagulation and should be avoided.[32] A side benefit of using a combination of alum and polymer is the fact that frequently the coagulant dosage can be reduced and less sludge is produced.

Dewaterability of Sludge

Another important criterion that should be considered when selecting appropriate coagulant is the sludge handling. The quantity of sludge generated from the process and the dewaterability of the sludge could significantly affect the operation cost. Factors that affect sludge dewaterability include the particle size, the bond water, and the floc density. It should be kept in mind that the mechanism of sludge dewatering is very complicated because of the complex sludge composition and small particle sizes. Therefore, no single parameter should be used as the dominant factor or sole avenue for optimization.

Water molecules that bond to the particle surface either physically or chemically are very difficult to remove. Fe(III) has a lesser tendency to bind water than Al(III) and, therefore, ferric sludge usually contains less bond water. Also, sludge that contains a larger portion of smaller particles tends to have more bond water (due to higher surface area) and, therefore, be more difficult to dewater. Smaller colloids also tend to build up a compact cake layer that posses higher hydraulic resistance during filtration. However, it is found that larger, rigid, settleable particles do not necessarily always give the best filterability due to possible compaction effect.

Density of the coagulated flocs could be the most important factor that determines the dewaterability of the sludge. As mentioned earlier, adsorption-charge neutralization and enmeshment (sweep flocculation) are the two major floc formation mechanisms. Knocke et al.[33] reported that the flocs formed by adsorption-charge neutralization possess a much higher density than the flocs formed by the enmeshment. Since low-density flocs have higher water content, it is generally more difficult to dewater. However, sweep flocculation is more effective for removing turbidity, and therefore the trade-off between flocculation performance and the sludge handling should be carefully justified. In the field practice, the sludge dewaterability could be enhanced by reducing coagulation pH, reducing coagulant-to-turbidity ratio, and utilizing adsorption-charge neutralization.

RAPID MIXING

The most commonly used method for designing rapid mix equipment is based on the velocity gradient or G-value, as developed by Camp.[34] It is a surrogate that measures the amount of energy dissipated into the water: Higher energy input provides better mixing. The power requirements are computed from the equation:

$$G = \left(\frac{P}{\mu V}\right)^{1/2} \tag{10–15}$$

where:

G = velocity gradient, sec^{-1}
P = power input, ft-lb/sec (N − m/s)
μ = dynamic viscosity, lb-sec/sq ft (N − s/m^2)
V = volume of water receiving input, cu ft (m^3)

Several studies have demonstrated that the velocity gradient does not completely describe the mixing process, but until a more complete understanding of the mixing process is developed, the *G*-value will continue to be used for designing rapid mix units.

Experience has shown that for the same amount of turbidity removal, a higher *G*-value (with the same mixing time, *t*) or a higher *Gt* (the product of *G* and *t*) produces a denser floc. However, there is a limitation to the premise that a higher *Gt* produces a denser floc, because high shear rates break up previously formed flocs. Therefore, the goal is to set *G* as high as possible without shearing the floc. Typical values for *G* and *t* are shown in Table 10–1.

Rushton developed a relationship for the power requirements for turbulent conditions:[35]

$$P = k\rho N^3 D^5 \tag{10–16}$$

where:

P = power, ft-lb/sec (N − m/s)
k = constant based on type of impeller
ρ = density of fluid, slug/cu ft (kg/m^3)
N = impeller rotational speed, revolutions/sec
D = impeller diameter, feet (m)

The values of the constant *k* range from 1.0 for a three-bladed impeller to 6.30 for a turbine with six flat blades.

The foregoing mathematical relations can be used to design a mechanical rapid mixer. The equations can be used to size the equipment and power requirements. Examples of their use are presented later.

TABLE 10–1. Typical Values for *G* and *Gt*

Description	*G*, sec^{-1}	*Gt* (dimensionless)
Camp[48]	20 to 74	2×10^4 to 2×10^5
Turb/color removal; no solids recirculation[68]	50 to 100	1×10^5 to 1.5×10^5
Turb/color removal; solids contact reactors[68]	75 to 175	1.25×10^5 to 1.5×10^5
Softening; solids contact reactors[68]	130 to 200	2×10^5 to 2.5×10^5
Softening; ultrahigh solids contact[68]	250 to 400	3×10^5 to 4×10^5

RAPID MIXING SYSTEMS

There are several types of rapid-mix systems used in water treatment:

- Mechanical mixers
- In-line blenders
- Jet injection
- Hydraulic mixing

Mechanical Mixing

Mechanical mixing is the most commonly used system for rapid mixers. This system is effective, has little headloss, and is unaffected by the volume of flows or flow variations. Typical design practice provides a contact time in the range of 10 to 30 sec and a *G*-value in the range of 700 to 1,000 sec^{-1}.

Gemmell recommends a 20-sec or less detention time and a power input of 1 to 2 hp/cu ft/sec (26.36 to 52.72 kW/m^3s) of flow.[36] He further states there are specific guidelines for determining detention time or power dissipation required to disperse chemicals. Examples of typical rapid-mix units are shown in Figures 10–6 and 10–7.

Camp conducted studies on Boston tap water using ferric sulfate.[37] These studies indicated that a maximum floc volume was obtained using a flat-bladed mixer with a velocity gradient of 700 to 1,000 sec^{-1}, and a detention time of 2 to 2.5 minutes. The detention time is significant because precipitation of ferric hydroxide occurred within 8 sec. The dosage of 15 mg/L created sweep-floc coagulation.

Another study states that the *G*-value ‘‘does not provide a complete characterization of the mixing in the rapid-mix operation.’’[38] This study found that the optimum period of rapid mix was 2 minutes at a *G*-value of 1,000 sec^{-1}.

TeKippe and Ham demonstrated that a rapid mixer design, which has a detention time of long enough duration to allow floc particles to reach near-equilibrium sizes, provides for optimum sedimentation.[39] The rapid mixer should be followed by a tapered flocculation velocity gradient. Visible floc formation ranged in time from 2 to 6 minutes.

Finally, Vrale and Jorden tested five different types of rapid mix units and concluded that the ‘‘backmix reactor is very inefficient for rapid mixing.’’[40] They further concluded that ‘‘a tubular reactor appears to be the most efficient type.’’

Amirtharajah summarized and recommended guidelines for designing a mechanical rapid mix unit:[41]

- A square vessel is superior in performance to a cylindrical vessel.
- Stator baffles are advantageous.
- A flat-bladed impeller performs better than a fan or propeller impeller.
- Chemicals introduced at the agitator blade level enhance coagulation.

The mechanical mixer has been used in numerous water treatment plants. Because the mixer speed can be changed by including a variable speed drive, it is amenable to operational changes due to changing conditions. For example, lower speeds (lower *G*-values) are applied when polyelectrolytes are used.

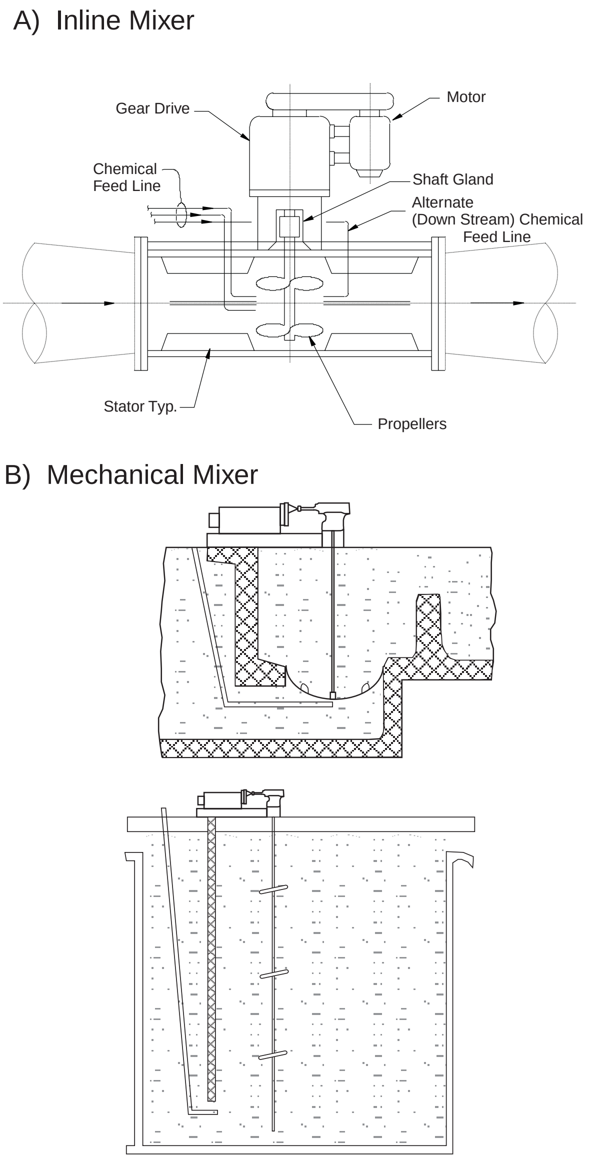


Fig. 10-6. Typical in-line blender (mixer) (*Source:* Reference 70. Reprinted from *Water Treatment Plant Design*, 3d ed., by permission. Copyright © 1998, American Water Works Association and American Society of Civil Engineers.

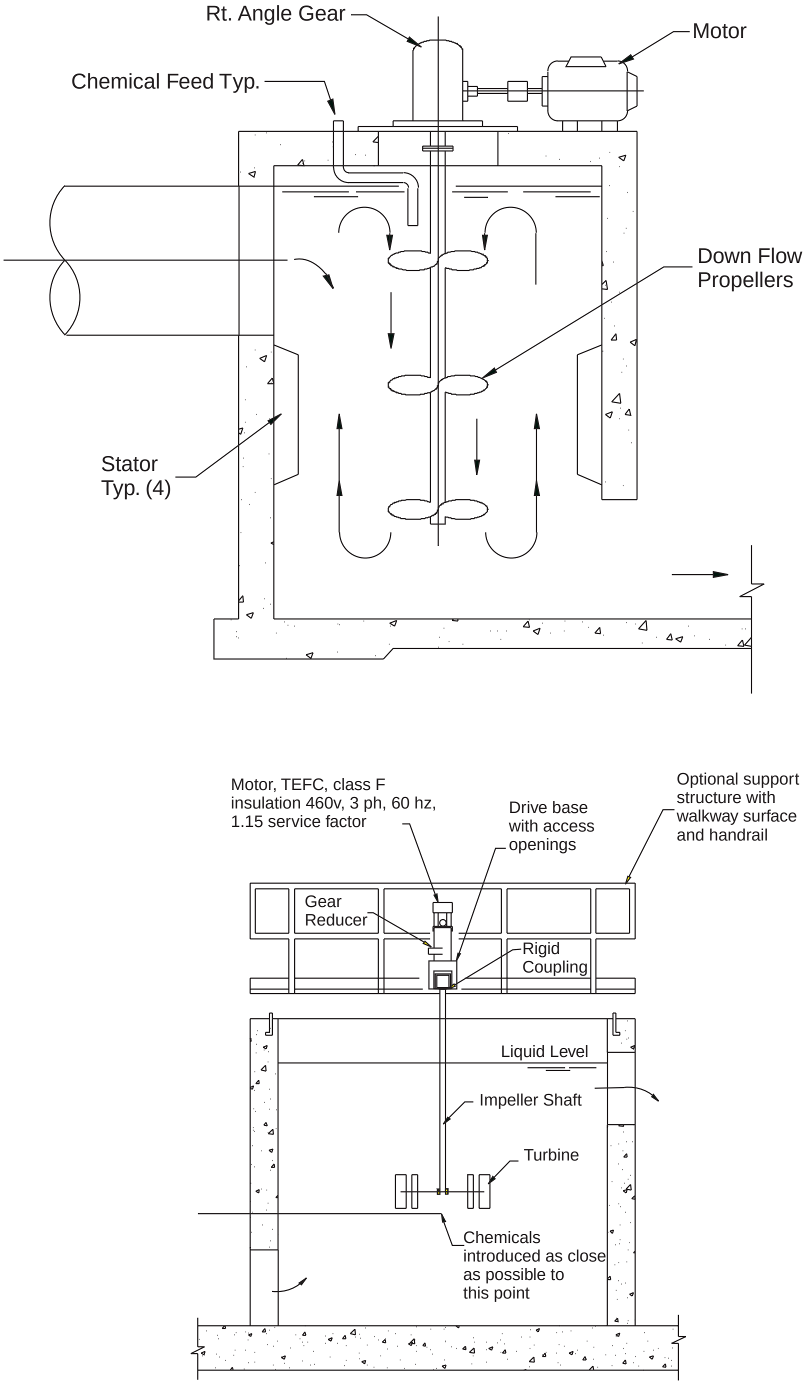


Fig. 10-7. Examples of typical rapid mix units: (a) propeller-type mechanical flash mixer; (b) turbine-type mechanical flash mixer (*Source:* Reference 70. Reprinted from *Water Treatment Plant Design*, 3d ed., by permission. Copyright © 1998, American Water Works Association.)

In-Line Blenders

In-line blenders were developed to approach instantaneous mixing of chemicals, and it is suitable for the rapid adsorption-destabilization reactions. The *G*-value suggested for in-line blenders is in the range of 3,000 to 5,000 sec^{-1}. Hudson recommended using in-line blenders with a residence time of 0.5 sec and a water hp of 0.5 hp/mgd (0.99 kW/ML/d) of flow.[42]

Kawamura prefers the use of in-line blenders and gives the following reasons:[43,44]

- In-line blenders provide virtually instantaneous mixing with a minimum of short-circuiting.
- There is no need to consider headlosses.
- In-line systems are less expensive than more conventional rapid-mix units.

Types of in-line mixers include:

- Custom-designed units
- Proprietary mechanical devices, such as the Water Champ manufactured by USFilter Stranco and Gas Master manufactured by the Master Company
- Static mixers manufactured by several companies, including Koch Engineering and Komax systems

In-line mixers may not be less expensive than other methods in all cases. A detailed cost–benefit evaluation should be conducted to determine the best engineering solution.

Jet Injection Blending

A study by Vrale and Jorden showed that a jet injection device, which introduced the coagulant through six holes [0.028 inch (0.71 mm) diameter], was superior to a typical backmix reactor.[40] The jet injection system required a *G*-value of 1,000 sec^{-1} to achieve the maximum particle aggregation rate. Other mixing units tested in the same study required *G*-values of 6,000 to 9,000 sec^{-1} to achieve the same aggregation rate. Chao and Stone developed a typical design for a jet injection system.[45] The result was a *G*-value $\sim$ 1,000 sec^{-1} and a detention time of 0.55 sec.

A unit used for full-scale applications is shown in Figure 10–8. The utilization of this type of unit is limited in practice, although it has been shown to have potential advantages. Two disadvantages were that the orifices plugged and mixing intensity could not be varied.

The unit shown in Figure 10–8 has design criteria of:[44]

- $G = 750$ to 1,000 sec^{-1}
- Dilution ratio at maximum alum dose = 100:1
- Flow velocity at injection nozzle = 20 to 25 ft/sec (6.1 to 7.6 m/s)
- Mixing time = 1 sec

The power input, P (ft-lb/sec or watts), for this type of flash mixer can be computed from:

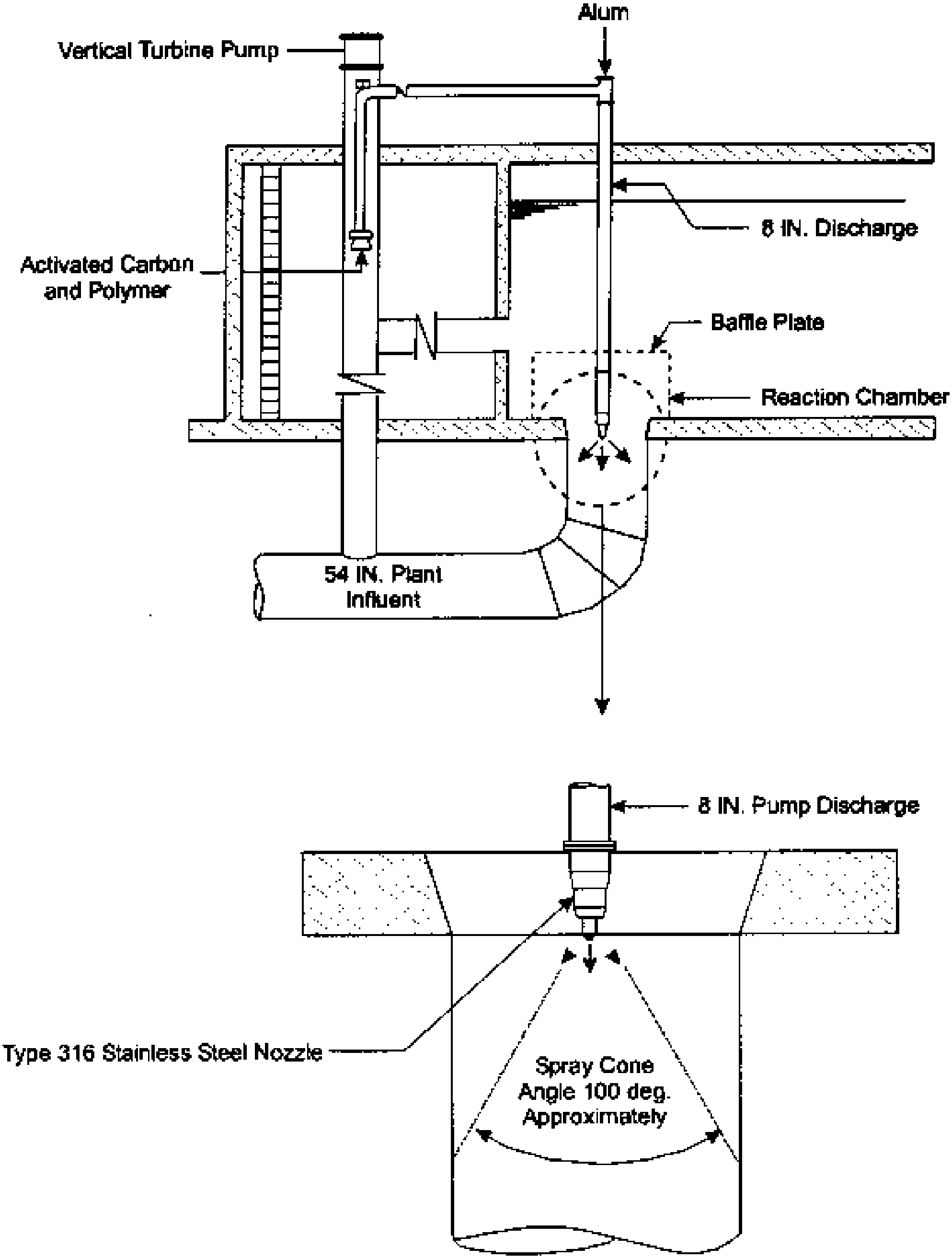


Fig. 10-8. Typical flash mixing facility (*Source:* Reference 44. Reprinted from *Water Treatment Plant Design*, 3d ed., by permission. Copyright © 1998, American Water Works Association and American Society of Civil Engineers.)

$$P = 1.356q(\Delta H)\rho g \qquad (10\text{–}17)$$

$$P = Q(\Delta H)\rho g \qquad \text{(metric)}$$

where:

P = power input, ft-lb/sec (W)
q = flow rate from orifice hole—jet discharge, cu ft/sec (m^3/s)
= $C_d a \nu$
ΔH = total jet energy loss, feet (m)
= $\nu^2/2g$
ρ = density of water, lb/cu ft (kg/m^3)
g = gravitational acceleration 32.2 ft/sec^2 (9.81 m/s^2)
a = area of orifice, sq ft (m^2)
ν = jet velocity, ft/sec (m/s)
C_d = discharge coefficient (± 0.75)

Substituting known values in Equation 10–17 yields:

$$P \cong 0.97\ C_d a \nu^3 \qquad (10\text{–}18)$$

$$P \cong 500\ C_d a \nu^3 \qquad \text{(metric)}$$

One important advantage of jet mixing is that untreated raw water or partially treated water (e.g., the treated backwash water from the filters) can be used for chemical injection. A valve on the chemical pump discharge line allows flexibility in changing the power input. This system can be adjusted to accommodate changing raw-water conditions.

Hydraulic Mixing

Hydraulic jumps have been used for mixing chemicals. Frequently, plant flows are measured by a Parshall flume or other similar device that incorporates a hydraulic jump downstream by including an abrupt drop in the channel. The coagulants are introduced immediately upstream of the flume. Typical residence times are about 2 sec with a G-value of about 800 sec^{-1}.

Chow presents the mathematical equations required to compute the G-values.[46] The headloss through the flume varies with the flow rate and can be computed or obtained from discharge tables. The principal advantages of this unit are:

- No mechanical equipment to operate and maintain
- Lower cost because there is no separate rapid mix unit

DESIGN EXAMPLES FOR RAPID-MIX SYSTEMS

In-Line Blender

If jar tests indicate that adsorption–destabilization is the preferred coagulation mechanism, an in-line blender is the appropriate mixer. Assume the following plant sizing and criteria:

- Average day flow = 2.0 mgd (7.57 ML/d)
- Maximum day flow = 3.0 mgd (11.36 ML/d)
- Temperature range = 46.4 to 68°F (8 to 20°C)

Select a typical commercial mixer at a water horsepower of 0.5 hp/mgd (0.099 kW/ML/d) of flow. For maximum day flows, the hp requirement is:

$$\text{hp} = 0.5\ \text{hp/mgd} \times 3.0\ \text{mgd} = 1.5\ \text{water hp}\ (1.12\ \text{kW})$$

Assume a detention time of 0.5 sec in the mixer. This should be checked for the commercial blender being considered. Lowest possible temperature should be used to calculate the required power because it gives the highest water viscosity and therefore the lowest *G*-value.

$$\begin{aligned}\text{Power input} &= 1.5 \times 550\ \text{ft-lb/sec hp}\\ &= 825\ \text{ft-lb/sec}\ (1.12\ \text{N} - \text{kW})\end{aligned}$$

$$\begin{aligned}\text{Volume of mixer} &= 3.0\ \text{mgd} \times 1.547\ \text{cu ft/sec/mgd} \times 0.5\ \text{sec}\\ &= 2.32\ \text{cu ft}\ (0.066\ \text{m}^3)\end{aligned}$$

Viscosity at 46.4°F (8°C):

$$\begin{aligned}\mu &= 1.387\ \text{centipoises} \times 2.088 \times 10^{-5}\ \text{lb sec/sq ft/centipoise}\\ &= 2.90 \times 10^{-5}\ \text{lb sec/sq ft}\ (1.379 \times 10^{-3}\ \text{N} - \text{s/m}^2)\end{aligned}$$

Velocity gradient *G*:

$$G = \left[\frac{P}{\mu V}\right]^{1/2} = \left[\frac{825}{2.90 \times 10^{-5} \times 2.32}\right]^{1/2} = 3{,}500\ \text{sec}^{-1}$$

At a temperature of 68°F (20°C):

$$G = \left[\frac{825}{2.092 \times 10^{-5} \times 2.32}\right]^{1/2} = 4{,}100\ \text{sec}^{-1}$$

The *G*-value for the higher temperature is below the recommended maximum value of 5,000 sec^{-1}.

Backmix Reactor

Assuming the jar test results indicate sweep-floc coagulation as the preferred mechanism, a backmix reactor is suitable. The same plant size and criteria as above will be used with an assumed mean alum dose of 25 mg/L. Using the equation $Gt_{\text{opt}}C^{1.46} = 5.9 \times 10^6$, the value of *Gt* can be estimated:

$$Gt_{\text{opt}} = \frac{5.9 \times 10^6}{25^{1.46}} \cong 53{,}700$$

Although this equation was developed for only one type of colloid suspension, it does provide a starting point and the results should be checked against normally accepted design practice.

- For $G = 1{,}000\ \text{sec}^{-1}$, $t_{\text{opt}} = 53.7$ sec
- For $G = 700\ \text{sec}^{-1}$, $t_{\text{opt}} = 76.7$ sec

According to *Water Treatment Plant Design*,[45] for mixing times greater than 40 sec, a G-value of 700 sec^{-1} should be used. Higher G-value (700–1000 sec^{-1}) should be used if detention time is less than 30 seconds. In this example, a detention time of 60 sec and a G-value of 700 sec^{-1} would be appropriate.

The volume of the rapid mix chamber is calculated as:

$$\begin{aligned} V &= 3.0\ \text{mgd} \times 1.547\ \text{cu ft/sec/mgd} \times 60\ \text{sec} \\ &= 280\ \text{cu ft}\ (7.9\ \text{m}^3) \end{aligned}$$

Design for at least two units and make the dimensions of each unit compatible with the raw-water influent channel or pipe.

Using two units:

$$\text{Volume} = \frac{280}{2} = 140\ \text{cu ft}$$

Use a square chamber with 5-foot side dimensions:

$$\text{Depth} = \frac{140}{5 \times 5} = 5.6\ \text{ft}$$

Design the rapid mix chamber for 6-foot depth for 150 cu ft volume. Use a turbine mixer with incline blades. The actual mixer dimensions and recommendations should be obtained directly from the manufacturer.

Compute motor horsepower by rearranging Equation 10–15 as follows:

$$\begin{aligned} P &= G^2 \mu V \\ &= 700^2 \times 2.9 \times 10^{-5} \times 150 \\ &= 2130\ \text{ft-lb/sec} \end{aligned}$$

$$\text{Water hp} = \frac{2130}{550} = 3.9\ \text{hp}$$

Using an overall efficiency for motor and drive of 80 percent:

$$\text{Motor hp} = \frac{3.9}{0.8} = 4.8\ \text{hp} \quad \text{(Use 5 hp motor)}$$

Jet Injection

Figure 10–9 illustrates a recommended design for in-line jet mixer presented by Chao and Stone.[45] Since specific design parameters closely associate with each manufac-

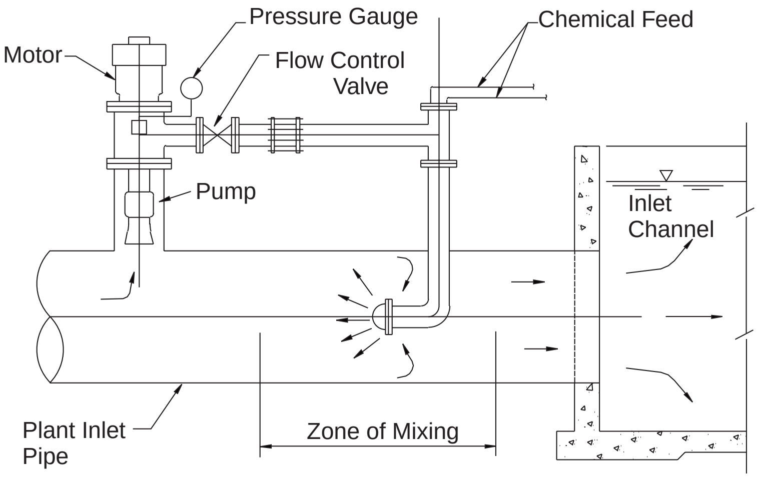


Fig. 10-9. In-line jet mixing system (*Source:* Reference 70. Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association.)

turer's proprietary design, it is recommended that reference should be made to Chao and Stone and to manufacturer's literature for a more thorough understanding of this rapid mix unit.[45]

TEMPERATURE EFFECTS

Morris and Knocke studied the effect of temperature on coagulation.[47] Specifically, the study:

- Determined the impact of low temperature conditions on the efficiency of metal ion coagulants for turbidity removal from surface waters
- Investigated fundamental parameters, such as reaction rate kinetics and particle size characteristics, to aid in describing the observed results

This study showed that low temperatures did not affect the rate of metal ion precipitation in the pH range 6.0 to 8.0. However, in the evaluation of turbidity removal, a decrease in water temperature was accompanied by a decrease in turbidity removal, as shown in Figures 10–10 and 10–11 for aluminum sulfate and ferric chloride. Other studies have also shown that the flocculation efficiency was significantly decreased at lower temperature while the solution pH was kept constant. This phenomenon can be explained by the fact that, although at constant pH, the hydroxyl ion concentration also decreases when temperature decreases, causing slower particle destabilization rate.[24,25] Therefore, maintaining a constant pOH, opposed to constant pH, is an appropriate approach to maintain constant coagulation/flocculation performance at lower temperature.[25] Alternatively, maintaining a constant mixing G value (by increasing energy input) to compensate higher water viscosity at lower temperature is another

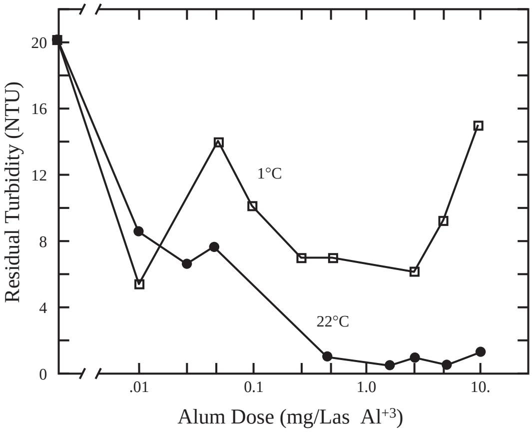


Fig. 10–10. Temperature effects on turbidity removal using aluminum sulfate (*Source:* Reference 47. Reprinted from *Proceedings of 1983 AWWA Annual Conference,* by permission. Copyright © 1983, American Water Works Association.)

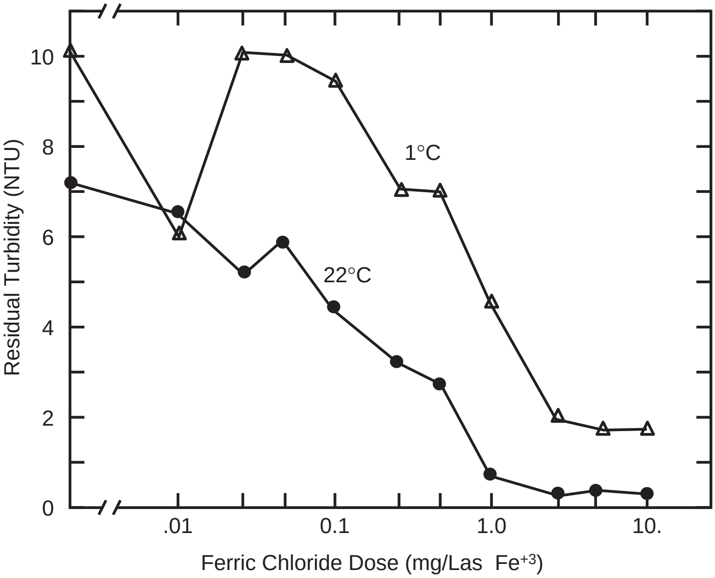


Fig. 10–11. Temperature effects on turbidity removal using ferric chloride (*Source:* Reference 47. Reprinted from *Proceedings of 1983 AWWA Annual Conference,* by permission. Copyright © 1983, American Water Works Association.)

way to preserve constant flocculation rate.[24] Compared to alum, PACL is less sensitive to the change of hydroxyl concentration and therefore could be more effective for low temperature application.

Morris and Knocke have also indicated that increasing alum dosage at low temperature will actually reduce the size of the coagulated flocs (Fig. 10–12). While this result may imply that an increase in alum dosage during cold temperatures may not offer any improvement in turbidity removal, other mechanical factors should also be carefully evaluated. For example, Hanson and Cleasby[25] conducted a study on the effect of temperature on turbulent flocculation and concluded that although the impeller geometry does not impact the particulate removal efficiency at warm temperature (20°C), it is important at lower temperature (5°C), especially in breaking up the flocs: turbine impellers tend to cause more floc breakup than the stake and stator impeller.

FLOCCULATION SYSTEMS

The flocculation process aggregates destabilized particles into larger and more easily settleable flocs. While the coagulation process destabilizes particles through chemical

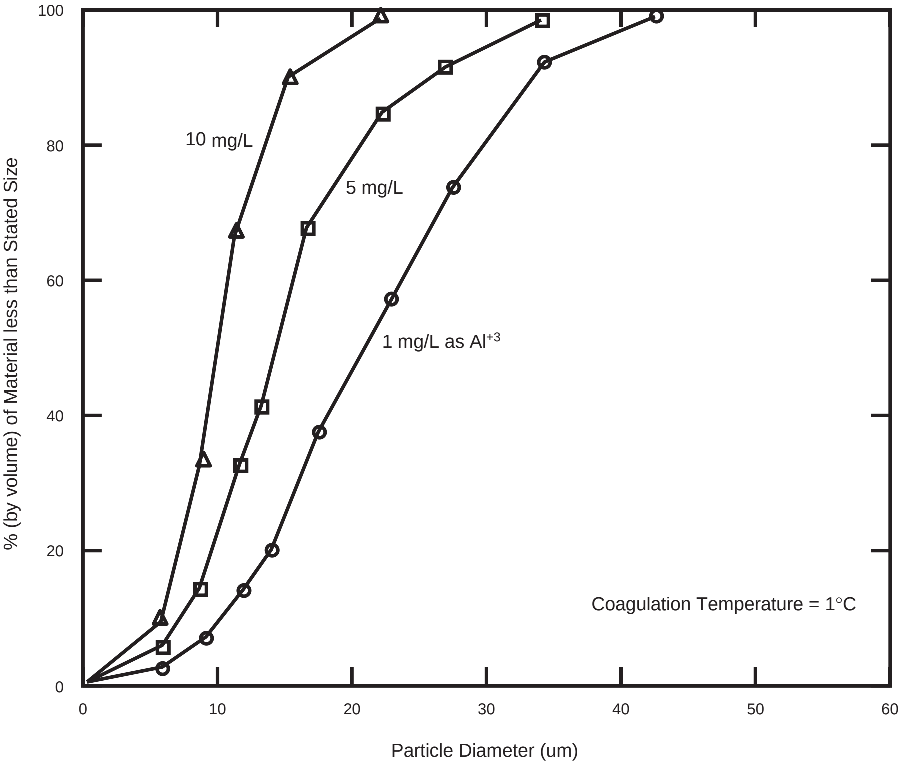


Fig. 10–12. Effect of alum coagulant dose on particle size distribution produced after flocculation (*Source:* Reference 47. Reprinted from *Proceedings of 1983 AWWA Annual Conference,* by permission. Copyright © 1983, American Water Works Association.)

reactions between the coagulant and the suspended colloids, flocculation is the transport step that causes the necessary collisions between the destabilized particles. The flocculation process typically follows rapid mixing.

There are three major mechanisms of flocculation:

- *Perikinetic,* which is the aggregation of particles as a result of random thermal motion (Brownian diffusion) caused by the continuous bombardment by surrounding fluid molecules. The driving force for this type of particle movement is the thermal energy of the fluid. This is significant for particles that are 1 to 2 μm in size.
- *Orthokinetic,* which is the aggregation of particles by induced velocity gradients in the fluid. The suspended particles follow the streamlines with different velocity and eventually lead to interparticle contacts. As mentioned earlier, the velocity gradient, G, is directly related to the energy dissipated into the water (via mixing).
- *Differential settling,* which is caused by different settling velocities of particles.

Orthokinetic flocculation is the predominant mechanism in water treatment. In addition, sludge blanket or solids contact clarifiers cause differential and fluctuating velocities, which lead to particle collisions and aggregation. An understanding and knowledge of the kinetics created by orthokinetic flocculation are required to optimize the design of flocculators. The parameters that define the rate of aggregation also define the physical dimensions of the process equipment and basin.

This section presents information on kinetic models that can be used to design flocculation systems. Typical design examples are presented.

Orthokinetic Flocculation

In systems that are mixed (velocity gradients are induced), the velocity of the fluid varies both spatially (from point to point) and temporally (from time to time). The spatial changes in velocity are termed the *velocity gradient, G,* sec^{-1}. In water treatment plants, mean velocity gradients of 10 to 100 sec^{-1} are typical for flocculation. Flocculation tanks are ineffectual until the colloidal particles reach a size of 1 μm, through contacts provided by Brownian motion. For example, flocculation tanks cannot aggregate viruses, which are 0.05 μm in size or smaller until they are adsorbed or enmeshed in larger flocs or particles.

Camp developed the following equations for computing the velocity gradients for mixing chambers and pipes:[34]

Mechanical mixing:

$$G = \left[\frac{P}{\mu V}\right]^{1/2} \tag{10–19}$$

$$G = 425\left[\frac{hp_w}{t}\right]^{1/2} \tag{10–20}$$

Ports and conduits:

$$G = \left[\frac{P}{\mu V}\right]^{1/2} \cdot V^{3/2} \qquad (10\text{–}21)$$

$$G = 172 \left[\frac{f}{D}\right]^{1/2} \cdot V^{3/2} \qquad (10\text{–}22)$$

Baffled chambers:

$$G = 178 \left[\frac{H}{t}\right]^{1/2} \qquad (10\text{–}23)$$

where:

P = power input, ft lb/sec (N − m/s)
V = volume, cu ft (m^3)
μ = dynamic viscosity, lb sec/sq ft (kg − s/m^2)
f = Darcy-Weisbach friction factor based on roughness height of 0.00085 ft (0.2591 mm)
D = conduit diameter, ft (m)
H = headloss, ft (m)
hp_w = wire to water energy input (hp)

Camp analyzed several flocculation basins and determined that flocculation basins having values in the range of 2×10^4 and 2×10^5 for the nondimensional parameter Gt performed satisfactorily.[48] G-values for satisfactory performance were found to be in the range of 20 to 74 sec^{-1}.

For paddle-type mechanical flocculators, the power dissipated in the liquid, P, can be determined from:

$$P = \frac{1}{2} C_d A \rho v^3 \qquad (10\text{–}24)$$

where:

P = power dissipated, ft lb/sec (N − m/s)
C_d = coefficient of drag
= 1.8 for flat plates
A = area of paddle, sq ft (m^2)
ρ = density of liquid, slugs/cu ft (kg/m^3)
v = velocity of paddles relative to liquid
≅ 0.5 to 0.75 × velocity of paddle, ft/sec (m/s)

Equation 10–24 should be integrated for an elemental area if velocity changes occur along the length of a paddle due to distance from the shaft.[49]

There are two important conclusions that can be drawn from past experience:[50,51]

- There is a minimum time below which no flocculation occurs, whatever the value of G (i.e., aggregations and breakup are equal).
- The use of reactors in series can significantly reduce the overall detention time for the same degree of treatment.

The second conclusion has been identified and confirmed by other investigators and the recommended minimum number of compartments is three to minimize short circuiting.[52,48]

One study has indicated that optimum values exist for the flocculation time, T, and the velocity gradient, G.[53] Bench-scale studies with alum as the coagulant and kaolin clay indicated that flocculation time should be in the range of 20 to 30 minutes. Increases in time did not improve flocculation significantly. The optimum value of G is computed from the equation:

$$(G^*)^{2.8} \cdot T = K \tag{10–25}$$

where:

G^* = optimum velocity gradient, sec^{-1}
T = flocculation time, minutes
K = constant
= 4.9×10^5 for alum concentration of 10 mg/L
= 1.9×10^5 for alum concentration of 25 mg/L
= 0.7×10^5 for alum concentration of 50 mg/L

Equation 10–25 can be simplified by combining the empirical results into a single approximate equation as follows:[54]

$$(G^*)^{2.8} \cdot T = \frac{44 \times 10^5}{C} \tag{10–26}$$

where C = alum concentration, mg/L in the range of 10 to 50 mg/L.

The optimum value of G was defined as the velocity gradient that minimizes residual turbidity by flocculation and settling. The range of optimum G was 20 to 50 sec^{-1}.

The expression $(G^*)^{2.8}\ CT$ is similar to the expression GCT suggested by other investigators as the design parameter.[36,55]

Studies have shown tapered flocculation with a diminishing velocity gradient to be more efficient than uniform velocity gradient flocculation.[44,48,56] The recommended overdesign factor in order to handle variation in flow rates is approximately 1.5.[57] This gives an objective value for providing operational adjustments in the design of flocculation systems.

FLOCCULATION ALTERNATIVES

The following types of mechanical mixing devices are typically used in water treatment flocculation:

- Paddle or reel-type devices
- Reciprocating units (walking beam flocculator)
- Flat-blade turbines
- Axial flow propellers or turbines

Typical units are shown in Figure 10–13. The spatial distributions of velocity gradients that the units produce are shown in Figure 10–14 for some of the mixers.

The paddle or reel-type devices are mounted horizontally or vertically and rotate at low speeds (2 to 15 rpm). Design is based on limiting the tip speed of the paddle farthest from the center axis to 1 to 2 ft/sec. Argaman and Kaufman used a stake and stator device for their studies.[50] This unit is similar to reel-type units but is mounted vertically and was found to be superior to a turbine.

Walking beam flocculators are driven in a vertical direction in a reciprocating fashion. The unit contains a series of cone-shaped devices on a vertical rod. The cone devices impart energy to the water as they move up and down, thereby creating velocity gradients. The manufacturer's literature should be consulted to design this unit.

Turbines are flat-bladed units connected to a disc or shaft. The flat blades are in the same plane as the drive shaft. The blades can be mounted vertically or horizontally and typically operate at 10 to 15 rpm. Walker found that plate turbines are effective up to a G-value of 40 sec^{-1}, but produced high-velocity currents at G-values greater than 45 sec^{-1}.[58] His suggested design criterion is to limit the maximum peripheral velocity to 2 ft/sec for weak floc and 4 ft/sec for strong floc. Other investigators have found that turbines are the least effective units for flocculation.[42,50,58]

The axial flow unit "pumps" liquid because the impeller has pitched blades. This unit may be installed vertically or horizontally. Typically, these units are high-energy flocculation devices operating at 150 to 1,500 rpm, and there is no limitation on the tip speed. Hudson and Walker favor these units because they are simple to install and maintain, and they produce uniform turbulence in the flocculator.[42,58]

DESIGN EXAMPLES-FLOCCULATION

The design examples presented will be continuations of the examples presented for the rapid-mix design.

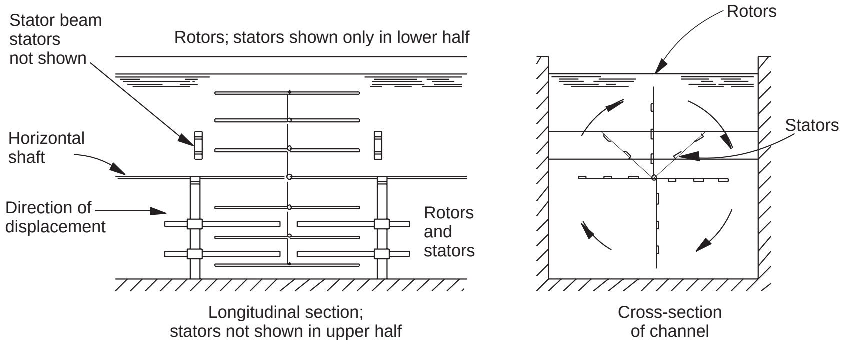
Example 1 (In-Line Blender)

Determine the optimum velocity gradient, based on alum doses of 15 to 25 mg/L. The advantages of compartmentalization have been shown before. Assume a four-compartment flocculator. Using Equation 10–26:

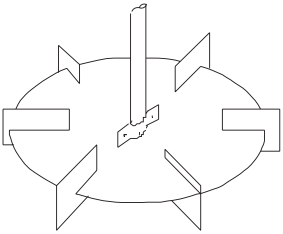
$$(G^*)^{2.8}T = \frac{44 \times 10^5}{15} \text{ to } \frac{44 \times 10^5}{25}$$

$$= 2.933 \times 10^5 \quad \text{to} \quad 1.76 \quad \times \quad 10^5$$

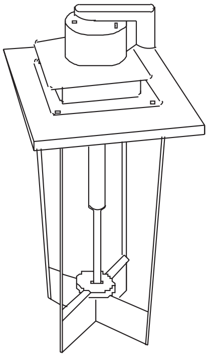
Using a flocculation time of 30 minutes because longer times do not increase efficiency significantly:



a) Paddle type with rotors and stators

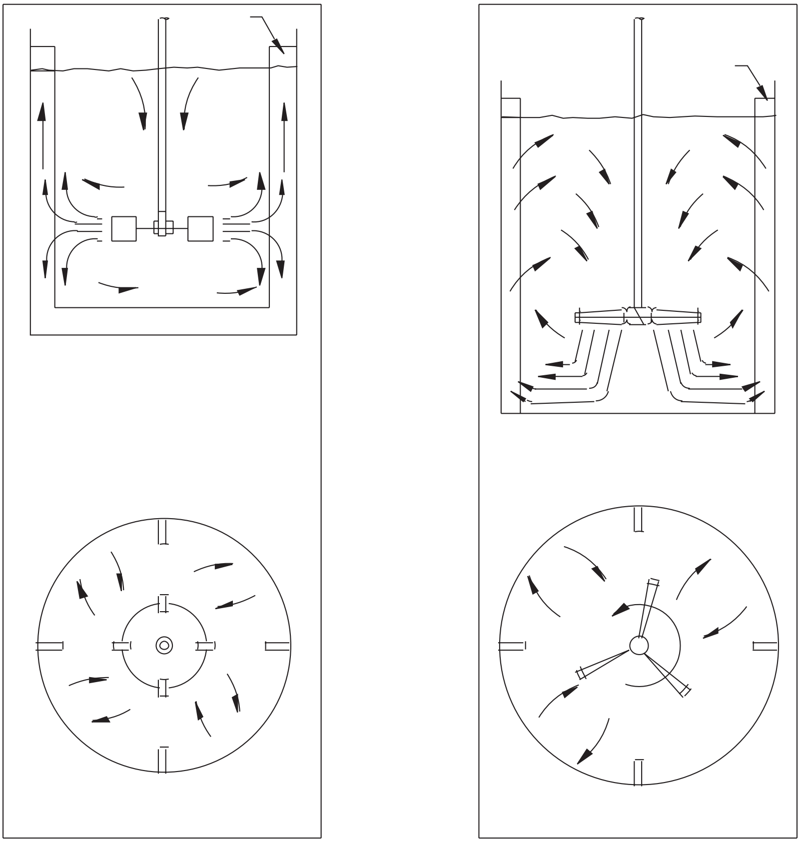


b) Plate turbine type



c) Axial flow propeller type with straightening vanes

Fig. 10–13. Typical flocculation units (*Source:* Reference 41. Courtesy of Butterworth-Heinemann.)



d) Schematic of radial flow pattern in baffled tank

e) Schematic of axial flow pattern in baffled tank

Fig. 10–13. (*Continued*).

$$(G^*)^{2.8} = \frac{2.933 \times 10^5}{30} \quad \text{to} \quad \frac{1.76 \times 10^5}{30}$$

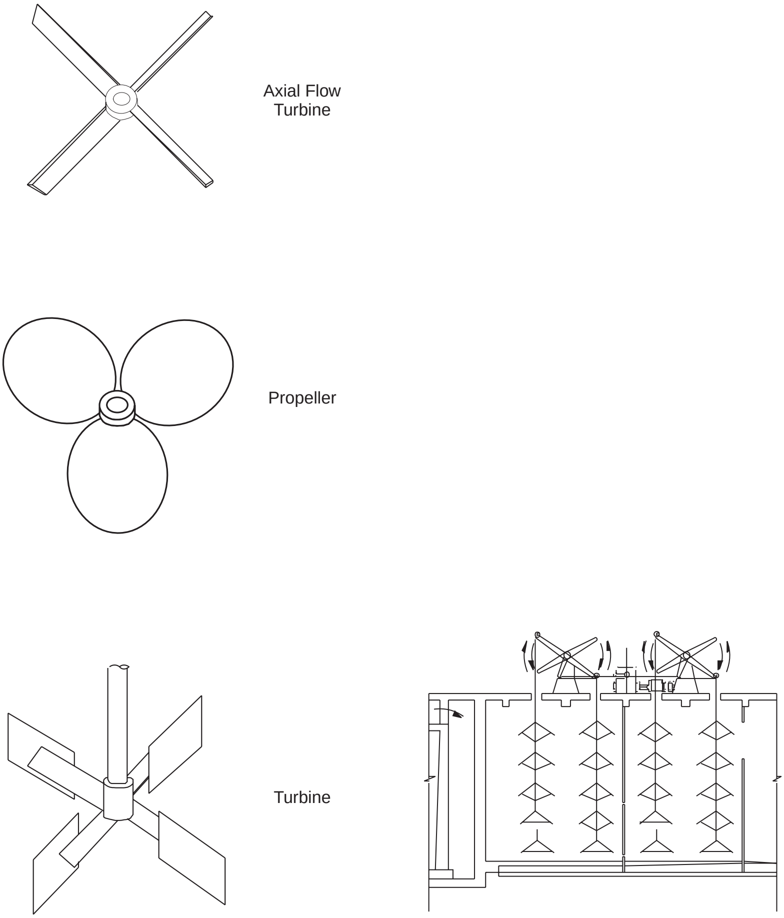
$$= 9780 \text{ to } 5870$$

$$G^* = 26.6 \text{ to } 22.1 \text{ sec}^{-1}$$

Check G^*-values for a detention time of $T = 20$ minutes. The corresponding range of G^* values is 30.7 to 25.6 sec^{-1}.

Use an optimal G^*-value of 25 sec^{-1} and a detention time of 20 minutes. The product Gt is 3.0×10^4, which is within the recommended guideline of 10^4 to 10^5.

To incorporate the advantages of tapered flocculation, the optimum design of G^* of 25 sec^{-1} would be used in the third compartment. The G-values are tapered on either side to yield $G_1 = 45\ \text{sec}^{-1}$, $G_2 = 35\ \text{sec}^{-1}$, $G_3 = 25\ \text{sec}^{-1}$, and $G_4 = 20\ \text{sec}^{-1}$ in the four compartments.



f) Basic impeller styles

g) Walking beam flocculator

Fig. 10–13. (*Continued*).

The detention time of each of the four equal-sized compartment is 5 minutes.

$$\text{Design flow rate} = 3.0 \times 1.547 = 4.64 \text{ cu ft/sec } (0.087 \text{ m}^3/\text{s})$$

$$\text{Volume in each compartment} = 4.64 \times 5 \times 60 = 1390 \text{ cu ft } (39.4 \text{ m}^3)$$

Assume square-shaped compartments and a water depth of 10 feet. Then:

$$\text{Side dimension} = \left[\frac{1390}{10}\right]^{1/2} = 11.8 \text{ ft.} \qquad \text{Use 12 ft (3.6 m)}$$

Each compartment would be 12 ft × 12 ft × 12 ft (3.6 × 3.6 × 3.6 m) deep, which provides a 2 ft (0.6 m) freeboard allowance.

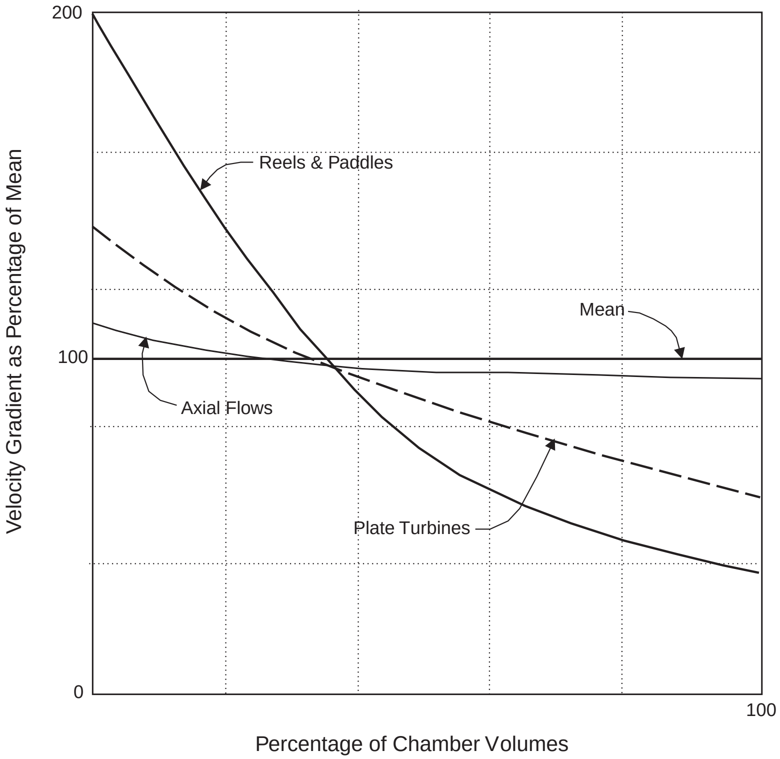


Fig. 10–14. Distribution of velocity gradients in mechanical flocculators (*Source:* Reference 42)

For simplicity, an axial flow impeller will be used for this example. For the first compartment, $G = 40\ \text{sec}^{-1}$. Provision should be made to operate the unit at variable speeds to allow adjustment of the *G*-value. Assume *G*-values in the range of 90 to 20 sec^{-1}. Use the highest viscosity with the highest probable *G*-value to obtain the power requirements:

$$P = G^2 \mu V$$

Viscosity at 46.4°F (8°C),

$$\begin{aligned} \mu &= 1.3872 \text{ centipoise} \times 2.088 \times 10^5 \text{ lb/sec/sq ft centipoise} \\ &= 2.896 \times 10^{-5} \text{ lb sec/sq ft } (1.377 \times 10^{-3}\ \text{N} \cdot \text{s/m}^2) \\ P &= (90)^2 \times 2.896 \times 10^{-5} \times (12 \times 12 \times 14) \\ &= 473 \text{ ft-lb/sec } (0.64 \text{ kW}) \end{aligned}$$

Assume overall efficiency of the motor and drive as 75 percent. Then:

$$\text{Motor horsepower} = \frac{473}{550 \times 0.75} = 1.15 \text{ hp } (0.85 \text{ kW})$$

Use a standard 1.25 hp (1.0 kW) motor. Now check the rotational speed and G-values.

$$P = \frac{k}{g} \rho N^3 D^5$$

Assume $D = 2$ ft (0.61 m); then each blade is 1 ft (0.305 m) long.

$$(1.25 \times 550 \times 0.75) = 1.0 \times 1.94 \times N^3 \times 2^5$$

$$N = 2.0 \text{ revolutions/sec}$$

$$= 120 \text{ rpm}$$

Use a 4-to-1 variable-speed drive, and check the value for G at the lowest speed (120/4 = 30 rpm).

$$P = 1.0 \times 1.94 \times \left(\frac{30}{60}\right)^3 \times 2^5$$

$$= 7.76 \text{ ft-lb/sec } (10 \text{ W})$$

and:

$$G = \left[\frac{7.76}{2.896 \times 10^{-5} \times (12 \times 12 \times 14)}\right]^{1/2} = 11.5 \text{ sec}^{-1}$$

The range in G-values is satisfactory. For standardization, put the same unit in all four compartments. It may be possible to reduce the horsepower in the third and fourth compartments because of the lower G-values required.

Example 2 (Paddle-Type Unit)

Assume the plant flow rate is 20 mgd (75. ML/d) and the lowest temperature is 53.6°F (12°C). The alum doses, based on jar tests, ranged from 20 to 30 mg/L. From Equation 10–26, determine the optimum G-value.

$$G = 24 \text{ sec}^{-1} \quad \text{and} \quad 20.8 \text{ sec}^{-1} \text{ respectively} \qquad \text{for } T = 30 \text{ minutes.}$$

Use a G-value of 24 sec^{-1} and a detention time of 30 minutes. The product Gt is 4.32×10^4, which is within the recommended range.

Assume a three-compartment system with the optimum G-value in the middle. The G-values on either side would be 30 sec^{-1} and 15 sec^{-1}.

Assume a rectangular-shaped flocculator and a 12-foot sidewater depth:

$$\begin{aligned}\text{Volume} &= 20 \times 1.547 \times 30 \times 60 \\ &= 55{,}692 \text{ cu ft } (1576 \text{ m}^3)\end{aligned}$$

Assume width of basin, *W*, is one-third of the length. Then:

$$\begin{aligned}W \times 3W &= \frac{55{,}692}{12} \\ W &= 39 \text{ ft.} \qquad \text{Use 40 ft (12 m).} \\ \text{Length of basin} &= 120 \text{ ft (36 m)} \\ \text{Width} &= 40 \text{ ft (12 m)} \\ \text{Water depth} &= 12 \text{ ft (3.6 m)}\end{aligned}$$

Now, for each third of length:

$$\begin{aligned}P &= G^2 \mu V \\ &= 24^2 \times 2.896 \times 10^{-5} \times (40 \times 40 \times 12) \\ &= 320 \text{ ft lb/sec (0.4 kW)}\end{aligned}$$

$$\text{Motor horsepower} = \frac{320}{550 \times 0.75} = 0.8 \text{ hp.} \qquad \text{Use 1.0 hp (0.75 kW).}$$

Limit the tip speed (v_p) to 1.5 ft/sec (0.46 m/s).

Compute the area of the paddles:

$$\begin{aligned}P &= \frac{1}{2} C_d A \rho \nu^3 \\ 320 &= \frac{1}{2} \times 1.8 \times A \times \frac{62.4}{32.2} \times (0.75 \times 1.5)^3 \\ A &= 129 \text{ sq ft } (12 \text{ m}^2)\end{aligned}$$

The diameter of the farthest paddle is the depth of water less a clearance allowance. Use an 11-foot (2.13-m) diameter.

$$\begin{aligned}1.5 \text{ ft/sec} &= 2\ \pi r n \\ &= 2\pi \frac{11}{2} \times \frac{n}{60} \\ n &= 2.6 \text{ rpm}\end{aligned}$$

Assume eight paddles per shaft and one shaft per third section. Each paddle is 36 feet (11 m) long.

$$\text{Width} = \frac{129}{8 \times 36 \times 1} = 0.45 \text{ ft} = 6 \text{ in (150 mm)}$$

The outer paddle is positioned at a radius of 5½ feet (1.68 m) to the outside edge and the inner paddle at a radius of 3¾ feet (1.14 m) to the outside edge.

COAGULATION CONTROL AND MONITORING

Techniques Available

Since achieving proper coagulation has been a universal problem for water treatment operators for many years, a wide variety of techniques have been developed for controlling the coagulation process. Most of these involve in-line water quality analysis and laboratory tests, the results of which then must be manually transferred to the full-scale plant operation by the plant operator.

Whether the coagulant dose that provides the optimum result in the laboratory tests will also provide optimum results on a plant scale depends on whether the same efficiency of mixing is achieved in both cases, which is unlikely. Also, the fact that the laboratory testing is a batch procedure results in an inherent time lag in responding to changes in raw-water conditions. This lag may be only an hour if the operator is on duty and alert to raw-water conditions, but it may be several hours if the operator is off duty or involved in another task, such as equipment maintenance. This method may give satisfactory—even if not optimum—results when applied to a raw water of relatively uniform quality that contains only a moderate amount of organic turbidity. For low-turbidity waters and waters containing large amounts of organic material, the optimum coagulation dosages for particulate removal may be difficult to determine. However, for waters that have high organic material (high TOC), higher coagulant dosage is usually required to achieve adequate TOC removal, leaving the particulate a lesser concern. With today's advanced in-line and laboratory particle and TOC measuring equipment, the process monitoring and control can be easily achieved compared to a decade ago.

Available coagulation control techniques fall into three general categories: conventional and modified jar tests (where supernatant quality, such as turbidity, floc formation time, and floc density, are analyzed); techniques based on particle charge; and techniques based upon the turbidity of filtrate using a pilot-scale filter column.

Jar Tests

Selection of the best coagulant is most often based on jar tests. Detailed procedures for conducting jar tests are described in AWWA Manual M37.[59] Also, a standard jar test procedure is described in AWWA Manual M12, titled *Simplified Procedures for Water Examination*. A standard testing plan is required to provide a meaningful comparison of coagulants or coagulation aids. The procedure basically consists of adding varying coagulant dosages to several water samples contained in beakers, mixing them simultaneously at the same speed with a gang mixer, allowing them to settle, and measure the settled water quality, such as turbidity and color.

The jar tests defined by Manual M12 are intended to identify the chemical treatment characteristics that would most benefit the full-scale plant. Evaluating the many chemicals available for use on a bench-scale basis in the laboratory provides an invaluable means to evaluate a wide variety of treatment chemicals, including: coagulants, coagulant aids, alkalinity/pH adjustment chemicals, softening chemicals, adsorption additives, and oxidation chemicals.

The manual also notes several considerations that may affect the outcome of jar testing, noting the importance of maintaining consistency in the jar testing conditions to avoid the potential impacts that even minor differences in conditions would have. Handling of each series of jar tests in a like manner is critical to the successful assessment of different testing conditions, such as flocculation and sedimentation characteristics. Minimizing the effect of temperature changes on the jar testing is recognized by Manual M12. Water sample collection is delayed until all preparations are completed to reduce the temperature fluctuations from collection time to laboratory room temperatures.

The importance of simulating plant conditions as closely as possible cannot be overemphasized. The manual states that ‘‘dosing solutions or suspensions should be prepared from the stock materials actually used in plant treatment. Distilled water used for the preparation of lime suspensions should be boiled for 15 minutes to expel the carbon dioxide and then cooled to room temperature before the lime is added.’’ Details are presented in AWWA Manual M12 on the preparation of coagulant dosing solutions and suspensions.

In testing polymeric coagulants or coagulant aids supplied by cooperating polymer manufacturers, solutions of these products should be prepared in accordance with manufacturer specifications.

Techniques Based on Particle Charge

Zeta Potential The first technique of using particle charge as a possible means of coagulation control was the zeta potential technique. It has received a great deal of attention and inspired numerous research activities since it was introduced. A few references are listed here to provide the reader with a more extensive bibliography as well as a more detailed introduction to the subject.[60–63]

Zeta potential (expressed in millivolts) is a measure of the electrical potential between the bulk liquid and the layer of counterions surrounding the colloids. Since particles with like charges repel each other and almost all natural colloids are negatively charged in water, suspended natural colloids are resistant to coagulation by nature. When this negative zeta potential is reduced, the repulsive forces are likewise reduced. If the system is agitated gently, the colloids will flocculate. In the treatment process, the reduction of zeta potential is accomplished by the addition of a positively charged ion or complex from such coagulants as aluminum sulfate, the iron salts, and cationic polyelectrolytes.

The procedure for measuring zeta potential is somewhat tedious for routine use in the average water treatment plant. Black has presented a simplified procedure that utilize the Briggs cell in conjunction with a microscope for zeta potential measurement. Similar to the jar test described above, the general procedure involves varying the coagulant dosage and measuring the resulting zeta potential. However, each water requires comparative tests to determine the correlation between zeta potential and finished water turbidity in the plant. Organic colloids, such as those that impart true color, generally require zeta potentials near zero, whereas clay-related turbidity is best removed at somewhat slightly negative zeta potentials. Typical values for optimum coagulation range from +5 to −10 millivolts, depending upon the nature of the material to be removed.

Although zeta potential measurements are useful as a research tool, it is generally agreed that this method is not easily adapted to the typical treatment plant because of the considerable degree of skill and patience required to make the measurements and the amount of interpretation required to make the data useful. It has been the authors' experience in visiting plants that have purchased zeta meter equipment for routine operation that generally it will be found stored and inoperative. Other observers also have reported a recent decline in the popularity of zeta potential as a control technique.[64] It is subject to the same shortcomings as any other batch test in that sudden changes in raw-water conditions may not be detected until a poor-quality finished water is produced in the full-scale plant.

Streaming Current Detector A streaming current detector may be used to provide a continuous measure of the relative charges of coagulated particles. This technique involves placing a sample in a special cylinder containing electronic sensing electrodes at the top and the bottom. A loose-fitting piston is then partly submerged into the sample and is reciprocated along its axis to produce an alternating current between the electrodes when the cylinder contains moving charges. A synchronous motor drives the piston and synchronous rectifier switch, by means of which the alternating current generated by the alternating fluid motion is made to register on a dc meter (Fig. 10–15). An amplifier with adjustable negative feedback is used to provide an output proportional to the current collected by the electrodes. Readout may be by a microammeter, with calibration in arbitrary units. The alternating current is analyzed and related

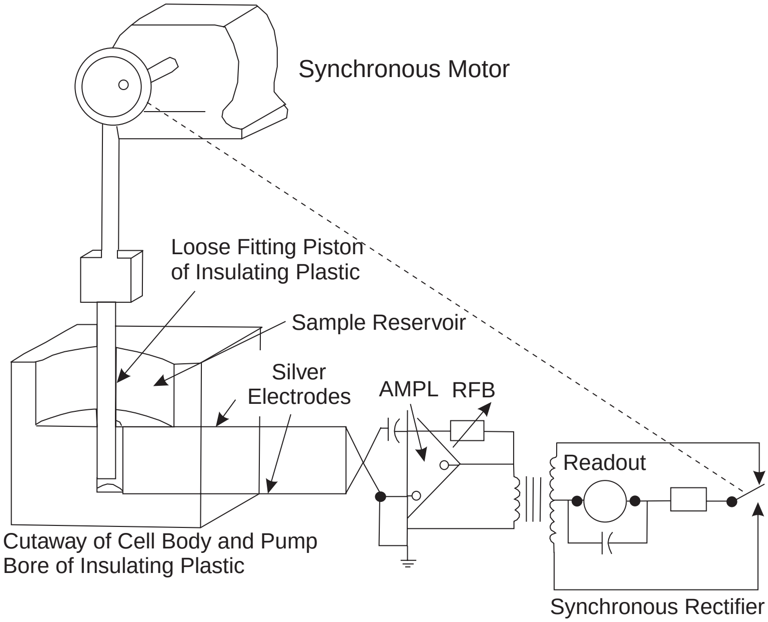


Fig. 10–15. Simplified diagram of a streaming current detector (*Source:* Reference 71; used with permission)

to zeta potential, which can be used to control coagulant dosage. A flow-through cylinder and recording ammeter may be built into the system to permit continuous monitoring. Almost all of the successful applications of the instrument have involved the titration of charge-influencing materials, either batchwise or continuously. In these applications, uncertainty as to what is measured is of little consequence as long as the addition of anionic and cationic materials changes the reading in a reproducible manner. Titrations are carried out in the same fashion as acid–base titrations with a pH meter. Continuous readings are obtained while one material is added to the other.

The limitations of using streaming current for continuous coagulation control are similar to that of zeta potential approach. For example, since this approach uses particles' electrical charge as the only parameter to evaluate coagulation performance, it may not be suitable when anionic polymers are used as coagulation aid to provide bridging effect. The readout of streaming current for such systems may indicate that the coagulation is poor (because particles are even more negatively charged compared to that in original raw water), whereas the actual performance may be satisfactory due to bridging effect. As with zeta potential, the data must be correlated with the usual indices of plan performance, as there may not be any consistent relationship between charge and filtered water clarity even at a given plant for various seasons of the year. However, the continuous nature of the streaming current detector may make it attractive in some instances where cationic coagulants are used in a water that shows little variation from season to season.

Colloid Titration A simple titration technique using an indicator that changes colors when a solution is titrated to electric neutrality has been developed.[65,66] An excess amount of positively charged polymer is added to the naturally negatively charged water. It is then back-titrated with a standard negatively charged polymer. An empirical correlation is used to relate the volume of titrant to the proper coagulant dosage, in a manner analogous to zeta potential measurement.

Like the streaming current detector, colloid titration has the advantage of using a larger, more representative sample than that used in zeta potential measurements. Since its conclusions are based on the electrokinetic properties of a suspension, its results are subject to the same limitations given for the streaming current detector and the zeta potential devices.

Continuous Filtration Techniques

Pilot Filters The goal of the water treatment plant should be to produce the minimum possible filter effluent turbidity at the minimum chemical cost. The best measure of the efficiency of the coagulation-filtration steps would be the direct continuous measurement of the turbidity of coagulated water that has passed directly through a pilot granular filter. The application of continuous turbidity monitoring equipment to the effluent of the plant-scale filters should be a must for monitoring plant performance. However, a real-time turbidity meter also has limited value as a control technique because of the substantial lag time between the point of chemical coagulant addition and the point of filtrate turbidity monitoring. For example, improper coagulation of the incoming raw water may not be immediately apparent at the discharge of the plant filters until the water has flow through the clarifier, which usually has a hydraulic detention time of 1½ to 3 hours.

The pilot filter technique is applied by sampling plant-treated coagulated water from the discharge of the plant rapid mix basin before it enters to flocculation basin and clarifier. This sample stream is then passed through a small (usually 4½-inch-diameter) pilot filter to determine whether the coagulant dose is proper, by continuously monitoring the pilot filter effluent turbidity. This technique provides a continuous, direct measurement of the turbidity, which is achieved by filtration of water that has been coagulated in the actual plant. Thus, no extrapolation from small-scale laboratory coagulation experiments is required. The only purpose of this test is to determine the proper coagulant dose; it is not to predict the length of filter run, nor to determine the optimum filter aid dose, nor to predict the rate of headloss buildup. Although pilot filter columns have been used for other purposes as mentioned above, the technique of interest here is for monitoring the coagulation process.

The pilot filter technique has the advantages of offering a continuous monitoring of the plant-scale coagulation process with a minimum lag time. Filtering the water through the pilot filter yields immediate information about the adequacy of the coagulant dose. In a typical situation, correctness of coagulant dose is determined within 10 to 15 minutes after the raw water enters the plant. Experience at many locations shows that the pilot filter effluent turbidity is a very accurate prediction of the plant-scale filter effluent turbidity.

The mixed-media filter bed design described in Chapter 12 is often used in the pilot filter. The mixed-media design has the ability to accept the high solids load associated with most unsettled, coagulated waters without excessive headloss buildup. The turbidity of the pilot filter is monitored continuously and recorded. High turbidity in the filter effluent could result from either an improper coagulant dose or a breakthrough of floc from a properly coagulated water. To ensure that breakthrough does not occur, supplemental polymers are injected into the pilot filter influent line. These additional polymer doses may shorten the pilot filter run times but can prevent breakthrough. Typically, it is desirable to backwash the filter every 1 to 3 hours. To provide a continuous monitoring of the coagulation process, two pilot filters are used in parallel. The system is equipped so that the filter is automatically backwashed on a high headloss signal. When one pilot filter enters the backwash cycle (which requires only about 10 minutes), the other pilot filter is automatically placed in service. Where very high raw-water turbidities can be expected, a miniature flocculator-tube settler device can be installed ahead of the pilot filters to ensure reasonable filter run times even during periods of high turbidity. Typically, the pilot filters are contained in a console unit or in the plant control panel, which may also house turbidimeters for monitoring the plant-scale filter effluent turbidity.

At least two manufacturers (Neptune Microfloc and Turbitrol Co.) offer commercial pilot filter systems, both based on the same principle but differing somewhat in mechanical aspects.

The pilot filter systems can be used to accurately indicate coagulation conditions with a minimum lag time, so that the operator can make the necessary adjustments in the plant chemical feed. Alternatively, the coagulant feed can be controlled automatically.

Design for Automated Coagulant Control This system is used in conjunction with a pilot filter system and automatically varies the plant coagulant dosage to maintain the effluent turbidity from the pilot filter at the desired set point value regardless of variations in raw-water quality and other related factors. Other chemicals can be adjusted automatically by this system in direct relation to the coagulant dosage. The coagulant control system normally consists of a pilot filter system, an automatic control unit, and switches and controls for the chemical feed equipment. The output signal from the automatic control unit to the chemical feeders can be a time duration, current, pneumatic, or other standard instrumentation control signal. In general, the time duration signal is found to be reliable and economical and is used quite extensively.

Basically, this automated control is accomplished by comparing a 0- to 10-NTU signal from the pilot filter turbidimeter with the desired turbidity value. If the turbidity signal is greater than the desired turbidity, the alum dosage will be increased. If the input turbidity is less than the set point turbidity, the alum dosage is automatically decreased. The plant operator establishes the desired water quality through a set point potentiometer, and the unit constantly adjusts the coagulant dosage to maintain the set point value.

The system may incorporate provisions for automatic plant flow pacing. To allow for flow pacing, a potentiometer must be integrated into the plant raw-water flow measurement so that the potentiometer wiper is at zero resistance with no flow and at maximum resistance at maximum plant flow. When the flow pacing feature is incorporated, the dosage output will be automatically adjusted for changes in plant flow. At the same time, changes in required coagulant dosage will be regulated by the control unit. The system may be provided with an override feature so that the plant coagulant feed system may be operated in various modes.

Since raw-water quality will affect coagulation efficiency, variables that could affect the pilot filter turbidity must be controlled. The most important variable is pH. If the pH of the coagulated water is not maintained in the proper range for optimum filtration, the unit cannot function properly.

Waters with low natural alkalinity will generally require addition of artificial alkalinity to maintain the coagulated water pH at an optimum value for filtration. In most cases, the alkalinity requirement is a direct function of the coagulant dosage and therefore the feed pumps for both chemicals can be proportioned over the entire range of expected coagulant dosages so that supplementary artificial alkalinity feed is not required.

Filter Aid Control by Interface Monitoring This technique has been developed in conjunction with dual-media filters. A sample of water is removed from the filter at a point near the interface between the coarse anthracite coal and the fine sand. The turbidity of this sample is continuously monitored and recorded, as is the filter effluent turbidity. The sample at the interface is obtained with a device that is a specifically constructed well screen with slits small enough that the coal and sand cannot pass through. The interface screen is placed 2 inches above the fine sand prior to anthracite placement. The turbidimeter is placed as close as possible to the filter. Samples flow by gravity to the turbidimeters. This prevents air bubbles, which frequently occur when turbidimeter samples are pumped, and also provides for the fastest possible response time.

The interface turbidity sample is designed as a tool to aid the treatment plant operator in obtaining optimum performance from a dual-media filter. The turbidity value obtained at the interface indicates whether floc removal is occurring within the filter bed. For instance, a high interface turbidity and a low filter effluent turbidity indicate that the proper coagulant dosage is being used but that significant amounts of floc are penetrating through the anthracite layer and must be removed by the fine sand. If the sand is forced to carry too much of the load, the headloss buildup becomes excessive or breakthrough of turbidity could occur in the effluent. When floc removal does not occur in the anthracite layer while the proper coagulant dose is applied, the physical strength of the coagulated flocs may be weak (so the flocs break up as they pass through the anthracite layer), and therefore polymers should be applied to increase the floc strength.

This method works with any material that can be successfully used as a filter aid or conditioner. The filter aid dosage must be started at a low dosage (even at zero dosage) and incrementally increased as the filter run processes until the desired floc removal is obtained in the coal layer. If an overdose is applied, it is usually impossible to use the interface concept until the filter has been washed.

Turbidity Monitoring

With the currently available, low-cost turbidimeters, which are accurate and require little maintenance, every municipal water plant in the United States, regardless of size, should provide a continuous record of the quality of its final product. Turbidity is an important parameter in that it:

- Reflects the efficiency of the coagulation process and the overall treatment provided.
- Is related to probability of escape of pathogenic organisms from the treatment plant.
- Is a sensitive indicator of the aesthetic acceptability of the product to the consumer.
- Is required by the Surface Water Treatment Rule.

For the measurement of a very small amount of turbidity, the principle of light scattering (nephelometry) is applied. In this process, as the light beam passes through the liquid, a portion of the light is reflected at right angles to the light beam by the particles in the liquid. The amount of reflected (scattered) light depends directly upon the amount of turbidity present in the liquid. This is similar to the common phenomenon of sunlight streaming through a window being reflected by otherwise invisible dust particles in the air. If the liquid is entirely free of turbidity, no scattered light will reach the photocells and the indicating meter will read zero; thus, increasing turbidity will result in an increased meter reading.

The advantages of using light scattering for turbidity measurement are:

- A very strong light source can be used to provide a high degree of sensitivity, and thus very small amounts of turbidity can be measured accurately.
- The meter reading or output from the instrument is zero when the turbidity is zero.

Units of this type (such as manufactured by HACH Company) are available with ranges of 0 to 0.2, 0 to 1, 0 to 3, and 0 to 30 NTU. A water sample stream of approximately 0.5 gpm at a head of 6 inches of water is required to flow through the in-line turbidity meter.

The conventional design of nephelometry as described above is intended for water samples that contain low and medium turbidity and are not suitable for high-turbidity water. A new design employing “surface scatter” was developed to overcome this limit (Fig. 10–16). A very narrow beam is directed onto the surface of the liquid at a

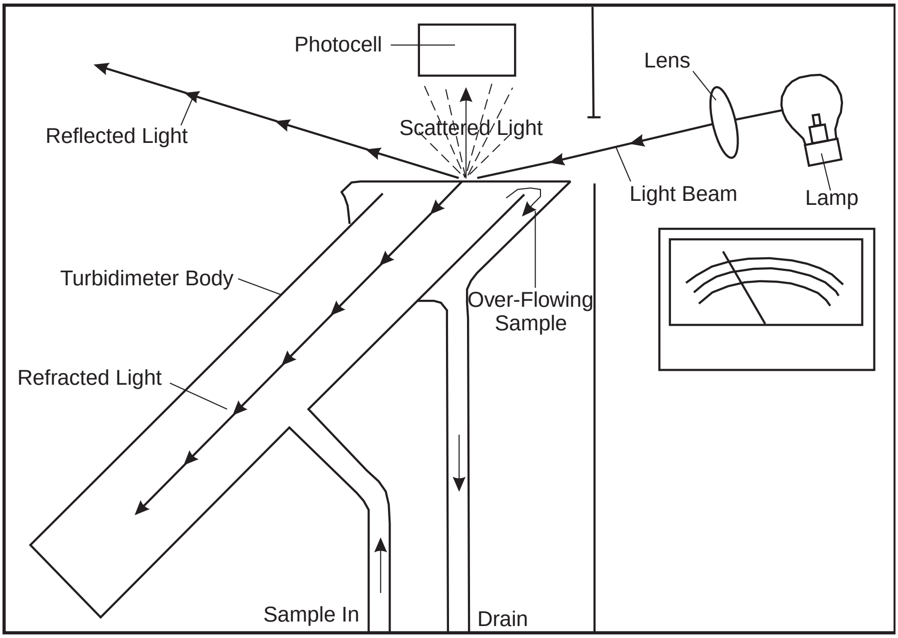


Fig. 10-16. Schematic diagram of surface scatter turbidimeter (Courtesy of Hach Co.)

15-degree angle. Part of the beam is reflected by the water surface and escapes to a light trap. The remaining portion enters the water at approximately a 45-degree angle. If particles of turbidity are present, light scattering will take place and some of the scattered light will reach the photocell. With this ‘‘surface scatter’’ design, there is virtually no upper limit for turbidity measurement. This type of turbidity meter can be calibrated by using a Jackson candle turbidimeter and measuring the turbidity of the same water that flows through both instruments. The in-line turbidimeter is adjusted so that its output reading corresponds to the Jackson candle reading. Ranges as high as 0 to 5,000 are available, and this instrument can be used for measuring the turbidity of raw water or the settling basin effluent. A 0.25–0.50-gpm (0.016 to 0.032 L/s) sample stream is required.

ENHANCED COAGULATION PROCESSES

In addition to particulate matter, coagulation is also capable of removing soluble contaminants, such as arsenic and true color, from the waters. The majority of true color is imparted by natural organic matter (NOM). Since NOM molecules are very complicated and do not have a definite structure, total organic carbon (TOC) concentration is a surrogate commonly used to quantify NOM. While NOM itself causes aesthetic problems (such as taste and odor) rather than health concern, the reaction between NOM and chlorine (during the chlorination process) could generate carcinogenic disinfection by-products (DBPs). The Stage I D/DBP Rule promulgated in 1999 set lower

MCLs for DBPs (e.g., 80 μg/L for TTHM and 60 μg/L for HAA5). To comply with this more stringent regulation, many utilities began relying on using coagulants to remove DBP precursors, mainly NOM.[67] Usually a higher coagulant dosage is required to achieve TOC removal than what is required for optimal turbidity removal.[68] With higher coagulant dosages applied, the formation of large-heavy flocs can be easily achieved and therefore the operating parameters (such as coagulant dosage and pH) are usually optimized for organic removal rather than for particulate removal. This type of TOC-emphasized coagulation is referred to as enhanced coagulation (EC).

TOC removal by coagulation increases with decreased pH and increased coagulant doses. Although both aluminum- and iron-based coagulants can remove significant amounts of TOC, ferric coagulants seem to have slightly higher sorption capacity for NOM than alum under the same condition. For waters that contain high TOC, coagulation at lower pH is usually required to achieve desirable TOC removal efficiency. Ferric coagulants are usually used (at pH as low as 3.8) under such circumstances, since the solubility of aluminum becomes significant at pH below 5.5. The pH of treated water should be adjusted back to 7.5~8.2 for corrosion control. When water coagulated at lower pH (e.g., pH 4.0) is readjusted to neutral pH after sedimentation, some reflocculation may occur and cause increased turbidity. This post-flocculation may be due to the precipitation of soluble iron or to the recoagulation of pinpoint ferric hydroxide flocs that do not settle at lower pH (due to high positive charge on the particle surfaces). These colloidal ferric particles will likely be recoagulated when the pH of settled water is adjusted to neutral, since the positive particle charge reduces at neutral pH, thereby reducing the electrostatic repulsion among particles.

Using the coagulation process for the removal of other trace contaminants (such as arsenic) may be accomplished at conventional coagulant dosages that are used for turbidity removal, since the concentrations of these contaminants are usually very low (in the ppb range). However, powdered activated carbon can also be added to the conventional coagulation process to remove trace organic contaminants, such as taste and odor compounds (e.g., geosmine and MIB) and/or pesticides. The addition of PAC will also assist flocculation and sedimentation processes through similar ballasted sedimentation mechanisms.

REFERENCES

1. Edzwald, J. K., Tobiason, J. E., Amato, T., and Maggi, L. J., ‘‘Integrating High-rate DAF Technology into Plant Design,’’ *JAWWA* 91:12:41, 1999.
2. Best, G., Mourato, D., and Singh, M., ‘‘Application of Immersed Ultrafiltration Membranes for Color and TOC Removal,’’ *Proceedings, AWWA Annual Conference,* Chicago, IL, 1999.
3. Stumm, W., ‘‘Chemical Interactions in Particle Separation,’’ in *Chemistry of Wastewater Technology* (A. J. Rubin, ed.), Ann Arbor Science Publishers/Technomic Publishing Co., Inc., Lancaster, PA, 1978.
4. Black, A. P., and Hannah, S. A., ‘‘Electrophoretic Studies of Turbidity Removal by Coagulation with Aluminum Sulfate,’’ *JAWWA,* Vol. 53, No. 4, p. 438, 1961.
5. Packham, R. F., ‘‘Some Studies of the Coagulation of Dispersed Clays with Hydrolyzing Salts,’’ *J. Coll. Science,* Vol. 20, 1965.
6. Rook, J. J., ‘‘Haloforms in Drinking Water,’’ *JAWWA,* Vol. 68, No. 3, p. 168, 1976.
7. Black, A. P., and Willems, D. G., ‘‘Electrophoretic Studies of Coagulation for Removal of Organic Color,’’ *JAWWA,* Vol. 53, No. 5, p. 589, 1961.

8. Cheng, R. C., Liang, S., Wang, H.-C., and Beuhler, M. D., ‘‘Enhanced Coagulation for Arsenic Removal,’’ *JAWWA* 86:9:79,1994.
9. Edwards, M., ‘‘Chemistry of Arsenic Removal during Coagulation and Fe-Mn Oxidation,’’ *JAWWA*, Vol. 86, No. 9, 64–77, 1994.
10. Scott, K. N., Gree, J. F., Do, H. D., and McLean, S. J., ‘‘Arsenic Removal by Coagulation,’’ *JAWWA* 87:4:114, 1995.
11. Dennett, K. E., Amirtharajah, A., Moran, T. F., and Gould, J. P., ‘‘Coagulation: Its Effect on Organic Matter,’’ *JAWWA* 88:4:129, 1996.
12. Hering, J. G., Chen, P-Y., Wilkie, J. A., Elimelech, M., and Liang, S., ‘‘Arsenic Removal by Ferric Chloride,’’ *JAWWA* 88:4:155, April 1996.
13. White, M. C., Thompson, J. D., Harrington, G. W., and Singer, P. C., ‘‘Evaluating Criteria for Enhanced Coagulation Compliance,’’ *JAWWA* 89:5:64, 1997.
14. Stumm, W., and O’Melia, C. R., ‘‘Stoichiometry of Coagulation,’’ *JAWWA*, Vol. 60, No. 5, p. 514, 1968.
15. O’Melia, Charles R., ‘‘Coagulation,’’ in *Water Treatment Plant Design* (Robert L. Sanks, ed.), Ann Arbor Science, Ann Arbor, MI, 1978.
16. LaMer, V. K., and Healy, T. W., ‘‘Adsorption-Flocculation Reactions of Macromolecules at the Solid-Liquid Interface,’’*Rev. Pure App. Chem.* 13:112, 1963.
17. Gouy, G., ‘‘Sur la Constitution de la Charge Electrique a la Surface d’un Electrolyte,’’ *Ann. Phys.* (Paris), Serie 4, 9:457, 1910.
18. Chapman, D. L., ‘‘A Contribution to the Theory of Electrocapillarity,’’ *Phil. Mag.* 25:475, 1913.
19. Stern, O., ‘‘Zur Theorie der Elektrolytischen Doppelschicht,’’ *A. Electrochem*, 30:508, 1924.
20. Benefield, Larry D., Judkins, J. F., and Weand, Barron, L., *Process Chemistry for Water and Wastewater Treatment*, Prentice-Hall, Englewood Cliffs, NJ, 1982.
21. O’Melia, C. R., ‘‘Coagulation and Flocculation,’’ in *Physicochemical Processes for Water Quality Control*, by Walter J. Weber, Jr., Wiley-Interscience, New York, 1972.
22. Ruehrwein, R. A., and Ward, D. W., ‘‘Mechanisms of Clay Aggregation by Polyelectrolytes,’’ *Soil Science* 673:485, 1952.
23. O’Melia, Charles R., ‘‘A Review of the Coagulation Process,’’ Public Works, p. 82, May, 1969.
24. Matsui, Y., Yuasa, A., Furuya, Y., and Kamei, T., ‘‘Dynamic Analysis of Coagulation with Alum and PACL,’’ *JAWWA* 90:10:96, 1998.
25. Hanson, A. T., and Cleasby, J. L., ‘‘The Effects of Temperature on Turbulent Flocculation: Fluid Dynamics and Chemistry,’’ *JAWWA* 82:11:56, 1990.
26. Stumm, W., and Morgan, J. J., ‘‘Chemical Aspects of Coagulation,’’ *JAWWA* 5:971, 1962.
27. Stumm, W., ‘‘Metal Ions in Aqueous Solutions,’’ in *Principles and Applications of Water Chemistry* (S. D. Faust and J. V. Hunter, eds.), John Wiley & Sons, Inc., New York, 1967.
28. Thompson, C. G., Singley, J. E., and Black, A. P., ‘‘Magnesium Carbonate—A Recycled Coagulant,’’ *JAWWA,* Vol. 64, No. 1, p. 11, 1972.
29. Thompson, C. G., Singley, J. E., and Black, A. P., ‘‘Magnesium Carbonate—A Recycled Coagulant,’’ *JAWWA,* Vol. 64, No. 2, p. 93, 1972.
30. Stumm, W., Huper, H., and Champlin, R. L. ‘‘Formation of Polysilicates as Determined by Coagulation Effects,’’ *Environmental Science and Technology*, Vol. 1, No. 3, p. 221, 1967.
31. Cohen, Jesse M., and Hannah, Sidney A., ‘‘Coagulation and Flocculation,’’ in *Physicochemical Processes for Water Quality Control* (W. J. Weber, Jr., ed.), John Wiley & Sons, Inc., New York, 1972.
32. Committee on Coagulation, ‘‘State of the Art of Coagulation,’’ *JAWWA,* Vol. 63, No. 2, p. 99, 1971.

33. Knocke, W. R., Hamon, J. R., and Dulin, B. E., ‘‘Effects of Coagulation on Sludge Thickneing and Dewatering,’’ *JAWWA* 79:6:89, 1987.
34. Camp, T. R., and Stein, P. C., ‘‘Velocity Gradients and Internal Work in Fluid Motion,’’ *J. Boston Soc. Civil Eng.* 30:219–237, 1943.
35. Rushton, J. H., ‘‘Mixing of Liquids in Chemical Processing,’’ *Ind. Eng. Chem.* 44:2931–2936, 1952.
36. Gemmell, R. S., ‘‘Mixing and Sedimentation,’’ in *Water Quality and Treatment,* American Water Works Association, Inc., McGraw-Hill Book Co., New York, 1971.
37. Camp, T. R., ‘‘Floc Volume Concentration,’’ *JAWWA* 60:656–673, 1968.
38. Letterman, R. D., Orron, J. E., and Gemmell, R. S., ‘‘Influence of Rapid-Mix Parameters on Flocculation,’’ *JAWWA* 67:716–722, 1973.
39. TeKippe, R. J., and Ham, R. K., ‘‘Velocity-Gradient Paths in Coagulation,’’ *JAWWA* 63:439–448, July, 1971.
40. Vrale, Lassi, and Jorden, Roger M., ‘‘Rapid Mixing in Water Treatment,’’ *JAWWA* 63:52–58, 1971.
41. Amirtharajah, A., ‘‘Design of Rapid Mix Units,’’ in *Water Treatment Plant Design* (Robert Sanks, ed.), Ann Arbor Science, Ann Arbor, MI, 1978.
42. Hudson, H. E., Jr., ‘‘Dynamics of Mixing and Flocculation,’’ *Proceedings of the 18th Annual Public Water Supply Conference,* University of Illinois, Dept. of Civil Engineering.
43. Kawamura, Susumu, ‘‘Coagulation Considerations,’’ *JAWWA,* Vol. 65, No. 6, pp. 417–423, 1973.
44. Kawamura, S., ‘‘Considerations on Improving Flocculation,’’ *JAWWA,* Vol. 68, pp. 328–336, 1976.
45. Chao, Junn-Ling, and Stone, Brian G., ‘‘Initial Mixing by Jet Injection Blending,’’ *JAWWA* 71:570–573, 1979.
46. Chow, V. T., *Open Channel Hydraulics,* McGraw-Hill Book Co., New York, 1959.
47. Morris, Juli K., and Knocke, William R., ‘‘Temperature Effects on the Use of Metal Ion Coagulants for Water Treatment,’’ presented at AWWA Conference, Las Vegas, NV, 1983.
48. Camp, T. R., ‘‘Flocculation and Flocculation Basins,’’ *Trans. Am. Soc. Civil Eng.* 120:1–16, 1955.
49. Fair, G. M., Geyer, J. C., and Okun, D. A., *Water and Wastewater Engineering,* Vol. 2, John Wiley & Sons, Inc., New York, 1968.
50. Argaman, Y., and Kaufman, W. J., ‘‘Turbulence and Focculation,’’ *J. Environ. Div., ASCE* 96: 223–241, 1970.
51. Argaman, Y. A., ‘‘Pilot-Plant Studies of Flocculation,’’ *JAWWA* 63:775–777, December, 1971.
52. Hudson, H. E., and Wolfner, J. P., ‘‘Design of Mixing and Flocculation Basins,’’ *JAWWA* 59: 1257–1267, 1967.
53. Andreu-Villegas, R., and Letterman, R. D., ‘‘Optimizing Flocculator Power Input,’’ *J. Environ. Eng. Div., ASCE* 102:251–264, 1976.
54. Amirtharajah, A., ‘‘Design of Flocculation Systems,’’ in *Water Treatment Plant Design* (Roberts Sanks, ed.), Butterworth Publishers, Stoneham, MA, 1978.
55. Ives, K. J., ‘‘Theory of Operation of Sludge Blanket Clarifiers,’’ *Proc. Inst. Civil Eng.* 39: 243–260, 1968.
56. Ives, K. J., Bhole, A. G., ‘‘Theory of Flocculation for Continuous Flow Systems,’’ *J. Environ. Eng. Div., ASCE* 99:17–34, 1973.
57. Cockerham, P. W., and Himmelbau, D. M., ‘‘Stochastic Analysis of Orthokinetic Flocculation,’’ *J. Environ. Eng. Div., ASCE* 100:279–294, 1974.
58. Walker, J. D., ‘‘High Energy Flocculation,’’ *JAWWA*, 60:1271–1279, 1968.

59. ‘‘Operational Control of Coagulation and Filtration Processes,’’ AWWA Manual M37, American Water Works Association, 1992.
60. Black, A. P., and Smith, A. L., ‘‘Determination of the Mobility of Colloidal Particles by Microelectrophoresis,’’ *JAWWA* 54:926, 1962.
61. Williams, R. L., ‘‘Microelectrophoretic Studies of Coagulation With Aluminum Sulfate,’’ *JAWWA* 57:801, 1965.
62. Riddick, T. M., ‘‘Zeta Potential and Its Application to Difficult Waters,’’ *JAWWA* 53:1007, August, 1961.
63. Shull, K. E., ‘‘Filtrability Techniques for Improving Water Clarification,’’ *JAWWA* 59:1164, 1967.
64. TeKippe, R. I., and Ham, R. K., ‘‘Coagulation Testing, A Comparison of Techniques,’’ *JAWWA*, Vol. 62, Part I, p. 594, Part II, p. 620, 1970.
65. Kawamura, S., and Tanaka, Y., ‘‘Applying Colloid Titration Techniques to Coagulant Dosage Control,’’ *Water and Sewage Works* 113:348, 1966.
66. Kawamura, S., Sanna, G. P., Jr., and Shumate, K. S., ‘‘Application of Colloid Titration Technique to Flocculation Control,’’ *JAWWA* 59:1003, 1967.
67. Cheng, R. C., Krasner, S. W., Green, J. F., and Wattier, K. L., ‘‘Enhanced Coagulation: a Preliminary Evaluation,’’ *JAWWA* 87:2:91, 1995.
68. Crozes, G., White, P., and Marshall, M., ‘‘Enhanced Coagulation: Its Effect on NOM Removal and Chemical Costs,’’ *JAWWA* 87:1:78, 1995.
69. ASCE, AWWA, CSSE, *Water Treatment Plant Design*, American Water Works Association, New York, 1969.
70. ASCE, AWWA, *Water Treatment Plant Design*, 3rd ed., American Water Works Association, McGraw-Hill, New York, 1997.
71. Culp, G. L., and Culp, R. L., ‘‘Concepts in Water Purification,’’ Van Nostrand Reinhold, New York, 1974.

CHAPTER 11

Sedimentation and Other Clarification Processes

SEDIMENTATION

Since ancient times, sedimentation has been the standard treatment process to clarify water. Since the 1980s, enhancements to the sedimentation process, such as flotation, ballasted sedimentation, adsorption clarifiers, and enhanced solids contact, have improved cost effectiveness and operations efficiency.

Theory of Sedimentation

Sedimentation processes can generally be classified in four categories:

- *Settling of nonflocculent particles*—settling of dilute suspensions of particles that have no (or limited) tendency to flocculate
- *Settling of flocculent particles*—settling of dilute suspensions of flocculent particles
- *Zone settling or hindered settling*—settling that occurs as a large mass rather than as discrete particles once the settling particles come close together
- *Compression*—accumulation of settling particles at the bottom of a settling basin, in which the particles contact each other and are supported by their compacting mass. The structure of the mass restricts further consolidation.

Settling of Nonflocculent Particles The settling of discrete, nonflocculent particles is determined solely by the properties of the fluid and the characteristics of the particles. The key factors are the particle density, the particle size and shape, and the density of the fluid. The terminal settling velocity can be calculated as

$$V_t = \frac{2g(\rho_s - \rho_l)}{C_D \rho_l} \cdot \frac{V_p}{A_p} \qquad (11\text{–}1)$$

where:

V_t = terminal settling velocity, ft/sec (m/s)
g = acceleration of gravity
= 32.2 ft/sec^2 (9.81 m/s^2)

ρ_s = particle density, $\text{lb}_\text{m}/\text{ft}^3$ (kg/m^3)
ρ_l = liquid density, $\text{lb}_\text{m}/\text{ft}^3$ (kg/m^3)
C_D = drag coefficient, dimensionless
V_p = particle volume, ft^3 (m^3)
A_p = projected particle area in the direction of flow, ft^2 (m^2)

Equation 11–1 identifies several properties that affect sedimentation: particle density, liquid density, and the size and shape of the particles. The settling velocity of a particle varies inversely with liquid density and liquid kinematic viscosity. Kinematic viscosity is related to water temperature, as shown in Table 11–1.

Water temperature has an important effect on sedimentation basin design. The settling velocity of a particle is directly related to the ratio of the kinematic viscosity at 50°F (10°C) to the kinematic viscosity at any other temperature. For example, increasing the water temperature from 50°F (10°C) to 86°F (30°C) would increase the settling velocity of a particle by 1.307/0.801, or 1.63, times (see Table 11–1). Similarly, reducing the temperature from 50°F (10°C) to 32°F (0°C) would reduce the settling velocity to 1.307/1.787, or 0.73, times its initial value. This illustrates the fact that, other conditions being equal, with cold waters, sedimentation basin overflow rates should be lower than with warmer waters.

Settling velocity also is a function of the specific gravity and the size of the particles. Table 11–2 presents some relative settling velocities that dramatically illustrate the effect of particle size.

The behavior of nonflocculent particles is often used to describe ideal settling. An ideal settling basin is divided into four zones, as illustrated in Figure 11–1: inlet, outlet, settling, and sludge zones.

Ideal settling theory results in the following equation for surface loading (i.e., surface overflow rate):

$$V_0 = \frac{Q}{A} \tag{11–2}$$

where:

V_0 = settling velocity of particle that settles the depth of the basin in detention time t_0, ft/sec (m/s)

TABLE 11–1. Relationship of Kinematic Viscosity to Water Temperature

Temperature		Kinematic Viscosity, ν	
°F	°C	10^{-5} ft²/sec	10^{-6} m²/s
32	0	1.923	1.787
41	5	1.634	1.519
50	10	1.406	1.307
59	15	1.227	1.140
68	20	1.08	1.004
86	30	0.862	0.801

Source: Adapted from Reference 1.

TABLE 11–2. Velocities at Which Particles of Sand and Silt Subside in Still Water

Diameter of Particle (mm)	Classification	Hydraulic Subsiding Rate (mm/s)	Comparable Overflow Rate gpm/ft^2	Comparable Overflow Rate m/h
10.0	Gravel	1,000	1,480	3,600
1.0		100	150	360
0.6		63	90	230
0.4	Coarse sand	42	60	150
0.2		21	30	75
0.1		8	12	30
0.06		3.8	5.6	14
0.04	Fine sand	2.1	3.1	7.6
0.02		0.62	0.91	2.2
0.01		0.154	0.23	0.55
0.004	Silt	0.0247	0.036	0.087

Source: Reference 2. (Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association.)

Note: Temperature, 50°F (10°C); specific gravity of sand and silt particles, 2.65; values for 10-mm to 0.1-mm particles from Hazen's experiments; values for 0.02-mm to 0.004-mm particles from Wiley's formula; intermediate values interpolated from connecting curve.

Q = rate of flow through the basin, ft^3/sec (m^3/s), or gpd (m^3/d) with appropriate conversion factors
A = surface area of the basin, ft^2 (m^2)

Also:

$$t_0 = \frac{C}{Q} \tag{11–3}$$

where:

t_0 = basin detention time, sec
C = volume of settling zone, ft^3 (m^3)

All particles with a settling velocity greater than V_0 are removed. Particles with settling velocities (V_i) less than V_0 are removed only if they enter the basin within a vertical striking distance $h_i = V_i t_0$ from the sludge zone.

Assuming a uniform distribution of particles in the inlet zone, these particles are removed in the ratio of V_i/V_0. If the term f_i is the fraction of particles with a settling velocity of V_0 or less, the removal efficiency E for the tank is

$$E = (1 - f_i) + \int_0^{f_i} \frac{V_i}{V_0}\, df \tag{11–4}$$

If V_0 is taken to be the terminal settling velocity of the critical particles, Equation 11–2 equates terminal velocity to overflow rate. However, the overflow, Q/A, should

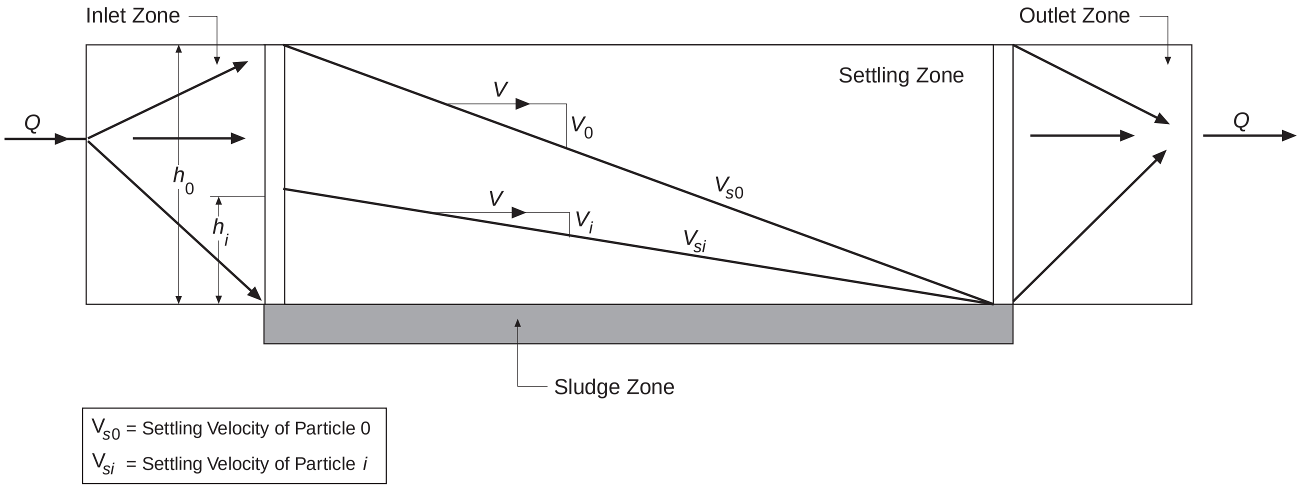


Fig. 11–1. Ideal sedimentation basin (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

preferably be somewhat less than the settling rate of the critical particles. Sedimentation basin loadings are often expressed in units of gallons per day per square foot (meters per day). Thus, under ideal settling conditions, sedimentation is independent of basin depth and detention time, and depends only on the flow rate, basin surface area, and properties of the particle and liquid.

Settling of Flocculent Particles Ideal settling, as discussed in the preceding section, assumes the following:

- All particles settle discretely.
- Particles that strike the bottom remain in the sludge zone.
- Water and particles are distributed uniformly over a vertical plane.
- Incremental volumes of water move from inlet to outlet without changing shape.
- Inlet and outlet conditions do not affect settling.

Sedimentation basins used in water treatment will not perform in accordance with ideal settling theory for the following reasons:

- Discrete particle settling is not obtained. Flocculation of particles occurs during settling. The greater the tank depth, the greater the opportunity for contact among particles.
- Inlet and outlet conditions are not ideal and do affect settling.
- Basin currents cause nonuniform flow and bottom scour.
- Sedimentation basins may include moving mechanical equipment.

Therefore, in the clarification of water, removal is dependent on the basin depth as well as on flow, basin surface area, properties of the particle, inlet and outlet conditions, and the presence of sludge removal equipment.

The performance of a sedimentation basin in settling a suspension of discrete particles can be calculated, but calculating basin performance for a suspension of flocculating particles is much more complex. Empirical analyses, however, may be performed to predict sedimentation basin performance. Settling tests can be used to approximate the design criteria that will be needed to achieve a specified removal of flocculent particles.[3,4,5] However, there are so many factors that affect the performance of full-scale settling basins that data from the operation of other full-scale settling basins are the best source of information for design.

Zone Settling or Hindered Settling When the settling particles come close together, they settle as a mass with a clear interface between the settling particles and the clarified liquid. For a single particle settling in a large vessel, the upward movement of the displaced water is insignificant, but if the concentration of particles is sufficiently great, the upward velocity of the water displaced by the particles becomes sizable in comparison with the settling velocity. Because the settling velocity is relative to the water, the upward flow of displaced water acts to reduce the velocity at which the particles approach the bottom of the basin. This *zone settling,* or *hindered settling,* phenomenon is significant in dealing with large quantities of floc. It is of more import in the handling of wastewater treatment sludges than in many water treatment applications and is often considered to be negligible in water treatment. However, for some specialized water treatment applications where the floc volume is large, as in solids contact units designed to maintain a high floc concentration (discussed later in this chapter) or in basins used to gravity-thicken sludges, zone settling must be considered.

The interface between the sludge and the clarified supernatant can be observed in a batch settling test to evaluate zone settling.[5] Initially, all the suspension is at a uniform concentration, and the height of the interface is h_0, as illustrated in Figure 11–2.

As the figure shows, hindered settling of the particle-liquid interface occurs from A to B at a constant rate. Deceleration occurs in a transition zone from B to C. Consolidation of the sludge blanket in the lower basin region represents the compression zone from C to D. The solids are supported mechanically by those beneath them in the zone from C to D.

The clarification capacity of the system can be estimated from the initial rate at which the interface height decreases. The area required for clarification may be calculated from

$$A = \frac{Q}{V_s} \tag{11–5}$$

where:

A = surface area, ft^2 (m^2)
Q = rate of flow through basins, ft^3/sec (m^3/s)
V_s = subsidence velocity for hindered settling, ft/sec (m/s)

The value of V_s may be computed from the slope of the hindered settling portion of the interface-height-versus-time curve, as illustrated in Figure 11–2.

The thickening capacity of a sludge can be determined by consideration of the batch sedimentation characteristics of a thick suspension.[5] Adequate thickening can be accomplished when the area of the basin is based on the following equation:

$$A = \frac{Qt_u}{h_0} \tag{11–6}$$

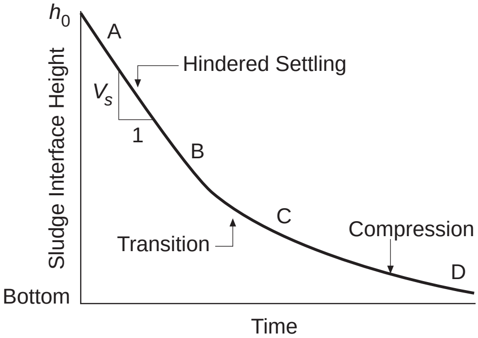


Fig. 11–2. Height of interface in zone settling (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

where:

A = thickening area, ft^2 (m^2)
t_u = time required to reach desired sludge concentration, sec
h_0 = initial height of interface in batch test, ft (m)
Q = flow, ft^3/sec (m^3/s)

Compression Compression occurs when the subsiding particles accumulate at the bottom of the sedimentation basin. The weight of the particles is supported by the compacting mass. Compression (or consolidation of the particles) is a relatively slow process, as illustrated in Figure 11–2.

Applications of Sedimentation

The two principal applications of sedimentation in water treatment are plain sedimentation and sedimentation of coagulated and flocculated waters. Plain sedimentation is used to remove solids that are present in surface waters and that settle without chemical treatment, such as gravel, sand, and silt. It is used as a preliminary process to reduce the sediment loads in the remainder of the treatment plant; as a result, it is referred to as *presedimentation.* Sedimentation is also used downstream of the coagulation and flocculation processes to remove solids that have been rendered more settleable by these processes. Chemical coagulation may be geared toward removal of turbidity, color, or hardness.

Presedimentation The purpose of presedimentation is to reduce the load of sand, silt, turbidity, bacteria, or other substances being applied to subsequent treatment processes, so that those processes may function more efficiently. When the raw water has exceptionally high concentrations of these substances, good removals are often obtained by plain settling and without the use of chemicals. Waters containing gravel, sand, silt, or turbidity in excess of 1,000 NTU may require presedimentation.

Presedimentation basins should have hopper bottoms and/or be equipped with continuous mechanical sludge removal apparatuses especially selected or designed to remove heavy silt or sand. Sludge is not removed continuously, and basin design must allow for sludge accumulation between cleanings. In manually cleaned basins, settled matter is often allowed to accumulate until it tends to impair the quality of the settled water, at which time the sludge is flushed out.

Sedimentation basins that are not equipped for mechanical removal of sludge should have sloping bottoms, so that they can be rapidly drained, allowing most of the sediment to flow out with the water. Because the bulk of the material settles near the inlet end, the slope should be greatest at this point. In some plants, sluice gates are arranged to deliver raw water to the sedimentation basin to flush out the sludge; the balance is generally washed out with a fire hose. At least two basins are needed so one can remain in service while the other is cleaned.

The time between cleanings varies from a few weeks, in plants that have short periods of settling and handle very turbid water, to a year or more, where the basin capacity is large and the water is not very turbid.[6]

Because the particles to be removed in presedimentation basins are more readily settleable than chemical flocs, the detention time may be shorter and the surface overflow and weir rates may be greater than for primary or secondary settling basins. Detention times of at least 3 hr (GL-UMR BSPHEM 1997),[7] maximum overflow rates of 3,500 gpd/ft^2 (5.9 m/h), maximum weir rates of 19,000 gpd/ft (240 m^3/d/m), and minimum water depths of 3 ft (0.9 m) have been recommended as design standards for presedimentation basins in the past (GL-UMR BSSE 1972).[8] When mechanical sludge removal is not provided, detention times of 2 to 3 days are often used to allow for sludge storage. Facilities for chlorination of the presedimentation basin influent are provided in many cases.

Presedimentation in itself can provide reductions in coliform organisms. Reported removals are as follows:[9]

- Ninety percent or more for heavy coliform loadings (20,000 to 60,000 most probable number [MPN] per 100 ml) and long presedimentation periods (5 to 10 days).
- About 20 percent for light coliform loadings (5,000 to 20,000 MPN per 100 ml) and shorter detention times (3 to 7 hr).

Sedimentation Following Coagulation and Flocculation Flocculation is typically followed by sedimentation to reduce the solids loading applied to filters. A majority of the solids can be removed by gravity settling following coagulation and flocculation. When the raw-water turbidity and the chemical dosage are low, it may be appropriate to consider direct filtration (coagulation and flocculation but no sedimentation).

It also may be necessary to provide multiple barriers to the passage of *Giardia* and *Cryptosporidium* through a treatment system. The concept of ‘‘multiple barriers’’ is particularly important where there is no absolute barrier, such as membranes. Multiple barriers are important when each barrier leaks; that is, each barrier removes a high percentage of solids and pathogenic material, but each barrier leaks a small percentage of material that needs to be removed. Sedimentation is a critical component of the multiple barrier concept in conventional water treatment technology.

Horizontal-Flow Sedimentation Basins

Conventional horizontal-flow sedimentation basins may be rectangular or circular. The key design considerations in any conventional basin are:

- Overflow rate
- Inlet and outlet conditions
- Sludge removal
- Basin geometry

Overflow Rate If ideal conditions could be achieved in a sedimentation basin, the overflow rate would equal the settling velocity of the particles to be removed. Unfortunately, ideal settling conditions have not been achieved in practice, and it is necessary to reduce overflow rates to less than theoretical values. Two factors are predominant in the departure of sedimentation basin performance from the ideal: currents and particle interactions.

The most common types of currents are:

- Surface currents, induced by the wind blowing over uncovered basins
- Convection currents, arising from temperature differences

- Density currents, due to the influent water having a different density from the water in the basin (because of the effects of temperature or the suspension load)
- Eddy currents, produced by the incoming water

These currents distort the flow pattern from ideal conditions. Particle interactions cause flocculation that result in the formation of irregular nondiscrete particles.

Because of currents, particle interactions, and other factors, sedimentation basin overflow rates are not based solely on settling velocities. Typical overflow rates in rectangular and circular sedimentation basins with alum as the primary coagulant are 600 to 1,000 gpd/ft^2 (1.0 to 1.7 m/h). The higher rates are used in warm waters, the lower rates in cold waters. Higher overflow rates with alum coagulation may be possible by using polymers to aid flocculation and settling. Ferric chloride and lime coagulants will form dense, fast-settling floc in many waters permitting the use of higher overflow rates. Typical overflow rates for lime floc are 1,400 to 2,100 gpd/ft^2 (2.4 to 3.6 m/h).

Inlet and Outlet Conditions Poor sedimentation basin performance can result from uneven influent flow distribution, inadequate dissipation of inlet energy, nonuniform collection of effluent flow, and the associated hydraulic short-circuiting within a basin.

An ideal inlet would distribute the water uniformly over the full cross section of the tank. The effects of density and inertial currents have been found to be more critical at the inlet than at the outlet.[2]

Because of the relatively fragile nature of chemical floc, the velocity in the influent channels or pipelines to a sedimentation basin must be kept low (0.5 to 1.0 ft/sec [0.15 to 0.3 m/s]). Also, low velocities are needed through any inlet ports to minimize floc breakup.

In rectangular basins, the flow may enter through an inlet channel across the head end of the tank (Fig. 11–3). The water passes through a number of inlet ports across

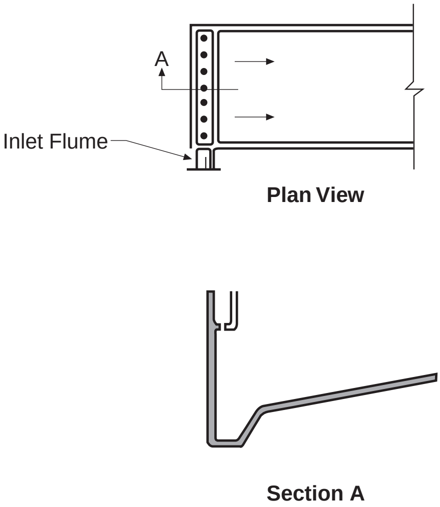
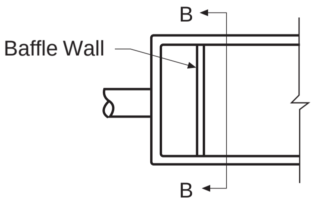


Fig. 11–3. Rectangular basin inlet channel (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

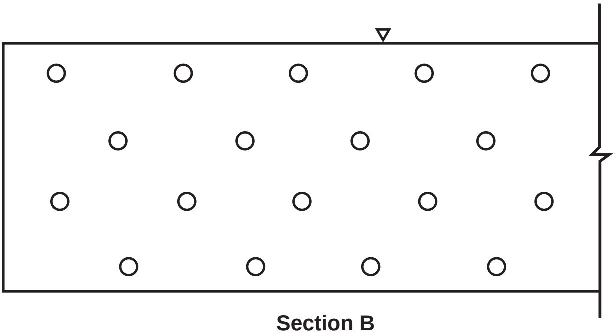
the tank, usually with entrance velocities less than 2 ft/sec (0.6 m/s), distributing the flow across the full width of the tank. A perforated baffle wall located across the tank can provide excellent flow distribution (Fig. 11–4). The best location for such a baffle has been reported to be 6.5 to 8 ft (2 to 2.5 m) downstream of the basin inlet wall.[10] In the design of perforated baffles, Hudson states that four requirements must be met:[11]

- The headloss through the ports should be about four times higher than the kinetic energy of any approaching velocities in order to equalize flow distribution both horizontally and vertically.
- To avoid breaking up floc, the velocity gradient through inlet conduits and ports should be held down to a value close to or a little higher than that in the last compartment of the flocculators.
- The maximum feasible number of ports should be provided in order to minimize the length of the turbulent entry zone produced by the diffusion of the submerged jets from the ports in the perforated-baffle inlet.
- The port configuration should be such as to ensure that the discharge jets will direct the flow toward the basin outlet.

It is desirable for the port diameter to be no more than the thickness of the permeable-baffle wall, so that the hydraulic behavior will cause the jets to emerge in the proper direction. Hudson presents a design procedure for determining the size and spacing of the ports. He reports that it is safe to use a velocity through the perforated baffles of about 0.65 to 1 ft/sec (0.2 to 0.3 m/s).[11] Uniformly distributed 5-in. (0.13-



Plan View



Section B

Fig. 11–4. Perforated inlet baffle wall (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

m) orifices with an opening ratio of 6 to 8 percent and a headloss of 0.08 to 0.12 in. (2 to 3 mm) have been found effective.[10] In cases where extreme wind currents, density flows, or large variations in flow rate occur, an intermediate diffuser wall at the basin midpoint will improve basin efficiency. Unfortunately, such a wall limits the type of sludge collection system that can be used.

In circular, center-feed tanks, the water is introduced into a center influent well by an inlet pipe brought up from underneath the basin or suspended from the clarifier walkway. The velocity in the inlet pipe should not exceed 1 ft/sec (0.3 m/s). Discharging the pipe horizontally into the feed well can introduce undesirable currents, so it is better instead to discharge vertically upward from the pipe into the influent well (Fig. 11–5). The influent well may be perforated to assist in dissipating inlet velocities. One approach for energy dissipation is to use an inlet diffusion well that utilizes ports to develop tangential flow in an outer zone formed by a skirt. This approach uses inlet energy to distribute flow evenly across the tank. The diameter of the outer skirt is a minimum of 25 percent of the tank diameter, which represents 6 percent of the tank area. Larger skirts are used in tanks with flocculator center wells.

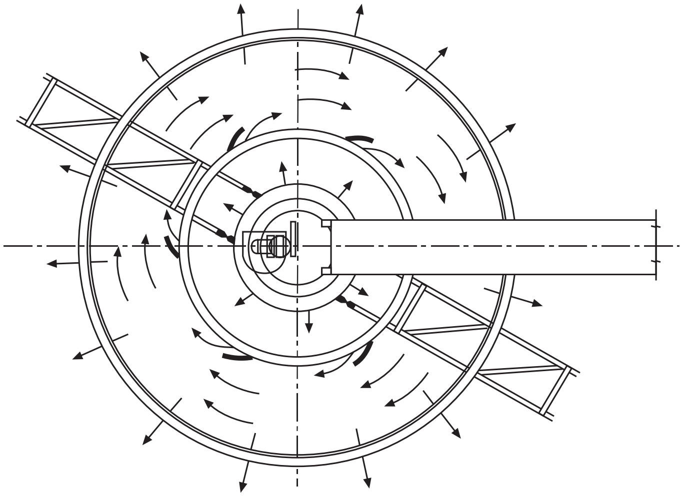
The basin outlet system should collect the water uniformly across the width of the basin to prevent localized high velocities from carrying floc out of the basin. Submerged weirs, or effluent ports, are sometimes used to avoid the breakup of floc that can occur with a freely discharging weir. Perforated launders with ports, commonly submerged 1 to 2 ft (0.3 to 0.6 m) below the surface, are useful in minimizing problems of floating trash passing to the filters. They are also useful when it is desired to vary the water level in the basin during operations that cannot be done with weirs (i.e., greater basin level variation can be achieved). Sometimes a utility finds it helpful to use the storage in the basins to permit some temporary differences between the inflow to the plant and the discharge from the plant.

In northern climates, launders should be placed at such a depth as to avoid problems with icing. Fluctuating levels may also minimize ice attachment to basin walls.

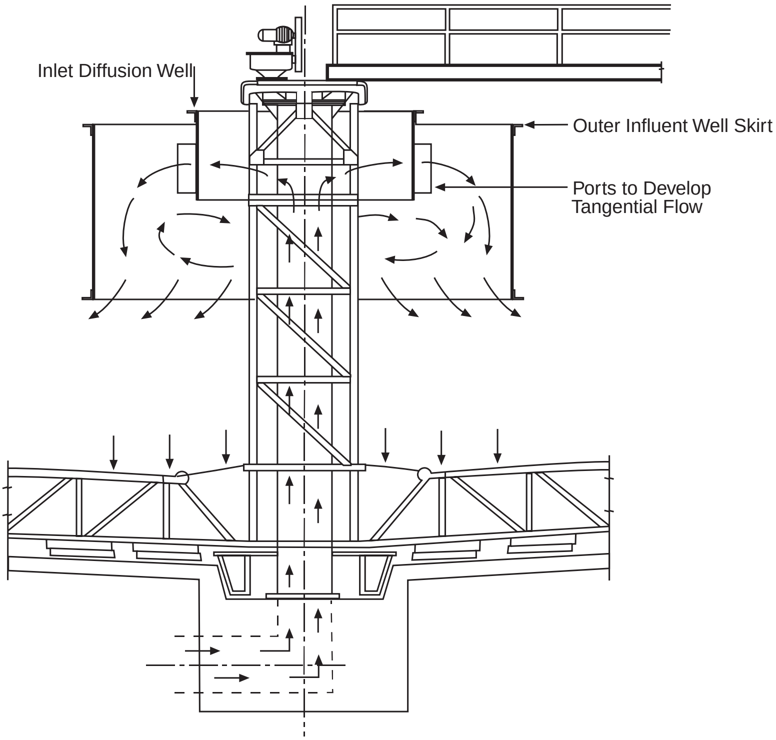
An adequate effluent weir length is necessary to avoid excessive velocity currents. In rectangular basins, adequate weir length usually cannot be obtained with a single weir across the end of the tank, so weirs are provided in the outlet quarter or third of the tank. The weirs or launders may be aligned either parallel or transverse to the direction of flow. In center-feed circular clarifiers, weirs or launders around the periphery of the tank usually provide sufficient weir length. In some cases, an inboard annular trough located about 10 percent of the radius in from the periphery is also used to provide low weir overflow rates for very light flocs. Commonly used weir overflow rates are shown in Table 11–3.

The use of inboard launders to provide added weir length has been reported to adversely affect circular clarifier performance.[12] As shown in Figure 11–6A, typical circular clarifier designs establish a current that moves across the basin floor, up the wall, and into the launders. A proposed approach to overcome the effects of such currents is shown in Figures 11–6B and 11–6C. Plant-scale tests using a 24-in. (0.610-mm) baffle beneath a simple peripheral weir (see Fig. 11–6C) showed improved clarifier performance.[12] It was also found that the larger-diameter inlet well design used in flocculating clarifiers provided better flow distribution and energy dissipation than the inlet wells in standard clarifiers.

Sludge Removal Modern sedimentation basins are equipped with sludge collection and removal mechanisms to eliminate the need to shut them down for cleaning. In rectangular tanks, the bottom is usually sloped gently downward about 5 percent from



Influent Well Plan



Influent Well Section

Fig. 11–5. Circular clarifier, center-fed with vertical inlet discharge

TABLE 11–3. Common Weir Overflow Rates

Type of Floc	Weir Overflow Rate gpm/ft	 $m^3/h/m$
Alum floc, low-turbidity water	8–10	5.9–7.4
Alum floc, high-turbidity water	10–15	7.4–11.2
Lime-softening floc	15–18	11.2–13.4

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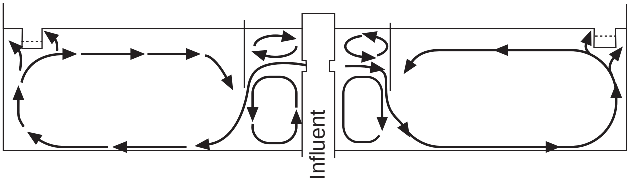
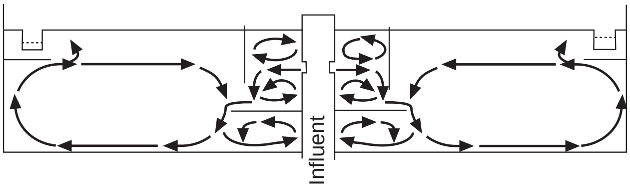
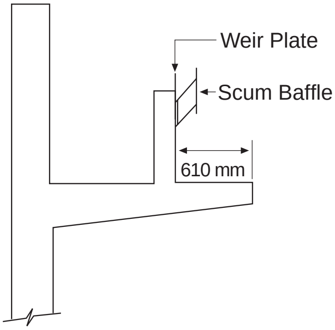


A. Flow Pattern in Conventional Circular Clarifier



B. Flow Pattern in Baffled Circular Clarifier



C. Clarifier Baffle at Weir Trough

Fig. 11–6. Circular clarifier, flow patterns resulting from baffling (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

the effluent end of the tank to a sludge hopper located at the tank inlet. Three types of sludge removal mechanisms are typically used in rectangular basins:

- *Chain and Flight.* These units consist of a series of chains and sprockets, with the sprockets mounted at the top and bottom of the tank (Fig. 11–7). Wood, plastic, or steel cross flights are fitted between the chains. The flights scrape along the basin bottom slowly (less than 1 ft/min [0.3 m/min]) to avoid resuspending the settled sludge dragging the sludge to a hopper. The standard width is 20 ft (6.1 m) for this type of mechanism, although other widths can be custom made.
- *Traveling Bridge.* This type of mechanism is mounted on a carriage that travels along the top of the wall of the tank. The sludge scrapers move slowly (less than 1 ft/min [0.3 m/min]) along the bottom to a hopper or, in some models, sludge is removed by vacuum to an external hopper.
- *Suction Type.* Several models are available from various manufacturers that remove sludge by suction pumping to an external hopper. One example of this type of unit is shown in Figure 11–8.

The bottom slope of circular clarifiers is typically about 8 percent from the outer wall to a central sludge hopper, although steeper slopes up to 15 percent may be used with very high turbidities or heavy sludges, such as lime-softening sludges. Usually, a rotating sludge collector pivots around the center of the tank and scrapes the sludges toward the center hopper (see Fig. 11–9).

Basin Geometry Rectangular basin widths are 20 ft (6.1 m) or less with a single sludge collector and are wider with parallel sludge collectors. Length-to-width ratios of 3:1 to 5:1 are typical and typical depths are 12 to 16 ft (3.6 to 4.9 m). Circular clarifier sludge removal mechanisms are typically available in 1-ft (0.3-m)-diameter increments between 10 and 30 ft (3 and 9 m); in 2-ft (0.6-m)-diameter increments between 30 and 50 ft (9 and 15 m), and in 5-ft (1.5-m)-diameter increments above 50 ft (15 m).

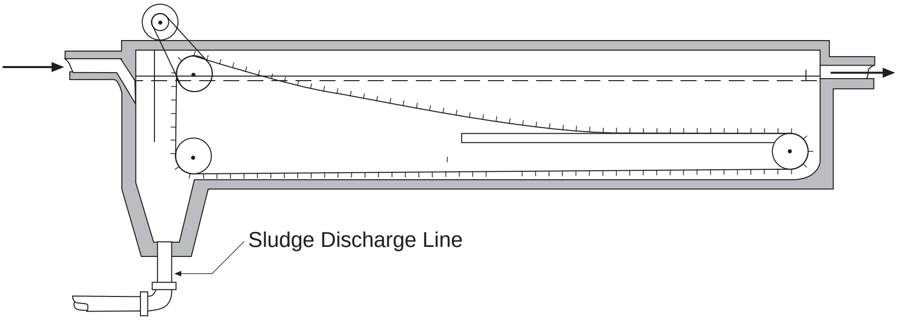


Fig. 11–7. Rectangular sedimentation basin with flight and chain sludge removal system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

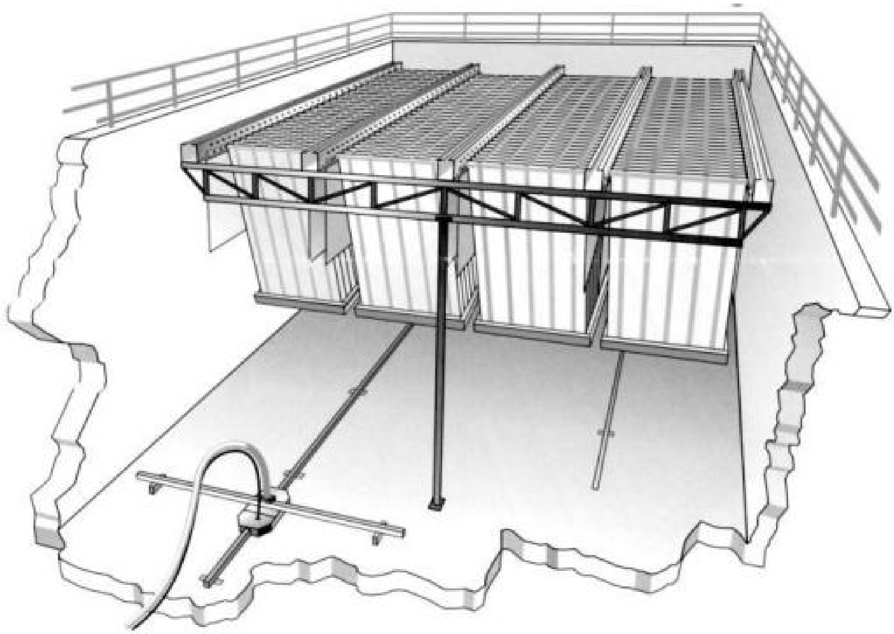


Fig. 11–8. Trac-Vac sludge collectors used with tube-style settlers (Courtesy of EIMCO Process Equipment)

Solids Contact Units

Solids contact units combine coagulation, flocculation, and sedimentation in a single basin. These units typically have two main features. They allow:

- Settled sludge to be recirculated either internally or externally.
- Formation of a sludge blanket.

Some combination flocculation-sedimentation basins do not provide for solids recirculation but have a similar configuration.

Solids contact units are frequently used in softening applications. The chemical and physical reactions of lime–soda softening are enhanced by contact with previously precipitated calcium carbonate. Treatment of surface waters to remove turbidity or organic matter is very site specific, and sludge recirculation may or may not improve performance. Similarly, operation with a sludge blanket may or may not improve performance.

Solids contact reactors are usually sized for a 1 gpm/ft^2 (2.4 m/h) or higher loading rates. When they are used in softening, it is customary to size them for a settling velocity of 1.5 to 2 gpm/ft^2 (3.7 to 4.9 m/h), and they may be operated at even higher rates. There are two types of solids contact units: sludge blanket units and slurry recirculation units.

Figure 11–10 illustrates a typical solids contact unit. In the solids contact mode of operation, previously settled sludge is recycled to the central mixing zone. The slurry

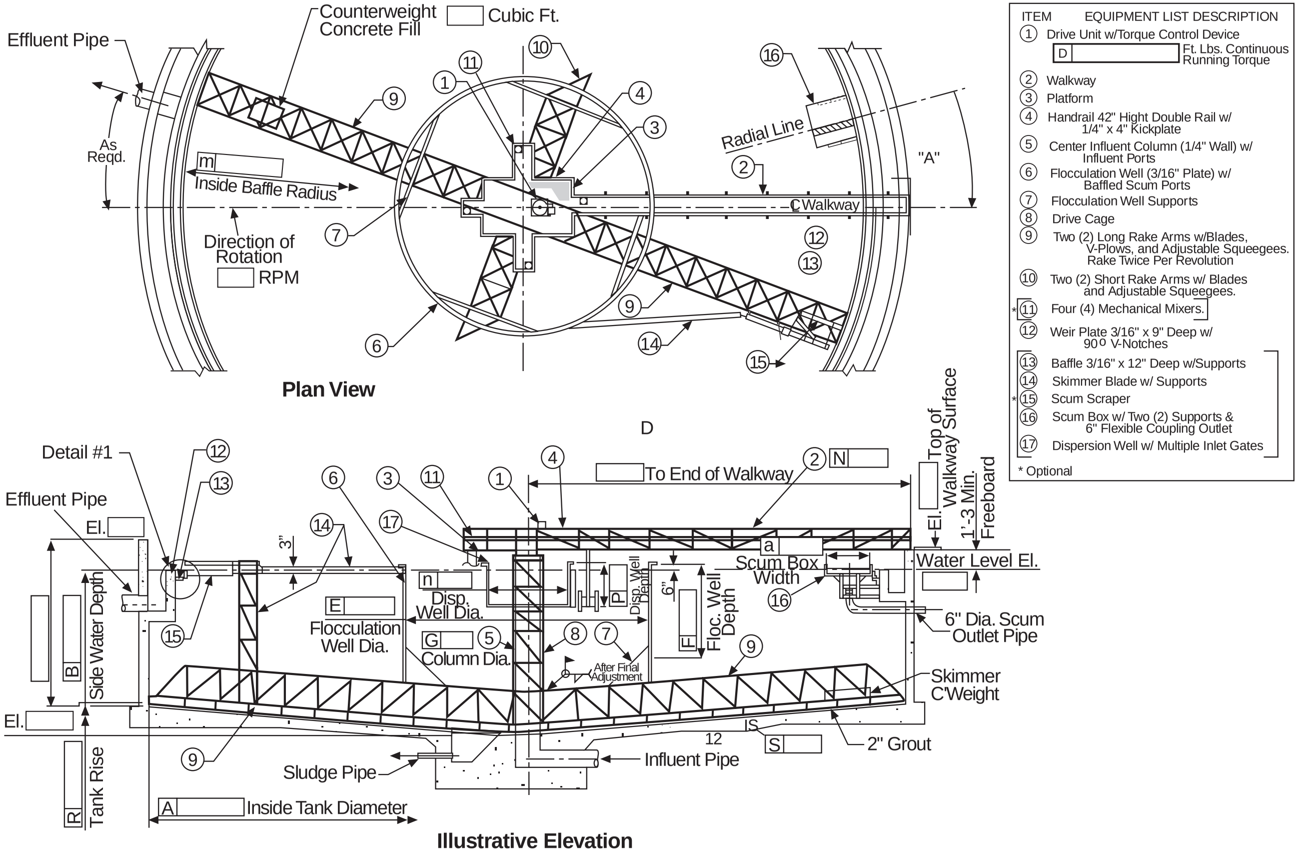


Fig. 11–9. Circular clarifier with sloped bottom to central sludge hopper (Courtesy of WesTech Engineering)

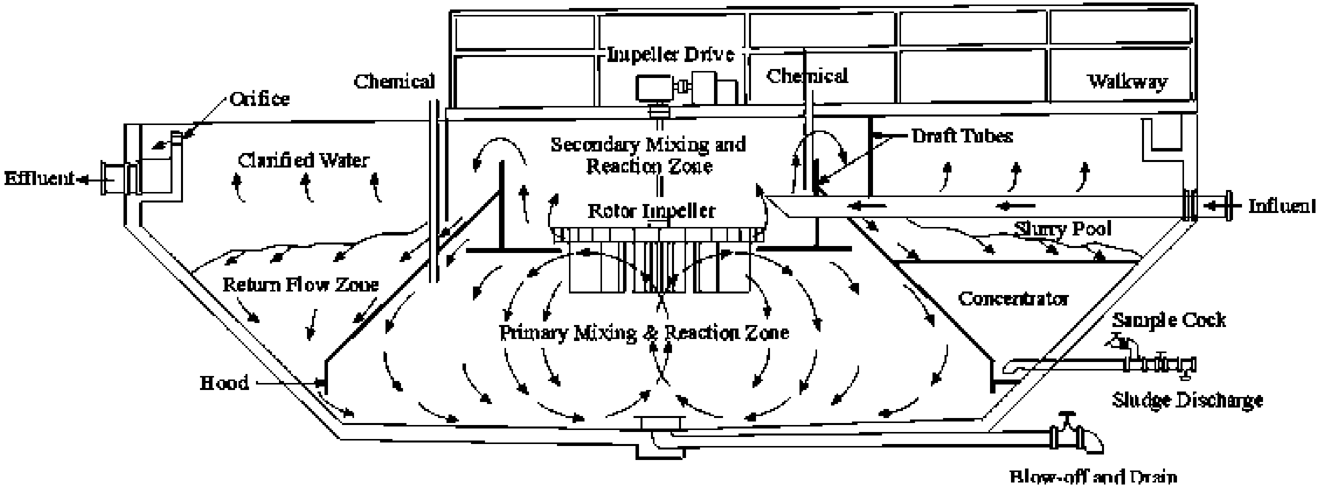


Fig. 11–10. Solids contact unit (Courtesy of Infilco Degremont, Inc.)

concentration is controlled by the amount of sludge drawn from the sludge sumps; it is usually maintained at 6 to 20 percent by volume (higher in softening applications). The recirculation rate is usually maintained at the highest rate consistent with the production of clear settled water. In some instances, too high a rate may cause floc to be discharged into the effluent of the unit. When operating with a sludge, after passing through a mixing zone in the center, the water passes up through a sludge blanket. Sludge removal in sludge blanket units usually occurs by means of a concentrating chamber into which the sludge blanket overflows. Sludge draw-off is regulated by a timer-controlled valve. Sludge blanket operation is sensitive to increases in flow rate and changes in water temperature, which tend to upset the sludge blanket.

Contact Clarification Contact clarification is a form of solids contact clarification that is marketed as a two-stage process in package plants with filtration following the contact clarifier. These units are covered with other package systems in Chapter 25.

Shallow-Depth Sedimentation

Basically, there are two types of shallow-depth sedimentation: plate settlers and tube settlers. There is general agreement that shallow-depth sedimentation offers a theoretically sound basis for operating clarifiers at surface loading rates two to four times higher than in deep, conventional basins, although the surface area may be increased up to tenfold.

Theory The earlier description of the settling paths of discrete particles in an ideal, rectangular basin illustrated in Figure 11–1 is useful in understanding the benefits of shallow-depth sedimentation. The purpose of settling plates or tubes is to reduce the distance a particle must travel before it strikes the bottom.

Allen Hazen pointed out in 1904 that the proportion of sediment removed in a settling basin is primarily a function of the surface area of the basin and is independent of the detention time.[13] He noted that doubling the surface area by inserting one horizontal tray would double the capacity of the basin. He thought that trays spaced at intervals as low as 1 in. (25 mm) would be very desirable if the problems of sludge removal could be resolved.

In 1946, T. R. Camp presented a design for a settling basin that would capitalize on these advantages. It had horizontal trays spaced 6 in. (0.15 m) apart, the minimum distance he thought permissible for mechanical sludge removal. The basin had a detention time of 10.8 min, a velocity of 9.3 ft/min (2.8 m/min), and an overflow rate of 667 gpd/ft^2 (1.13 m/h). Outlet orifices were used to distribute the flow over the width of the trays. In discussing Camp's design, Eliassen noted that although tray tanks had been used for many years in the chemical and metallurgical industries, they had been used in only a few water or sewage treatment systems.[14] Camp believed the lack of early acceptance was due to the reluctance of design engineers to depart from previously accepted practice in the size and shape of basins.[3]

Theoretically, the use of very shallow settling basins enables the detention time of the settling process to be reduced to only a few minutes, in contrast to conventional settling basin designs that use 1- to 4-hr detention. Application of this theory offers tremendous potential for minimizing the size and cost of water treatment facilities. This section describes the techniques that are now available for successful application of shallow-depth sedimentation principles.

Basic Systems Available Two basic types of shallow-depth settling systems available from several manufacturers are illustrated in Figure 11–11: essentially horizontal tube settlers, and steeply inclined tube or plate settlers.

Essentially Horizontal. The operation of essentially horizontal tube settlers is coordinated with that of the downstream filter (see Fig. 11–11). The tubes essentially fill with sludge before any significant amount of floc escapes. Solids leaving the tubes are captured by the filter. Each time the filter backwashes, the settler is completely drained. The tubes are inclined only slightly in the direction of normal flow (5°) to promote the drainage of sludge during the backwash cycle. The rapidly falling water surface scours the sludge deposits from the tubes and carries them to waste. The water drained from the tubes is replaced with the last portion of the filter backwash water so that no additional water is lost in the tube-draining procedure. This tube configuration is ap-

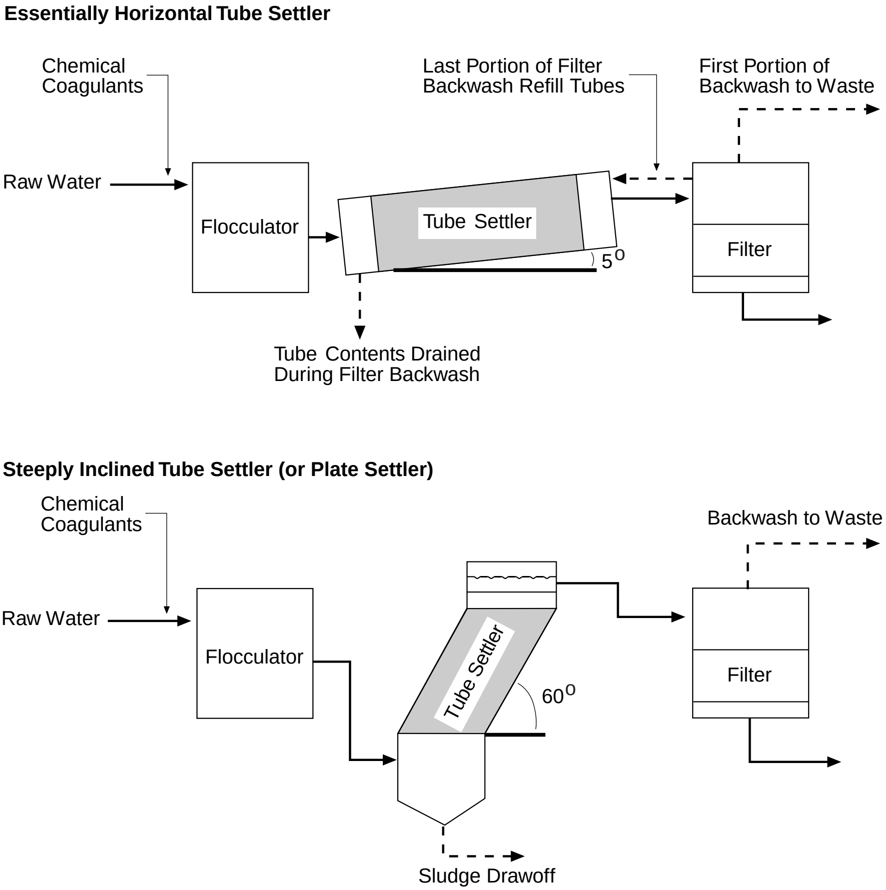


Fig. 11–11. Two shallow-depth settling systems (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

plicable primarily to small plants (1 mgd [3.8 ML/d] or less in capacity) and is often used in package plant systems (described in Chapter 25).

Steeply Inclined. Sediment in tubes or on plates inclined at angles in excess of 45° (normally 45° to 60°) does not accumulate; rather, it moves down the tubes and eventually exits into the plenum below (see Fig. 11–11). A flow pattern is established in which the settling solids are trapped in a downward-flowing stream of concentrated solids, as shown in Figure 11–12. The continuous sludge removal achieved in the steeply inclined tubes or plates eliminates the need for drainage or backflushing of the tubes for sludge removal. The advantage of shallow settling depth coupled with that of continuous sludge removal extends the range of application of this principle to installations with capacities of many millions of gallons per day.

Various manufacturers have developed alternative approaches for incorporating steeply inclined tubes into a modular form that can be built economically and can easily be supported and installed in a sedimentation basin. One type of modular construction is shown in Figures 11–13 and 11–14, in which the material of construction may be polyvinyl chloride (PVC) or alkyl benzene sulfonate (ABS) plastic. Extruded PVC or ABS channels are installed at a 60° inclination between thin sheets of PVC or ABS. Inclining the tube passageways rather than the entire module enables the rectangular module to be readily installed in either rectangular or circular basins. If the direction of inclination of each row of the channels forming the tube passageways alternates, the module becomes a self-supporting beam that needs support only at its ends. The tubular passageways provide approximately 2 to 4 in.^{2} (0.0013 to 0.0026 m^2) of cross section and are available with inclined passageway lengths of 24 in. (0.61 m), 36 in. (0.91 m), and 48 in. (1.2 m).

Other manufacturers use rectangular channels similar to the construction shown in Figure 11–13 but with all of the channels inclined in the same direction.

All of the systems just described are used in configurations in which the influent is introduced beneath the tubes and the flow passes up through the tubes. In the Lamella-type separators, which were initially developed at the Chalmers University of

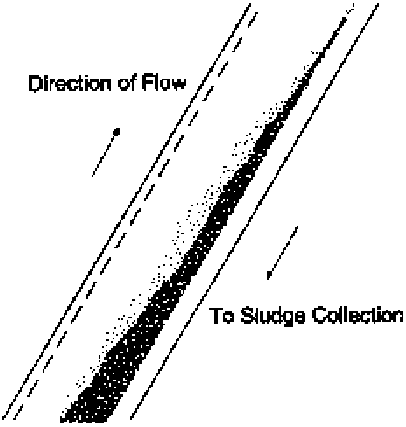


Fig. 11–12. Liquid versus sludge flow pattern in steeply inclined settler tubes (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

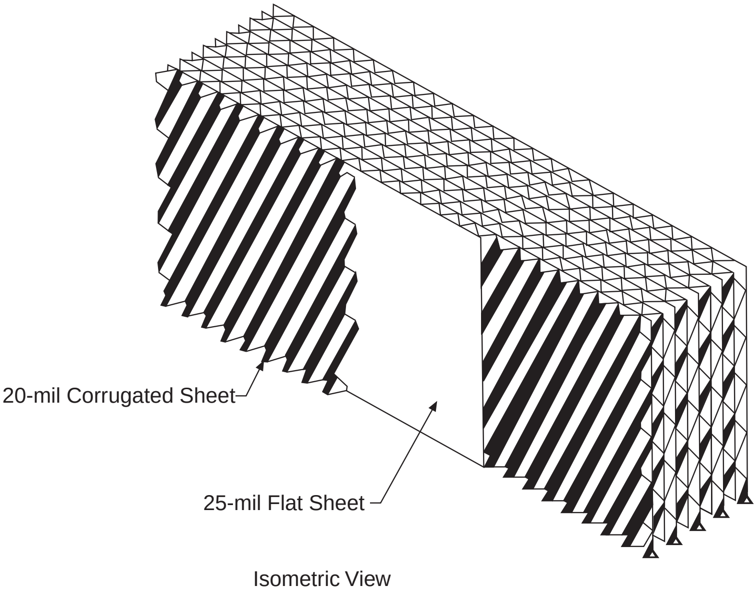


Fig. 11–13. Modular tube construction for steeply inclined settler tubes (Courtesy of Enviropax, Inc.)

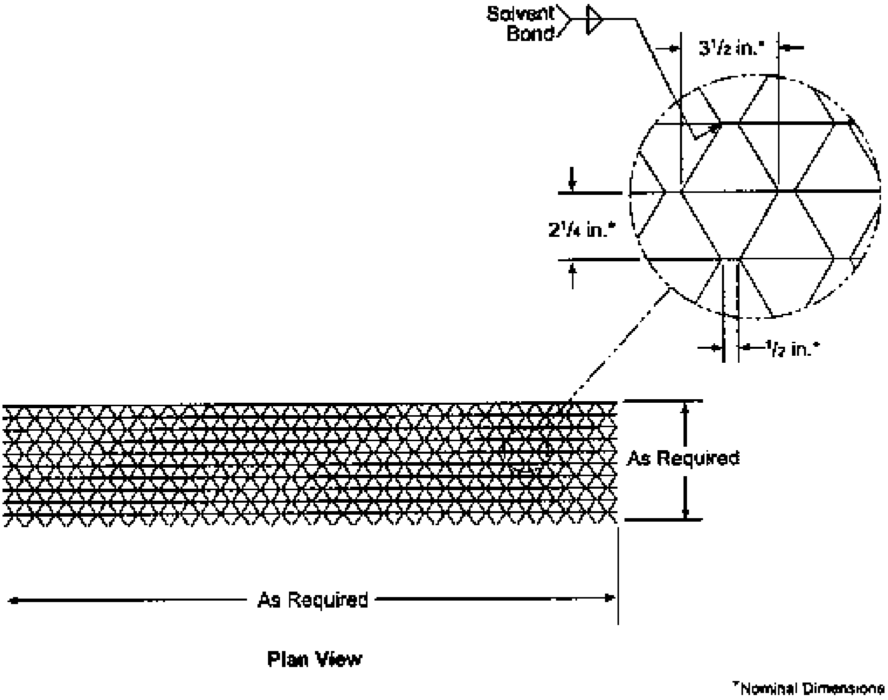


Fig. 11–14. Plan view of modular tube construction (Courtesy of Enviropax, Inc.)

Technology (Sweden) in the mid-1960s and introduced in North America by the Parkson Corporation in 1971, the influent enters approximately one-third from the top of the clarification basin and is directed downward and then upward through a series of parallel plates, as illustrated in Figure 11–15. The sludge is collected at the bottom of the basin, with the sludge flow initially moving in the same direction as the water flow, and then countercurrent as in the other types of systems. The clarified water is conveyed between the plates to the top of the clarifier (see Fig. 11–15). The plates are typically 3 to 12 ft (0.9 to 3.7 m) wide by 7 to 21 ft (2.1 to 6.4 m) long, spaced

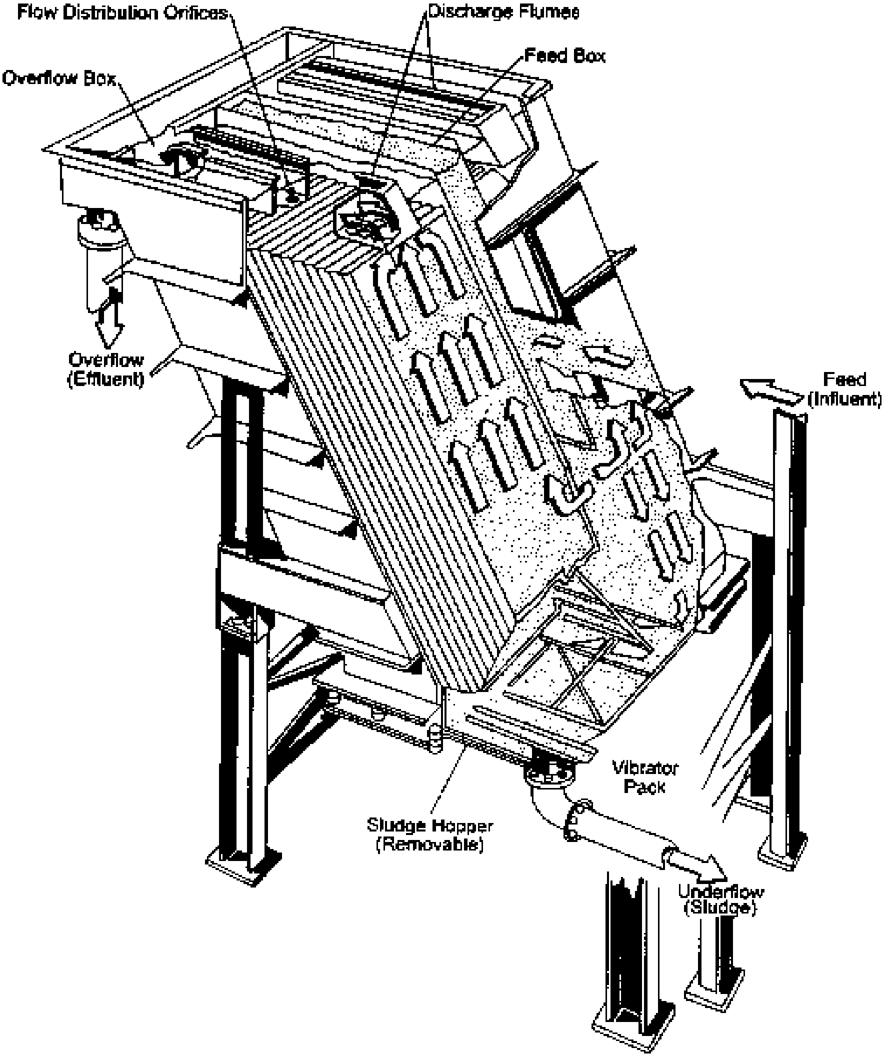


Fig. 11–15. Parallel plates type settler (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

1.5 in. (38 mm) apart, inclined at 55° to the horizontal, and usually constructed of fiberglass-reinforced plastic (FRP).

Although for many years plate settlers were manufactured and sold in the United States and Canada by Parkson Corporation—and Parkson registered the name Lamella—the corporation's licensing agreement expired in 1996. Since that time, Nordic Water Products, the company that licensed the process to Parkson, has been acquired by Waterlink Technologies, Inc. As of 1998, Waterlink Technologies and Purac Engineering, Inc., both manufacture and sell plate settlers under different brand names.

General Design Considerations Steeply inclined tubes can be used in either upflow solids contact clarifiers or horizontal-flow basins to improve performance and/or increase capacity of existing clarifiers. Of course, they can also be incorporated into the design of new facilities to reduce basin size and cost. Capacities of existing basins can usually be increased by 50 to 150 percent with similar or improved effluent quality. The overflow rate at which tubes can be operated is dependent upon the design and type of clarification equipment, the character of the water being treated, and the desired effluent quality. The following sections describe the most important design and operational variables that affect tube installations in existing clarifiers.

Essentially horizontal tubes (7.5° incline from the horizontal) are used in packaged filtration systems discussed in Chapter 25.

Type of Clarifier. The different operational characteristics of horizontal-flow clarifiers and upflow clarifiers necessitate different design considerations.

HORIZONTAL-FLOW CLARIFIERS. The nature of the existing clarification equipment determines to some extent the allowable tube rate and the physical arrangement of modules in a basin. Ideal flow patterns in clarification basins are rarely experienced in practice. Velocities in rectangular horizontal-flow basins vary throughout the basin. Flow lines diverge at the inlet and converge at the outlet. The velocity gradient across the basin does not remain uniform because of basin drag, density currents, inlet turbulence, temperature currents, and so on. In radial-flow circular basins, the flow cannot be introduced to impart velocity components in the horizontal direction only. The use of a center feedwell imparts downward currents that cause turbulence and produce a general rolling motion of the contents in an outward and upward direction.

When tube modules are installed in horizontal-flow basins, it is best not to locate them too near entrance areas, where possible turbulence could reduce the effectiveness of the tubes as clarification devices. For example, in a horizontal-flow basin, often as much as one-third of the basin length at the inlet end may be left uncovered by the tubes so that it may function as a zone for the stilling of hydraulic currents. This is permissible in most basins because the number of tubes needed to achieve a significant increase in sedimentation capacity will require only a portion of the basin. In radial-flow basins, the required number of modules can be placed in a ring around the basin periphery, leaving an inner-ring open area between the modules and the centerwell to dissipate inlet turbulence.

UPFLOW CLARIFIERS WITH SOLIDS CONTACT. The flow paths in upflow solids contact basins are in a vertical direction through a layer or blanket of flocculated material, which is held at a certain level and maintained at a certain concentration by the controlled removal of sludge. The clarification rate is governed by the settling velocity of this blanket. The purpose of maintaining the blanket is to entrap slowly settling, small particles that otherwise would escape the basin. When the flow is increased, the level of the blanket will rise. The efficiency of the tubes is dependent upon both the overflow rate and the concentration of incoming solids. The allowable loading rate on the tubes in this situation is dependent upon the average settling velocity of the blanket, the ability of the clarifier to concentrate solids, and the capacity of the sludge removal system to maintain an equilibrium solids concentration. If sludge is not withdrawn quickly enough or if the upward velocity exceeds the average settling velocity of the blanket, the unit can become solids critical, with the result that the blanket will pass through the tubes—leading to excessive carryover of solids into the effluent.

In expanding the capacity of an upflow solids contact clarifier, the ability to handle increased solids may be the limiting factor. The solids loading of the basin establishes the basin's maximum capacity. The amount of increased capacity is often limited to 50 to 100 percent of the original capacity.

Basin Geometry. The shape of a basin, whether horizontal flow or upflow, determines how the tube modules can be most efficiently arranged to utilize the available space. The best arrangement may be determined strictly by basin geometry once the required area of tube modules is established. Of course, other factors must also be considered. For example, it is desirable to locate the tubes as far as possible from areas of known turbulence or to place them so as to take advantage of an existing effluent launder system.

In circular basins, the tube modules are often placed in pie-shaped segments, as shown in Figure 11–16. This approach is used where the entire clarification area is covered by tube modules or where the tube modules are placed in a ring around the

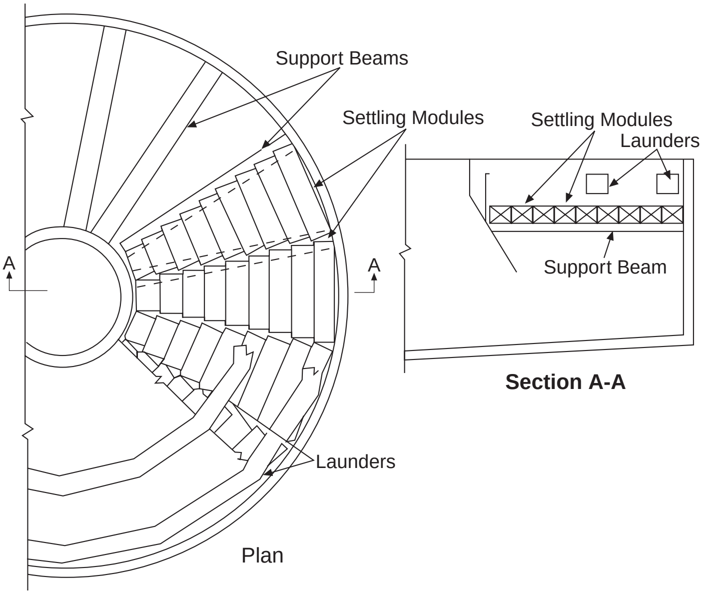


Fig. 11–16. Circular clarifier with tube modules (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

basin outer wall. Where total coverage is required, the modules are supported by radial members that extend from an inner cone or ring to the outer wall of the basin. Where partial coverage is used and where the module ring width does not exceed 10 to 12 ft (3.1 to 3.7 m), the support members may be cantilevered from the exterior walls. In any application where less than the entire area is covered, a baffle wall must be installed at the inner perimeter of the modules to ensure that all flow passes through the modules. The maximum width of a pie-shaped segment at the basin perimeter is limited by the maximum module length, which varies from manufacturer to manufacturer but is usually on the order of 10 to 12 ft (3 to 4 m).

In basins that have radial effluent launders, it is often possible to suspend the modules from the launders, as illustrated in Figure 11–17. In rectangular basins, tubes are simply oriented with the long axis parallel to the sidewalls of the basin, with the support beams spanning the width of the basin, as shown in Figure 11–18.

Tube Support Requirements. The tube support system must be able to support the weight of the tube modules when the basin is drained, as well as to make some allowance for the possibility of a worker standing on the modules and solids adhering to the modules. Most manufacturers recommend a surface loading of 10 lb/ft^2 (48.8 kg/m^2) above the weight of the tube modules. However, experience with the use of alum and activated carbon in treating surface waters indicates that a higher loading should be used for designing the settling tube media support. The bearing surface width of a support member should be more than 1 in. (25 mm) to prevent possible shear failure of the module at the points of contact under extreme loading conditions.

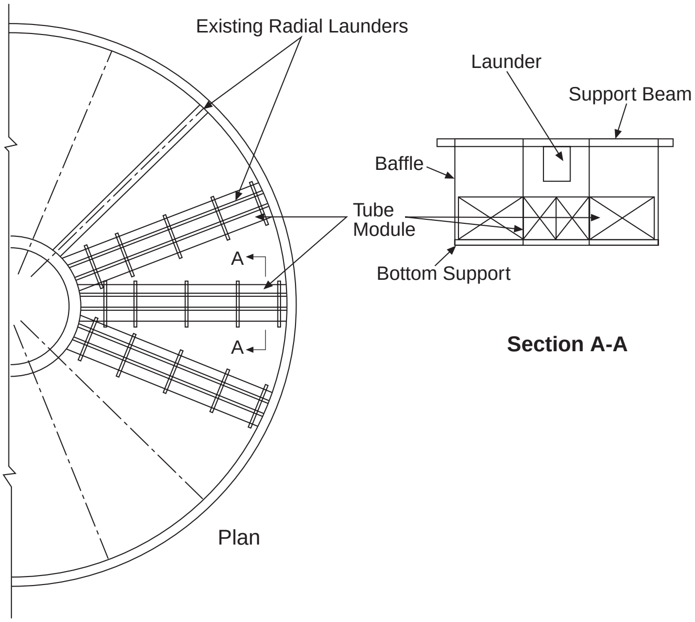


Fig. 11–17. Circular clarifier with tube modules suspended from radial effluent launders (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

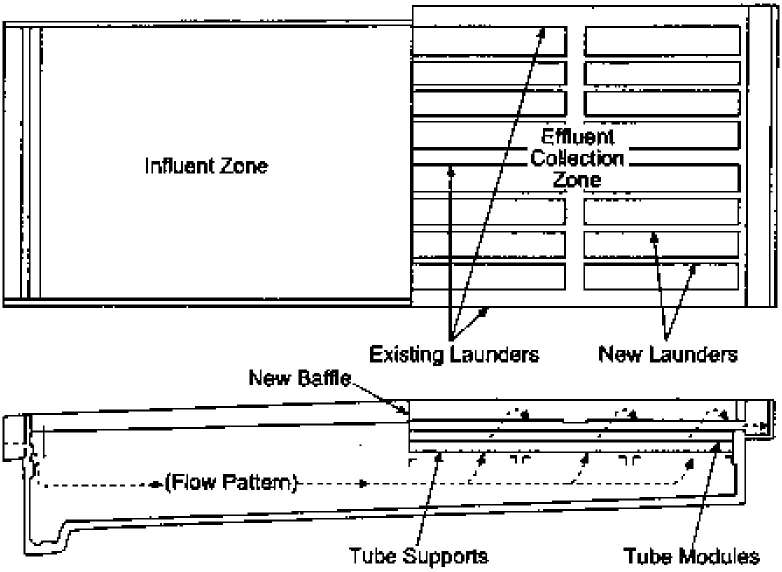


Fig. 11–18. Rectangular sedimentation basin with tube module support beams spanning basin width (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

On the other hand, the width should be as narrow as possible so as to block only a minimum number of tube openings. The support members should be located a minimum of 6 in. (0.15 m)—preferably 1 ft (0.3 m)—in from the module end to enable maximum support of the module.

Flocculation. In expanding a plant's capacity with settling tubes, a utility must closely study the existing flocculation facilities to ensure that the capacity of those facilities will not be overtaxed. The tubes are settling devices—they cannot remove material that has not been flocculated to the point of being settleable.

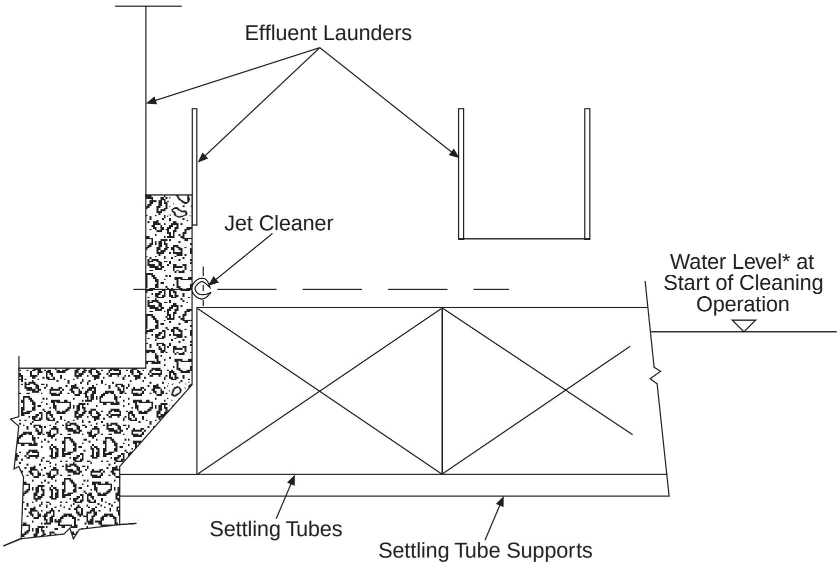
Some plants may have nonmechanical baffle flocculators that generally perform well over a limited flow range. When these facilities are expanded, the flocculation compartment may have to be replaced, supplemented, or modified using mechanical units. The exact flocculation time needed depends in part on the energy input of the mixing device. A rough rule of thumb is that flocculation facilities provide 20 min of detention at water temperatures below 45°F (7.2°C) and 15 min with warmer temperatures. These detention times assume that the basins are designed well enough that they are free of short-circuiting and that a polymer is used as a coagulant aid.

Sludge Removal Facilities. Sometimes when the raw water is of very low turbidity, plants may include settling facilities designed to be manually cleaned. These basins may be cleaned infrequently, perhaps only once a year, if the sediment load is very light. In other cases, they may be cleaned every 30 days. Although it is usually desirable to have continuous mechanical sludge collectors in basins equipped with tubes, tube modules have been successfully used in manually cleaned basins. The frequency of cleaning will obviously need to increase if the basin's throughput is increased following the tube installation.

In general, the only adjustment required with mechanically cleaned water treatment basins will be to increase the frequency of withdrawal of the concentrated sludge from the basin. Most mechanical units have a substantial reserve capacity for handling larger quantities of sludge. In many plants, increasing the frequency of sludge withdrawal will merely involve the adjustment of a timer. In others, it may require complete modification or installation of supplementary sludge concentration facilities.

Tube Cleaning. In certain waters, floc has a tendency to adhere to the upper edges of the tube openings. This is of no serious consequence other than detracting from the appearance of the installation. In some cases, however, the floc buildup eventually bridges the tube openings and results in a blanket of solids on top of the tubes that may reach 3 to 10 in. (76 to 250 mm) in depth unless some remedial action is taken. One method of removing this accumulation is to drop the water level of the basin several inches beneath the top of the tubes occasionally. Care should be taken when dropping the water level so that the accumulated solids are dislodged from the tube settlers as the water level is slowly dropped to the bottom of the tube settlers. The floc particles will then be dislodged and will fall to the bottom of the basin.

If it is not possible to remove the basin from service to drop the level, then directing a very gentle water current across the top of the tubes by a fixed-jet header (as shown in Fig. 11–19) is one method of removing floc accumulation. The header is operated



* If it is possible to lower the basin water level, lower water level only to this point prior to the start of cleaning operation. Slowly lower to below bottom of settling tubes as sludge is cleaned from tubes to prevent over stressing media and/or tube support system from weight of accumulated sludge solids.

Fig. 11–19. Fixed-water-jet header for washing floc accumulation from tube modules infrequently, typically only a few minutes per day. A fixed header can also be used in cases where basins can be removed from service; however, the cost may be greater than for just hosing manually. The general design guidelines for a water wash system are as follows:

- Provide water distribution headers at intervals not exceeding 20 ft (6 m).
- Provide nozzles at 1-ft (0.3-m) centers.
- Provide water supply at 60 psi (410 kPa) and about 6 gpm (0.4 L/s) per nozzle.
- Provide valving so that the cleaning system can be operated in sections in order to reduce the water supply required.

Another cleaning technique involves the installation of a grid of diffused air headers beneath the tubes (Fig. 11–20). General design guidelines for an air wash system are as follows:

- Provide air at a pressure of 6 psi (41 kPa) and a rate of 1 ft^3/min per square foot (0.3 m^3/min per square meter) of tube surface area.
- Provide air distribution laterals 1 ft (0.3 m) below the bottom of the tubes and space at 1-ft (0.3-m) centers.
- Provide ¹⁄₁₆ in. (1.6-mm) diameter orifices at 1-ft (0.3-m) centers, 30° from bottom centerline of lateral and staggered, as shown in Figure 11–20.
- Provide valving so that the system can be operated in sections in order to reduce the air supply required.

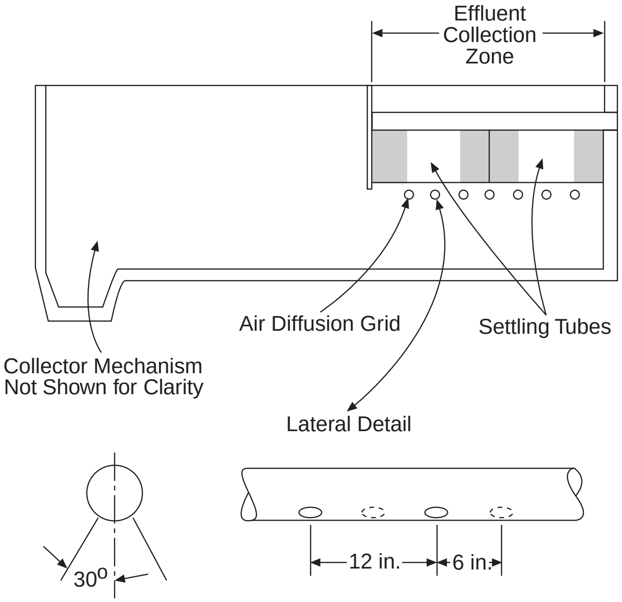


Fig. 11–20. Diffused air header for removing floc accumulation from tube modules (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

When this system needs to be used, the influent is stopped and the air is turned on and allowed to rise through the tubes, scrubbing away any attached floc. A quiescent period of 15 to 25 min follows before the basin is placed back in service.

Loading Rates. For upflow clarifiers in areas where cold water temperatures (less than 40°F [4.4°C]) occur frequently, the guidelines in Table 11–4 apply.

In warm water areas where temperatures are nearly always above 50°F (10°C), the guidelines in Table 11–5 apply.

Of course, these guidelines are based on the assumption that both the chemical coagulation and flocculation steps have been carried out properly and that the sludge-removal equipment is adequate.

For horizontal-flow basins, the raw-water turbidity has a direct influence on allowable tube overflow rates, as does the raw-water temperature. In cold water areas where temperatures are frequently 40°F (4.4°C) or less, the guidelines in Table 11–6 apply.

In warm water areas where temperatures are nearly always above 50°F (10°C), the guidelines in Table 11–7 apply.

Effluent turbidities above 5 NTU will often obscure the tube modules through 2 ft (0.6 m) of water. This may not be aesthetically desirable to a casual observer, but such turbidities are readily treated by a mixed-media filter. If the tube clarification application is not followed by mixed-media filters, the loading rates should be selected to provide effluent turbidities in the range of 1 to 3 NTU.

Location of Tube Modules within the Basin. The tubes should be located such that they are not placed in a zone of unstable hydraulic conditions. Thus, they are frequently placed over the one-half to three-fourths of the basin located nearest the effluent launders to permit the inlet portion of the clarifier to dampen out hydraulic currents. The top of the tubes should be located 2 to 4 ft (0.6 to 1.2 m) below the water surface. In general, the 2-ft (0.6-m) minimum is used in shallow basins, and a submergence of 4 ft (1.2 m) would be considered only in clarifiers with a sidewater depth of 16 to 20 ft (4.9 to 6.1 m). In most basins, where sidewater depths rarely exceed 13 ft (4.0 m), a submergence of 2 to 3 ft (0.6 to 0.9 m) is used. The collection launders should be placed on 10- to 12-ft (3- to 4-m) centers over the entire area, and covered by tubes to ensure uniform flow distribution.

TABLE 11–4. Upflow Clarifier Loading Rates for Water Temperatures Less Than 40°F (4.4°C)

Overflow Rate Based on Total Clarifier Area without Settling Tubes		Overflow Rate for Portion of Basin Covered by Settling Tubes		Probable Effluent Turbidity (NTU)
gpm/ft^2	m/h	gpm/ft^2	m/h	
1.5	3.7	2.0	4.9	1–3
1.5	3.7	3.0	7.3	1–5
1.5	3.7	4.0	9.8	3–7
2.0	4.9	2.0	4.9	1–5
2.0	4.9	3.0	4.9	3–7

Source: Based on information from References 15 and 16.

TABLE 11–5. Upflow Clarifier Loading Rates for Water Temperatures Greater Than 50°F (10°C)

Overflow Rate Based on Total Clarifier Area without Settling Tubes		Overflow Rate Portion of Basin Covered by Settling Tubes		Probable Effluent Turbidity (NTU)
gpm/ft²	m/h	gpm/ft²	m/h	
2.0	4.9	2.0	4.9	1–3
2.0	4.9	3.0	7.3	1–5
2.0	4.9	4.0	9.8	3–7
2.5	6.1	2.5	6.1	3–7
2.5	6.1	3.0	7.3	5–10

Source: Based on information from References 15 and 16.

Example of Applications

Horizontal Basins. Suppose an existing water treatment plant with a rated capacity of 4 mgd (15 ML/d) has a horizontal, rectangular settling basin and rapid sand filters. The basin dimensions are 30 ft wide by 133 ft long (9.1 m by 40.5 m). The surface overflow rate at design capacity is 1,000 gpd/ft^2 (1.7 m/h). The average depth of the basin is 15 ft (4.6 m). The basin has a single overflow weir across its outlet end. The raw water is obtained from a river that has a normal maximum turbidity of 25 to 30 NTU. The water temperature rarely falls below 50°F (10°C). The settling basin is preceded by mechanical flocculation with 40 min of detention time at 4 mgd (15 ML/d). Coagulant aids are fed during periods of high turbidity and low water temperature to improve coagulation.

A capacity increase from 4 to 8 mgd (15 to 30 ML/d) is desired. At 8 mgd, the overflow rate increases to 2,000 gpd/ft^2, or 1.4 gpm/ft^2 (3.4 m/h). The total basin loading of 1.4 gpm/ft^2 is below any of the values shown in Table 11–7's guidelines. However, at a higher basin loading of 2 gpm/ft^2 (4.9 m/h) and a tube rate of 3 gpm/

TABLE 11–6. Loading Rates for Horizontal-Flow Basins Where Water Temperatures Are Frequently 40°F (4.4°C) or Less

Overflow Rate Based on Total Clarifier Area without Settling Tubes		Overflow Rate Portion of Basin Covered by Settling Tubes		Probable Effluent Turbidity (NTU)
gpm/ft²	m/h	gpm/ft²	m/h	
Raw-Water Turbidity = 0–100 NTU				
2.0	4.9	2.5	6.1	1–5
2.0	4.9	3.0	7.3	3–7
3.0	7.3	4.0	9.8	5–10
Raw-Water Turbidity = 100–1,000 NTU				
2.0	4.9	2.5	6.1	3–7
2.0	4.9	3.0	7.3	5–10

Source: Based on information from References 15 and 16.

TABLE 11–7 Loading Rates for Horizontal-Flow Basins Where Water Temperatures Are Frequently 50°F (10°C) or Greater

Overflow Rate Based on Total Clarifier Area without Settling Tubes		Overflow Rate Portion of Basin Covered by Settling Tubes		
gpm/ft²	m/h	gpm/ft²	m/h	Probable Effluent Turbidity (NTU)
		Raw-Water Turbidity = 0–100 NTU		
2.0	4.9	2.5	6.1	1–3
2.0	4.9	3.0	7.3	1–5
2.0	4.9	4.0	9.8	3–7
3.0	7.3	3.5	8.5	1–5
3.0	7.3	4.0	9.8	3–7
		Raw-Water Turbidity = 100–1,000 NTU		
2.0	4.9	2.5	6.1	1–5
2.0	4.9	3.0	7.3	3–7

Source: Based on information from References 15 and 16.

ft² (7.3 m/h), the expected effluent turbidity is 1 to 5 NTU. This turbidity value is compatible with mixed-media filters, which, as discussed elsewhere in this text, can readily replace the existing sand filters. In light of the moderate raw-water temperature and turbidity, a tube rate of 3 gpm/ft² and a basin loading rate of 1.4 gpm/ft² should give excellent results.

$$\begin{aligned}\text{total tube area required} &= \frac{\text{capacity, gpm}}{\text{allowable tube rate, gpm/ft}^2}\\ &= \frac{8\text{ mgd} \times (700\text{ gpm/mgd})}{3\text{ gpm/ft}^2}\\ &= 1{,}870\text{ ft}^2\ (173.7\text{ m}^2) \qquad (11\text{–}7)\end{aligned}$$

The dimensions of the area to be covered by the tube modules are determined as follows:

$$\begin{aligned}\text{area} &= \text{length} \times \text{width}\\ 1{,}870\text{ ft}^2 &= \text{length} \times 30\text{ ft}\\ \text{length} &= \frac{1{,}870\text{ ft}^2}{30\text{ ft}} = 62\text{ ft (19 m)}\end{aligned}$$

The length of 62 ft (19 m) would be rounded off to a length readily compatible with the standard module dimensions associated with the specific modules purchased.

The modules would be installed over an area extending back from the discharge end of the basin for a distance of 62 ft (19 m). A baffle wall would be installed at the inner edge to force all flow through the modules. To improve uniform flow through the modules, three new effluent launders extending 62 ft (19 m) back from the existing end wall launder would be required. The launders would be installed on 10-ft (3.1-m) centers, and the tubes would be submerged for a depth of 4 ft (1.2 m) because the basin is deep. The appearance of the basin would be similar to that shown for an upflow basin in Figure 11–21.

Upflow Basins. Assume a plant has two square upflow clarifiers (42 ft [13 m] to a side), each designed for a flow of 3,000 gpm (16 ML/d), with peripheral collection launders. The total surface area is 1,760 ft^2 (164 m^2). The influent centerwell reduces the available settling area by 200 ft^2 (19 m^2). The peak overflow rate currently reaches 1.92 gpm/ft^2 (4.68 m/h), which is high enough that the clarifier does not perform well, especially when water temperatures drop.

It is desired to increase the plant capacity to 4,000 gpm (22 ML/d) per settling basin. At this flow, the loading on the total basin settling area is 2.6 gpm/ft^2 (6.3 m/h). The raw-water turbidity is moderate, 30 to 70 NTU, and the water temperature seldom falls below 50°F (10°C).

Table 11–5's guidelines for upflow basins indicate that a maximum total basin loading of 2.5 gpm/ft^2 (6.1 m/h) with a corresponding tube rate of 2.5 to 3 gpm/ft^2 (6.1 to 7.3 m/h) can be used. Complete coverage of the settling area would provide a tube rate of 2.6 gpm/ft^2 (6.3 m/h) and would also provide a simplified support problem when compared to only partial coverage. Thus, coverage of the settling area

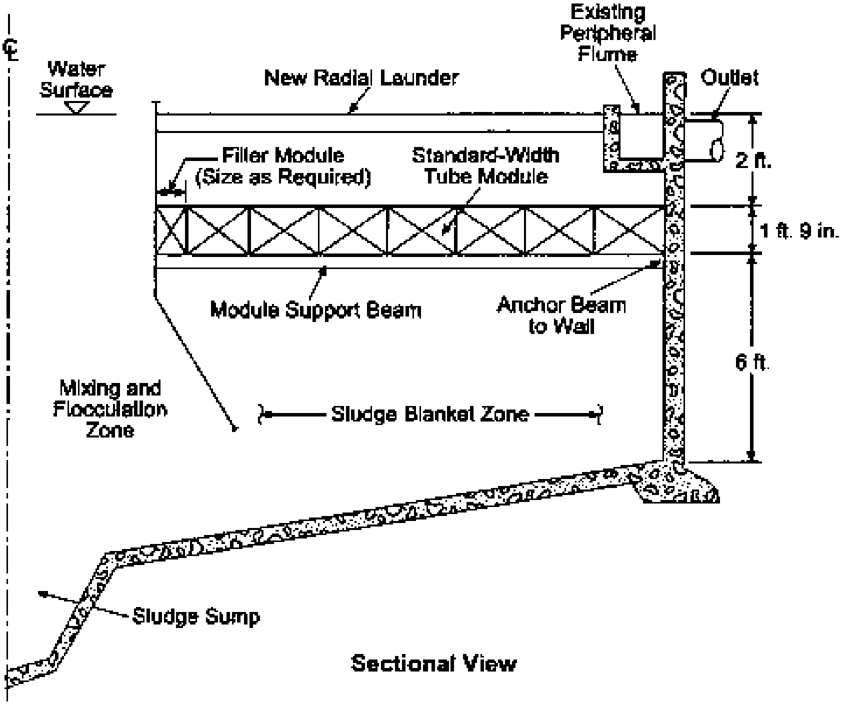


Fig. 11–21. Upflow basin with total coverage of surface area with tube modules (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

with 1,560 ft^2 (145 m^2) of inclined tubes would be provided. Radial launders would be added to improve the flow distribution in the basin, as shown in Figure 11–21. Because the sidewater depth is only about 10 ft (3.1 m), the modules would be submerged 2 ft (0.6 m).

HIGH-RATE CLARIFICATION

High-rate clarifiers are solids contact type units that incorporate settling plates or settling tubes in the sedimentation zone. This technology has been developed by several manufacturers and has been used in several potable water plants in the United States and in many plants in Europe.

Infilco Degremont, Inc. (IDI), manufactures two high-rate clarification systems: the Superpulsator Clarifier and the DensaDeg clarifier and thickener. Kruger manufacturers one high-rate system called the Actiflo Process.

Superpulsator Clarifier

The Superpulsator can operate at loading rates up to 7,000 gpd/ft^2 (16.8 m/h). This is achieved by combining the efficiency of the two processes. This system, shown in Figures 11–22 and 11–23, is a modified sludge blanket clarifier with modified Lamella plates to enhance floc formation and capture.

The water entering the clarifier is ‘‘pulsed’’ through the application of a vacuum in an upstream vacuum chamber. The vacuum causes the water to rise in the vacuum chamber, and the vacuum is released after it reaches a predetermined level. The water ‘‘surges’’ into the clarifier through the distribution conduit and laterals and causes the

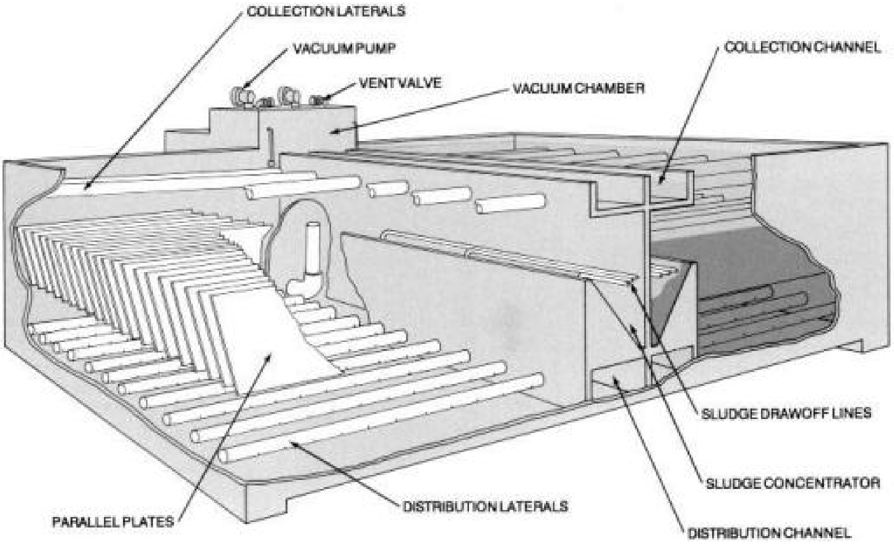
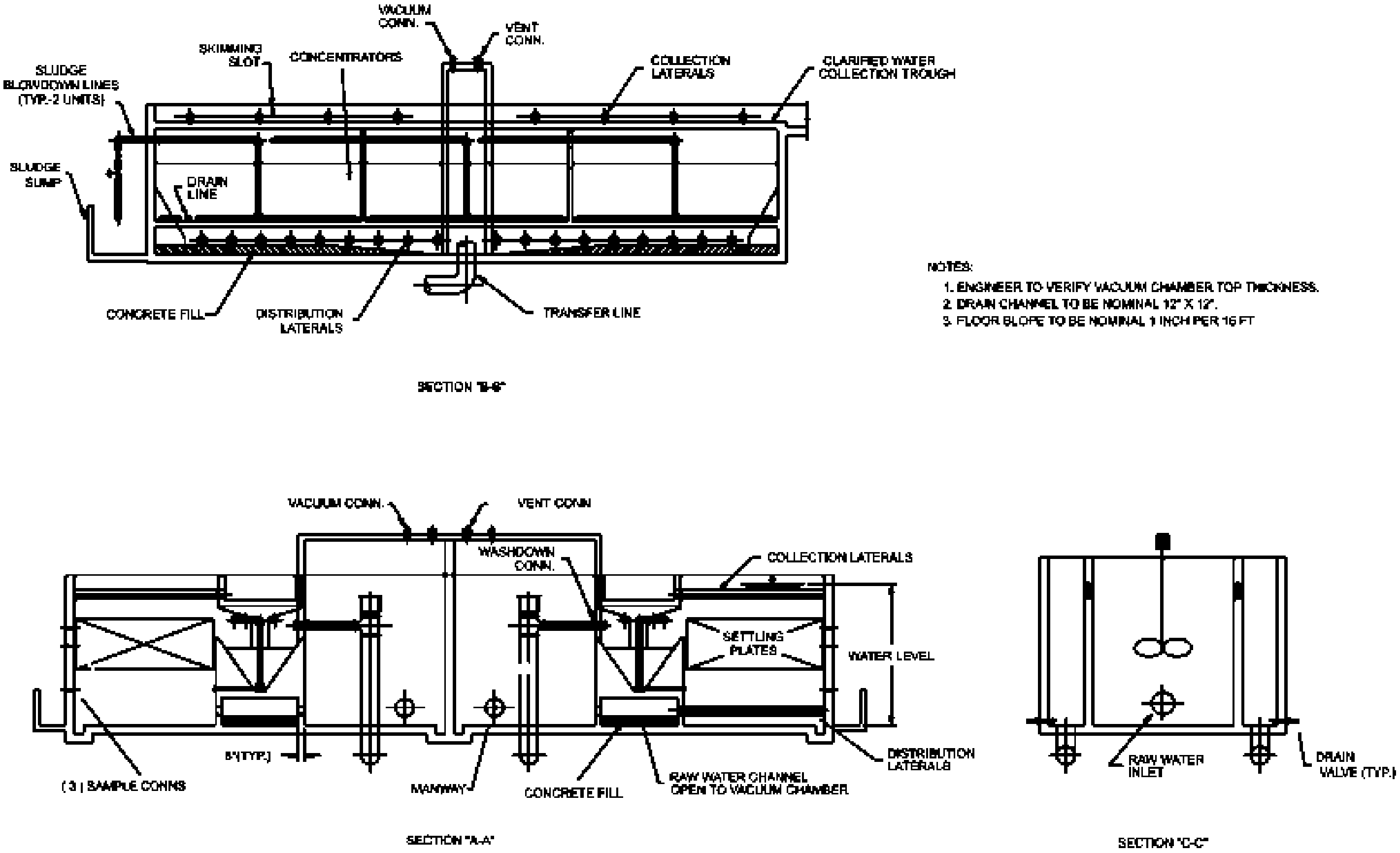


Fig. 11–22. An exploded view of the superpulsator clarifier illustrating typical components and configuration (Courtesy of Infilco Degremont, Inc.)



NOTES:
1. ENGINEER TO VERIFY VACUUM CHAMBER TOP THICKNESS.
2. DRAIN CHANNEL TO BE NOMINAL 12" X 12".
3. FLOOR SLOPE TO BE NOMINAL 1 INCH PER 16 FT

Fig. 11–23. Superpulsator clarifier layout and section views (Courtesy of Infilco Degremont, Inc.)

sludge blanket to expand uniformly. The chemically treated water passes through the sludge blanket, causing particle contact and the formation of larger flocs. The sludge blanket is retained within the clarifier to the level of an overflow weir. When the blanket is expanded with each surge, a portion of the solids flow over the weir into the sludge concentrator zone. The waste solids are allowed to thicken in the concentrator. Concentrated waste solids are removed using a vacuum lift system through multiple pipes.

The supernatant flows up through the Lamella-type plates, which have been modified to include deflectors. The surges through the plates cause small eddy currents at each deflector, which redirect water into the flocculated sludge. The intent is to form a series of miniature clarifiers, with the lower surface acting as an interceptor, on which the settling solids agglomerate and slide toward the bottom. The eddies cause some degree of slurry recirculation, thereby enhancing the coagulation effect.

Superpulsators are in service at over 70 locations in the United States, including the Delaware River Plant and the Intercoastal Waterway Plant.

- The Delaware River Plant treatment train includes preozonation, Superpulsator units designed for a loading of 5,700 gpd/ft^2 (9.8 m/h) with ferric chloride and an anionic polymer as the coagulants. Average influent turbidities are 6–12 NTU and the Superpulsator effluent less than 2 NTU.
- The Intercoastal Waterway Plant has capacity of 30 mgd, with Superpulsators designed for a loading of 4,300 gpd/ft^2 (7.3 m/h). The water is difficult to treat with low alkalinity and high color (average raw-water color, >100 units). Influent turbidity varies with flow from about 5 NTU to 25 NTU. Clarifier effluent turbidity from July 1996 to June 1997 was always below 2 NTU.

DensaDeg Clarifier

The DensaDeg clarifier was initially developed to produce dense sludges. It is now a process that combines internal and external solids recirculation, sludge thickening, and lamellar clarification in three connected compartments for rapid mixing, and flocculation and settling with tubes in the sedimentation compartment. A DensaDeg shown in Figure 11–24 can operate at loadings up to about 14,000 gpd/ft^2 (24 m/h) in the settling area. A 1966 patent (No. 3,247,105) describes the testing that was completed to obtain a denser sludge. The test results are shown here in Table 11–8.

The denser sludge was achieved through recirculation of already flocculated waste solids. Therefore, the design of the DensaDeg clarifier incorporates internal recycling in the Reactor Zone and external recycling from the clarifier to the inlet to the Reactor Zone. The Reactor operates as a flocculator, but the process is enhanced through sludge recirculation, producing an optimum density slurry.

The slurry passes over a submerged weir into the presettling zone of the clarifier. The mixture is forced downward due to a baffle arrangement, and the slurry solids separate from the water and settle to the thickening zone. A sludge collection mechanism slowly rotates to enhance thickening and convey the settled sludge to the hopper for removal.

The separated water flows upward from the baffle through Lamellar tubes. These tubes provide the removal of the remaining solids.

The DensaDeg clarifier is particularly suited to waters in which large volumes of sludge are produced. These include lime softening, chemical precipitation of phospho-

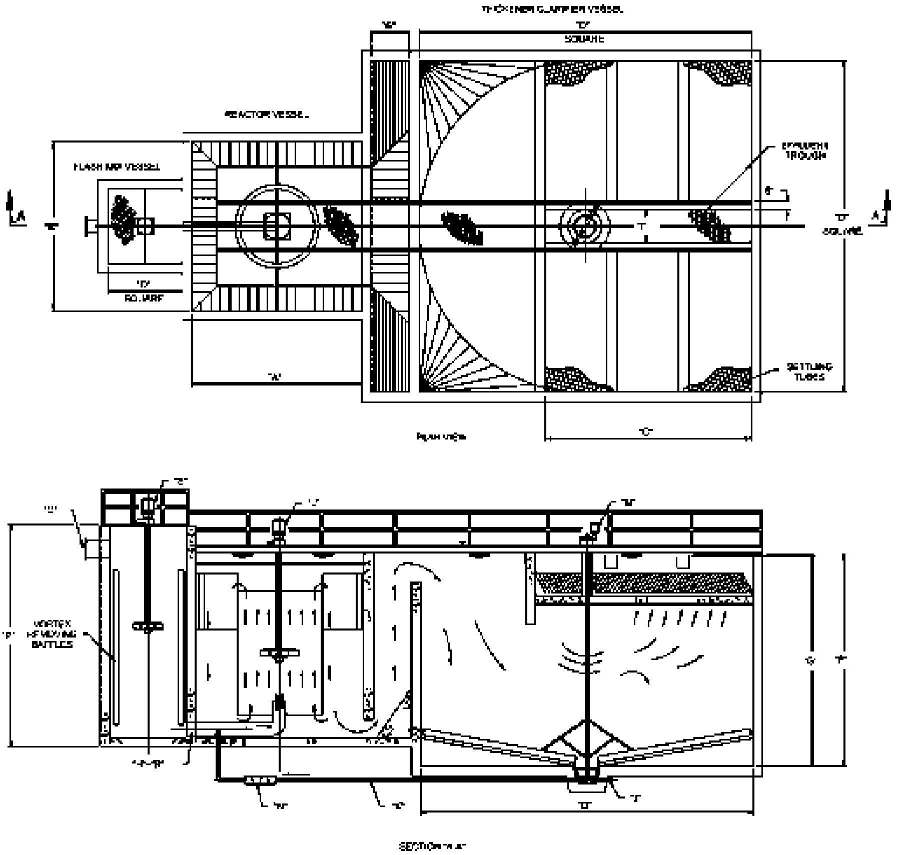


Fig. 11–24. DensaDeg high rate clarifier and thickener plan and section views (Courtesy of Infilco Degremont, Inc.)

TABLE 11–8. Effect of Sludge Density Due to Recirculation

Cycle No.	5-Min. Sludge Volume, mL	5-Min. Sludge Calculated Dry Solids, grams/L
1	50	3.66
4	87	7.1
10	155	9.65
15	163	13.7
20	162	18.2
25	158	23.4
30	160	27.7
35	163	31.5
40	163	36.0
45	158	41.8
51	158	47.2

(Courtesy of Infilco Degremont, Inc.)

rus, and places where large quantities of metal salts are added. However, these clarifiers can be used in more typical applications based on an appropriate analysis. DensaDeg units are in service at plants in Wilmington, Delaware, treating river water, and in Richfield, Minnesota, softening well water.

Actiflo Process

The Actiflo high-rate clarification system, illustrated in Figure 11–25, is in service in several drinking water plants in the United States and is widely used in Europe. There are separate rapid mixing, flocculation, and sedimentation compartments in the Actiflo process. The process utilizes microsand to enhance flocculation and settling. Particles adhere to the microsand and are removed from a center hopper in the sedimentation compartment. Settled solids are pumped to a hydrocyclone, where the microsand is separated and reused. The lower-density sludge is discharged from the top of the hydrocyclone.

The first full-scale Actiflo plant in the United States was put in service in May 1998 at the North Table Mountain Water Treatment Plant in Golden, Colorado. There are two Actiflo units in parallel that operate at a nominal overflow rate of 23,000 gpd/ft^2 (39 m/h). Alum is the primary coagulant at a dosage of 15–25 mg/L. Polymer is also used. The Actiflo system replaced a conventional coagulation, flocculation, and sedimentation plant. The clarified water in the conventional plant was 4–6 NTU. After nine months of operation, the Actiflo clarified effluent was 0.4–0.7 NTU.

FLOTATION

Introduction

Flotation processes have been used for solid-liquid separations since the early 1900s. The main types of flotation are electrolytic flotation, dispersed-air flotation, and dissolved-air flotation (DAF). These processes differ mainly in how the gas bubbles are produced. Dissolved-air flotation is by far the most common of these methods in the drinking water industry and is the focus of this section.

While DAF is still an emerging technology for drinking water treatment in North America, the process is well accepted in other parts of the world, including the United Kingdom, the Netherlands, Belgium, South Africa, and the Scandinavian countries. However, dissolved air flotation is commonly used for sludge thickening in wastewater plants in the United States. The first application of DAF for potable water treatment was in the early 1960s in South Africa, and other systems were built shortly thereafter in Scandinavia.[17] In Finland, there are presently over 30 DAF plants serving about 20 percent of the country's population.[18] By 1987, contracts for more than 25 DAF plants had been awarded in the United Kingdom.[17]

The first DAF plant in the United States was built in 1982 in Lenox, Massachusetts. There now are over a dozen full-scale DAF plants in operation in the United States.[19] Pilot studies for several large U.S. and Canadian cities have recommended including DAF in the treatment schemes.[20,21]

General Description

Dissolved-air flotation is used as a clarification process in drinking water treatment, providing an alternative to sedimentation in a conventional treatment plant and an

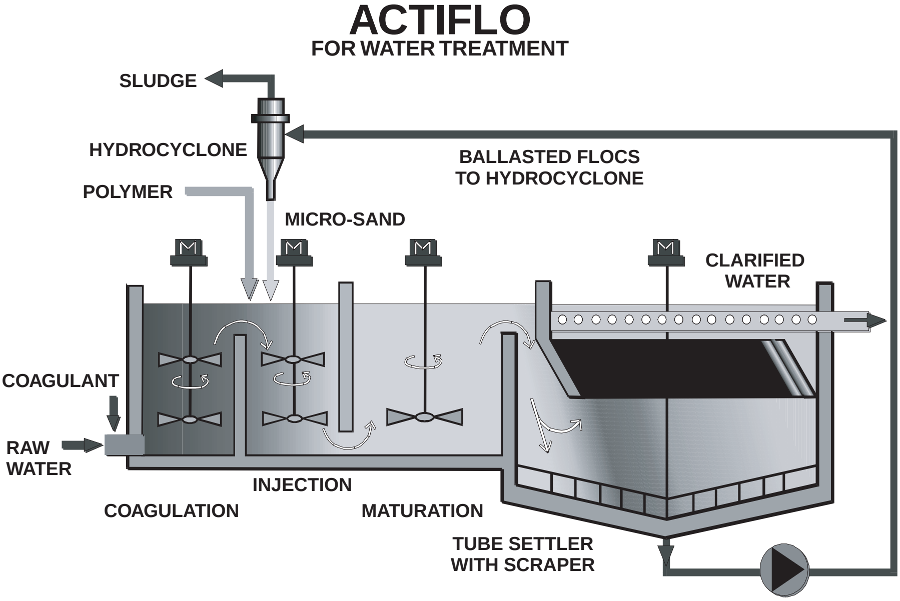


Fig. 11–25. Actiflo process—A high rate clarification system (Courtesy of Krüger, Inc.)

improvement to in-line and direct filtration plants. Figure 11–26 presents a schematic diagram of a typical DAF treatment plant. The processes upstream and downstream of a DAF unit are similar to those in a conventional treatment plant.

In the DAF process, a recycle stream is saturated with air at high pressure and then injected into the flotation tank to mix with the incoming flocculated water. As the recycle stream enters the flotation tank, the drop in pressure results in the release of the now supersaturated dissolved air. As the air bubbles form, they attach to floc particles and create a layer of sludge (or float) at the surface of the tank. The float is removed either by a mechanical scraper or by flooding the tank over a weir. The clarified water is collected near the bottom of the tank and passes to the filters. Figure 11–27 is a picture of the DAF facilities at the Arvika Water Treatment Plant in Sweden.

DAF is particularly effective in removing low-density particles from water, such as algae, protozoan cysts, coagulated natural organic matter, and floc from alum and ferric salts used on low-turbidity, soft waters. These particles are a particular problem for a conventional treatment plant because they do not readily settle. They usually require high coagulant doses and long flocculation and settling times. Carryover of these particles to the filters causes shorter filter runs. DAF is often the best available technology in these circumstances.

Principles

Removal of particles in flotation involves three key steps: (1) bubble formation, (2) bubble-particle attachment, and (3) flotation of the bubble-particle agglomerate. Some principles of these steps are discussed in the following paragraphs.

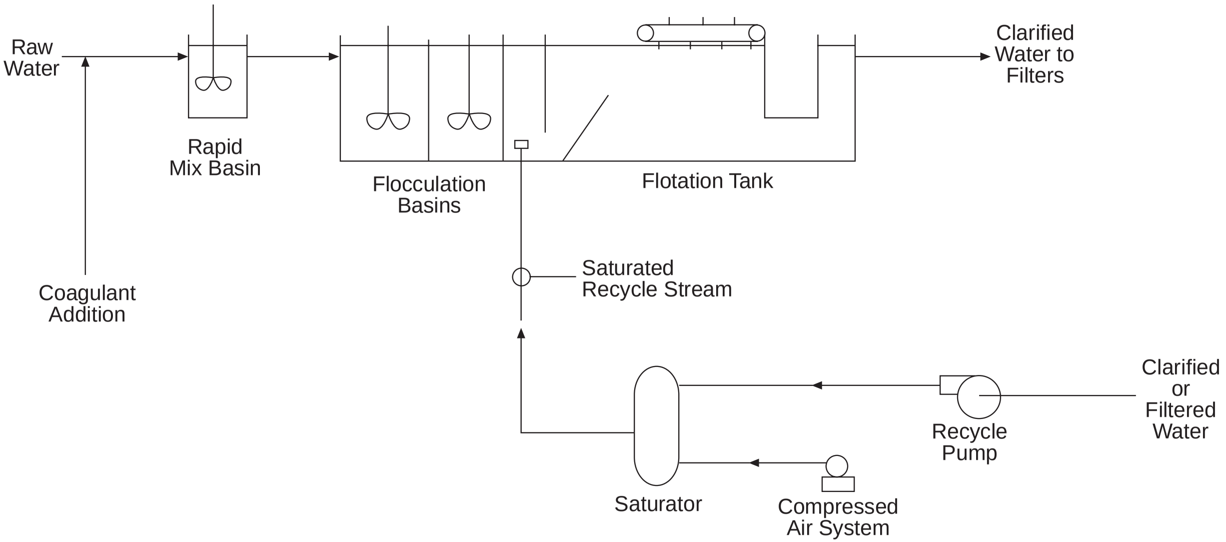


Fig. 11–26. Dissolved air flotation clarification system schematic

Fig. 11–27. Dissolved air flotation system at the Arvika Water Treatment Plant in Sweden (Courtesy of The F. B. Leopold Company Inc.)

Bubble Formation Bubbles are formed in the DAF process as the recycle stream enters the flotation tank and the pressure drops. The amount of air released can be approximated based on the solubility of air in water and the system operating conditions.

Solubility of Air in Water. The solubility of air in water depends upon both temperature and pressure. Assuming air is a single, ideal gas, the relationship between these parameters is described by Henry's law:

$$C_{\text{air}} = P_{\text{atm}}/K_H \tag{11–8}$$

where:

C_{air} = the concentration of air in water
P_{atm} = atmospheric pressure
K_H = Henry's law constant, which varies with temperature (4.18 kPa/mg/L at 20°C)

The amount of dissolved air predicted by Equation 11–8 is not fully achieved in DAF saturation systems. Lower air concentrations result because of the particular equilibrium conditions and mass transfer limitations in the saturator. Air is made up of mostly nitrogen and oxygen, which have different solubilities in water. The solubility of oxygen is greater than that of nitrogen, and this difference leads to a nitrogen-rich atmosphere in a saturator tank at equilibrium. The result is that there is approximately 9 percent less dissolution of gas in a saturator than is predicted by Henry's law.[22] The concentration of air in solution in a saturator system is also limited by the rate at which air is dissolved (i.e., by the mass transfer rate). In order to increase mass transfer, systems are designed with eductors and packing material that improve the mixing of air and water.

To account for the overall discrepancies between the dissolved concentration of air as predicted by Henry's law and that achieved in practice, saturators are rated with an all-inclusive efficiency term. Depending on design, efficiencies range from about 60 to 90 percent.

Amount of Air Released. To determine the amount of air released in a continuous-flow DAF unit, a mass balance is performed before and after the recycle is mixed with the DAF influent.[23] The following equation results:

$$C_r = \frac{(C_s - C_a)R_r - k}{1 + R_r} \tag{11–9}$$

where:

C_r = the mass concentration of air released in the flotation tank
C_s = the concentration of air in the saturated recycle stream
C_a = the concentration of air remaining in the solution at atmospheric pressure
R_r = the recycle ratio, calculated as the flow of recycle divided by the plant flow
k = the raw-water saturation factor, given by $C_a - C_0$
C_0 = the concentration of air in the raw water

In most cases, the raw water is saturated and therefore k is zero.[23] The mass concentration of air released is plotted against the percent recycle for three typical saturator pressures in Figure 11–28A.

The process of flotation relies on the physical actions of particle-bubble collisions and particle density reduction for good particle removal. The extent of these actions can be investigated based on the volume of bubbles released. The air or bubble volume concentration, ϕ_b, is determined as follows:

$$\phi_b = C_r/\rho_{\text{sat}} \tag{11–10}$$

where:

ρ_{sat} = the density of air saturated with water (1.19 mg/cm^3)

Figure 11–28B shows the bubble volume concentration plotted against the percent recycle.

It can also be instructive to consider the number of bubbles released in DAF systems. Based on the bubble volume concentration calculated in Equation 11–9, the number concentration of bubbles, N_b, can be calculated as follows:

$$N_b = 6\phi_b/\pi d_b^3 \tag{11–11}$$

where:

d_b = the bubble diameter

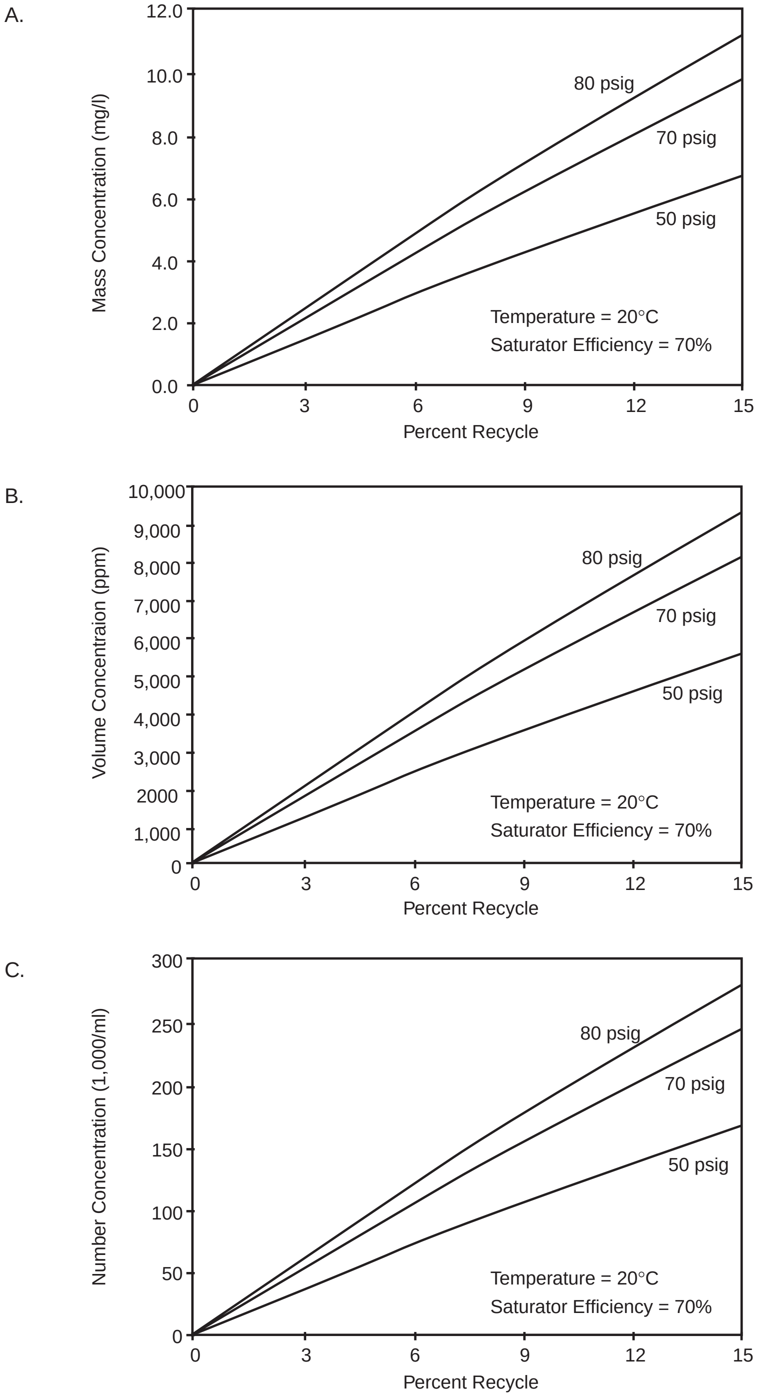


Fig. 11–28. Bubble mass (A), volume (B), and concentration (C) as a function of percent recycle (Adapted from Reference 28)

Bubbles released in DAF systems have been measured in the size range of 10 μm to 120 μm, with a mean diameter of 40 μm.[24,25] Figure 11–28C shows the bubble number concentration versus the percent recycle flow, assuming a bubble diameter of 40 μm.

Bubble-Particle Attachment Researchers have suggested three possible mechanisms for the formation of bubble-floc agglomerates[26,27] entrapment of bubbles within floc particles, formation of bubbles within floc particles, and bubble-floc attachment due to collision. In water treatment, the third mechanism is considered the most important.[28]

For good particle-bubble agglomeration, the particles must be destabilized. Two conditions of the particles are significant: neutral charge and hydrophobic surfaces.[28] These conditions lead to the strong attachment between particle and bubble necessary for successful flotation. Proper coagulant dosing and pH conditions result in low particle charge and the formation of hydrophobic particle surfaces. The concepts involved are similar to those applied in sedimentation and filtration and in direct filtration.

Flotation of the Particle-Bubble Agglomerate Particles are floated in DAF by attached bubbles that reduce the density of the particle-bubble agglomerate. Table 11–9 shows the density of alum floc and clay particles with a single bubble attached, similar to the analysis presented by Edzwald.[28]

The densities of alum floc and clay particles were assumed to be 1.01 g/cm^3 and 2.00 g/cm^3, respectively, and a bubble size of 40 μm was used. All particle-bubble agglomerates with densities less than that of water (i.e., 1.00 g/cm^3) would be expected to float. As the table shows, all of the alum floc and the 1-μm and 10-μm clay particles would float; however, the 100-μm clay particle, with a density of 1.88 g/cm^3, would not float. This large clay particle would require 16 attached bubbles for it to float.

To be removed from the flotation tank, particle-bubble agglomerates must rise quickly enough to reach the tank surface. Agglomerates that do not reach the tank surface will be swept out with the clarified water. Estimates of particle-bubble rise velocities can be made using Stokes law.[22,28] For example, it can be shown that the 100-μm alum floc with a single bubble attached (see Table 11–9) would rise in a quiescent tank at a velocity of 3.3 ft/hr (1.0 m/h). This may be too slow for removal, and the attachment of more bubbles may be necessary. Calculations of rise velocity are instructive; however, when the hydrodynamics of the flotation tank are considered, the situation becomes significantly more complex. As a result, prior experience is important in establishing the design factors for a particular application.

TABLE 11–9. Densities of Particle-Bubble Agglomerates for Single Bubble Attachment

Particle Diameter (μm)	Density of Particle-Bubble Agglomerate (g/cm^3): Alum Particle and Bubble	Density of Particle-Bubble Agglomerate (g/cm^3): Clay Particle and Bubble
1	0.00120	0.00122
10	0.0167	0.0319
100	0.949	1.88

Note: Bubble size of 40 μm; density of alum floc alone = 1.01 g/cm3; density of clay alone = 2.00 g/cm^3.

System Description

Tanks Both circular and rectangular tank configurations are used for DAF, though circular tanks are mostly limited to smaller or packaged treatment plants. Advantages of rectangular designs include greater hydraulic efficiency between processes and smaller space requirements.

DAF units are typically designed with overflow rates of 2 to 6 gpm/ft^2 (5 to 15 m/h), detention times of 5 to 20 min, and tank water depths of 5 to 12 ft (1.5 to 3.7 m) Rectangular tanks vary from nearly square to long and thin. Tank length is more important than width in process performance.[17] Short tanks do not allow enough time for floc-bubble agglomerates to rise to the tank surface; some particles then exit the tank with the clarified water. However, long tanks can result in an area at the end of the tank under which bubbles do not rise and therefore do not support the sludge layer. Tank width is usually sized based on the tank length and overflow rate, though compatibility with the size of the floc basins must be maintained.

A typical DAF tank is designed in such a way that flocculated water enters at the bottom of one end, where the saturated recycle injectors are located (see Fig. 11–26). An inclined baffle, normally 60° from the horizontal, is located near the inlet of the tank. The baffle separates the tank into reaction and separation zones. The reaction zone, which is the area upstream of the baffle, is where particles and bubbles mix, leading to the formation of particle-bubble agglomerates. The inclined baffle also aids in directing the agglomerates toward the tank surface. Extending the baffle to 1.1 to 1.5 ft (0.34 to 0.46 m) below the water surface has shown good results.[17] Lesser depths can result in high velocities over the baffle and poor tank hydrodynamics.

Clarified water is collected at the end of the DAF tank. Often the sludge collection trough acts as a baffle under which the clarified water passes (see Fig. 11–26). An effluent weir and trough located behind the sludge trough is then used to collect the clarified effluent. Another design uses a series of perforated laterals for the collection of clarified water. The laterals extend out from the end of the tank and are located above the bottom of the tank to avoid drawing in settled material.

Air Saturation System In a DAF plant, a sidestream of treated water is passed through an air saturation system and then to the DAF tank. Several types of air saturation equipment are used, including packed and unpacked saturators, eductors to entrain the air, and recycle pumps with air injection on the suction side. The air saturation system accounts for about one-half the power used in the DAF process,[25] so efficiency is an important consideration in recycle system design.

Packed saturators, with typical efficiencies of 90 percent, are more efficient than unpacked saturators, which have typical efficiencies of 70 percent. The advantages of the unpacked design are that it is less expensive and easier to maintain. With packed saturators, one concern is biological growth and the accumulation of precipitates on the packing material. Also, the high efficiency of packed saturators can lead to bubble coalescence in the piping downstream of the saturator.[17] The release of these relatively large bubbles in the flotation tank can disturb the sludge layer at the tank surface.

Air release devices are used to create a pressure drop as the recycle stream enters the flotation tank. With the drop in pressure, the air dissolved in the recycle stream is released as bubbles, the size of which can depend strongly on the type of device. Several types of nozzles and valves are used, including proprietary and nonproprietary designs. Nozzles are generally designed with a tortuous flow path, used to create turbulence and consequently a more rapid release of air. The size distribution of bubbles released is largely determined by the configuration of the air release device.[29]

The velocity and distribution of the recycle stream as it mixes with the flocculated water are essential considerations in designing and locating air release devices. High recycle velocities will shear floc particles. In some cases, to reduce recycle stream velocity, air release devices have been installed in pipes upstream of the flocculated water. However, this design has caused problems with bubble coalescence and poor process performance. Although there are no specific guidelines for locating air release devices, generally they should be installed so as to produce an even distribution and good mixing of air with the flocculated water.

The method of controlling the rate of recycle must be considered in conjunction with the type of air release device used. Whereas valves can be used to control recycle flow, nozzles must be sized for the recycle flow needed. If a large range of flows is expected in a single DAF unit, shutoff valves upstream of the nozzles can be used. In some systems, flow control valves have been used with nozzles. However, the valves can create a large enough pressure drop to release air before the nozzles.[25] Subsequent bubble coalescence can reduce process performance.

Float Removal System Both mechanical and hydraulic devices are used for removing the float from the surface of a flotation tank. With a hydraulic system, the float is intermittently carried over a weir along with water from the tank. This is done either by restricting the tank outlet or by lowering a weir. An advantage of hydraulic systems is that they are inexpensive and easy to maintain. The main disadvantage of these systems is the dilute sludge produced, typically less than 0.2 percent solids.[25] Mechanical float removal devices have the advantage of creating a thicker sludge (1 to 3 percent solids) and the ability to operate either continuously or intermittently. Mechanical systems utilize one of two design types: (1) partial- or full-length scrapers located above the flotation tank that push the float along the surface of the tank and over the beach, or (2) rotating beach scrapers that only push the float up and over the beach. Scrapers are typically made of rubber blades or brushes. An advantage of beach scrapers is their smaller, simpler design. Beach scrapers also avoid float knockdown due to the scrapers entering the tank. The sludge produced with beach scrapers is thinner, however, because the float flows with the water to the sludge beach.

Key operational considerations with mechanical scraper devices are the rate and frequency of sludge removal. Some sludges are best removed continuously, whereas others may remain stable at the tank surface for over 24 hr. Also, with a stable sludge, scrapers can be operated at higher speeds without risking float breakup. The type of floated sludge produced and the appropriate timing of removal depend on the raw-water quality, chemical treatment, amount of air released, and tank configuration. Based on several applications, Zabel notes that soft, highly colored water shows float breakup within 30 min of accumulation, whereas a turbid or algae-laden water results in a float that is stable for over 24 hr.[25]

Combined Flotation and Filtration In some designs, flotation and filtration are combined in one tank (Fig. 11–29). The filter covers most of the tank bottom, with a small area used for the tank inlet and recycle injection system. The overall depth is greater than that of a conventional DAF tank to allow for the filter media and underdrain. Combining the two processes allows for a highly compact plant, but it offers less flexibility for plant design and operation. For instance, the loading rates of the

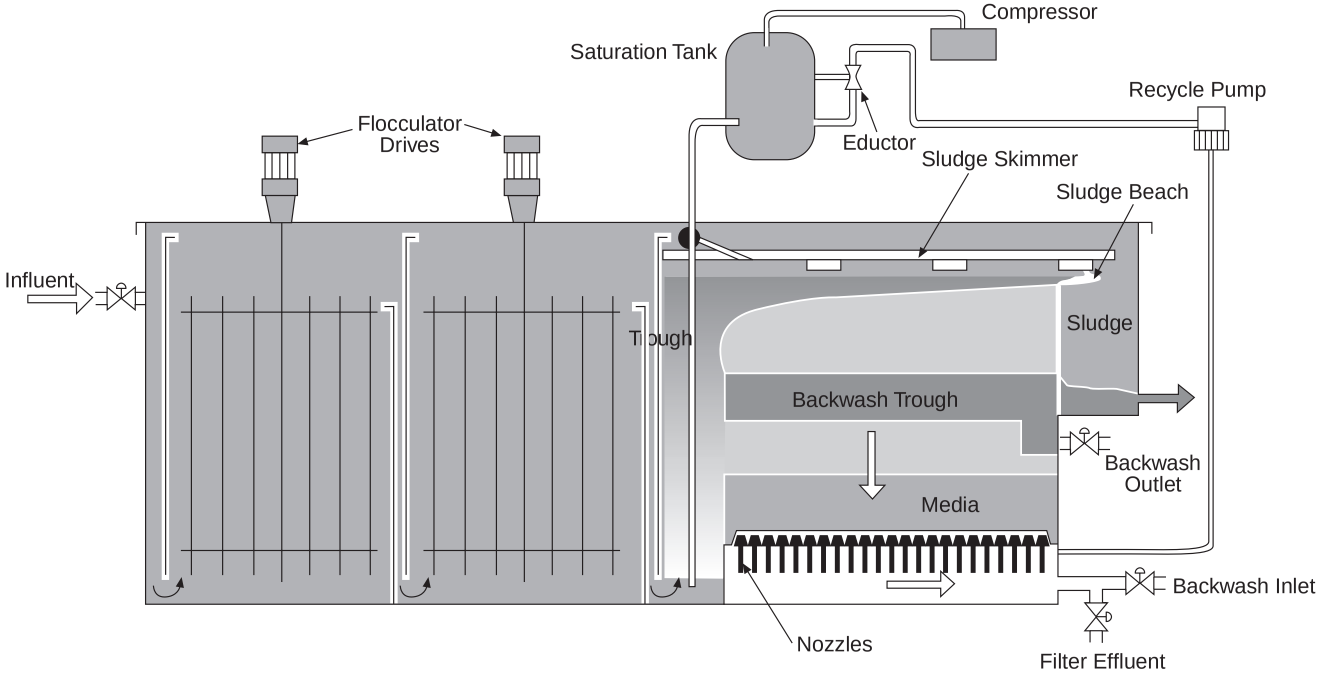


Fig. 11–29. The WaterLink FloFilter™ package water treatment system, which combines two-stage flocculation, flotation clarification, and filtration (Courtesy of Waterlink Separations, Inc.)

two processes are linked. In addition, the flotation process must be stopped during the filter backwash process, so the recovery time for the flotation process needs to be considered.

Factors Influencing DAF Performance

Coagulation Particle destabilization by coagulation is required for particle-bubble attachment that results in effective particle removal. Coagulant addition is also required to precipitate and/or adsorb dissolved organic material, allowing the material to be removed by flotation. This is the same as with coagulation used for sedimentation. However, unlike pretreatment for sedimentation, DAF does not require large, dense floc.[30] Small floc particles are as well or even better removed by DAF than large floc particles. Chemical dosages are commonly 30 percent less than those required for sedimentation.

Several coagulants have been shown effective with DAF, including alum, ferric salts, and polyaluminum chlorides.[31] The most effective chemical will depend upon the raw-water quality. Coagulant dose and pH have a strong effect on DAF and should be tested at the pilot and/or bench scale. Generally, flocculent aid polymers are not necessary for DAF, though they can improve process performance in some cases.

Flocculation Until recently, DAF plants have been designed with flocculation times of 20 to 30 min or more.[32] These times are similar to those used in sedimentation, where large, dense floc is desired. Less flocculation time, however, can be sufficient or even better for DAF performance. New designs typically call for between 5 and 15 min flocculation time.[19,20,33] These shorter times tend to produce ‘‘pinpoint’’ floc—similar to that produced for direct filtration—that is effectively removed by DAF. The practical consequences of the short flocculation time used in DAF are a more compact layout and lower capital cost than for a facility designed for sedimentation.

Flocculation intensity is also important in producing appropriate floc for flotation. Whereas early DAF plants used mixing intensities similar to those for sedimentation, more intense mixing, without tapering, is now commonly used prior to DAF. Zabel found an optimum mean velocity gradient (G) of 70 sec^{-1} for flotation, compared to 10 to 50 sec^{-1} used for sedimentation.[25] Others have reported that mixing intensities from 50 sec^{-1} to 120 sec^{-1} can be used.[31,33] Bunker et al. suggested that higher mixing intensities are better for flotation because stronger floc is needed.[31] The high shear conditions present where the recycle stream enters the flocculated water tends to break up weaker floc into sizes that are not effectively removed by flotation.

Air Requirements The amount of air released in a flotation tank depends on four main system conditions: (1) recycle flow rate, (2) saturator efficiency, (3) saturator pressure, and (4) water temperature. The recycle flow rate is the most commonly used parameter in describing and controlling the amount of air released. The saturator efficiency and water temperature cannot be controlled by the plant operator. The saturator pressure can sometimes be adjusted, but normally it is not. One should keep in mind, however, when comparing the performance of two systems, that the recycle ratio does not completely describe the amount of air released. As discussed earlier in this discussion of flotation, the quantity of air should be described in terms of mass or volume concentrations.

The quantity of air required for good DAF performance depends on the raw water quality and the treatment steps upstream of the flotation tank.[19,23] Generally, DAF plants recycle between 5 and 15 percent of the plant flow through the air saturation system. For a saturator pressure of 80 psig (550 kPa [gauge]), saturator efficiency of 70 percent, and water temperature of 50°F (10°C), recycle ratios of 5 and 15 percent correspond to bubble volume concentrations of 3,000 and 8,100 ppm, respectively.

Advantages and Disadvantages of DAF

DAF has several advantages over conventional sedimentation. These advantages can mean better performance of the unit process and overall plant, as well as lower capital and operations costs.

- *More compact design.* The higher loading rates used with DAF allow for much smaller tanks than those used in sedimentation. Although the saturation system in a DAF plant requires space that is not needed in sedimentation, the overall footprint of the DAF process is still substantially smaller.
- *Shorter start-up time.* The smaller tanks used with DAF result in good effluent quality in less time. Typical start-up times for DAF are about 45 min, whereas a sedimentation basin may require hours to produce a good clarified water.
- *Better performance.* For many water quality types, DAF produces a superior effluent than sedimentation. This is particularly true for waters containing low-density particles.
- *Lower chemical dose.* In many cases, DAF requires less coagulant than sedimentation for optimal performance. Unlike sedimentation, DAF does not rely on the formation of large, dense floc for effective particle removal.
- *Shorter flocculation time.* DAF typically performs well with pinpoint-size floc. Flocculation times for DAF are therefore one-half to one-fifth those used with sedimentation.
- *Thicker sludge.* The floated sludge from a DAF tank normally has a much higher solids concentration than sludge produced from sedimentation.

Although DAF offers several advantages over conventional sedimentation, the conditions of each individual application must be considered to determine the best process. Sedimentation is more appropriate for some types of water quality, particularly waters with high turbidity and those requiring softening. In addition, where land is available, the size of sedimentation basins may not be an issue. The mechanical equipment used in DAF requires a higher level of operator skill and more maintenance than sedimentation. These aspects may be undesirable to a utility.

The relative costs of DAF and sedimentation depend on several factors, including the plant capacity, location, and water quality. When comparing the two processes, a utility needs to consider both capital costs and operation and maintenance (O&M) costs. The capital costs are lower for the smaller DAF tanks, but equipment and equipment housing costs are higher. For a new plant, the smaller flocculation basins needed for DAF may make for lower overall capital costs. Energy costs are higher with DAF because of the recycle/saturation system, but lower chemical doses often offset the energy costs.

REFERENCES

1. Weast, R. C., Astle, M. J., and Beyer, W. H., *CRC Handbook of Chemistry and Physics,* 66th ed., CRC Press, Boca Raton, FL, 1985.
2. AWWA (American Water Works Association), *Water Treatment Plant Design,* AWWA, New York, 1969. AWWA/ASCE, *Water Treatment Plant Design,* 3rd ed., McGraw-Hill, New York, 1998.
3. Camp, T., R., ‘‘Sedimentation and the Design of Settling Tanks,’’ *Transactions of the ASCE* 111:895, 952, 1946.
4. Zanoni, A. E., and Blomquist, M. W., ‘‘Column Settling Tests for Flocculent Suspensions,’’ *Pro., American Society of Civil Engineers* 101(EE3):309–318, June 1975.
5. Weber, W. J., *Physicochemical Processes for Water Quality Control,* Wiley-Interscience, New York, 1972.
6. Pontius, Frederick W. (AWWA technical editor), *Water Quality and Treatment: A Handbook of Community Water Supplies,* 4th ed., McGraw-Hill, New York, 1990.
7. GL-UMR BSPHEM (Great Lakes–Upper Mississippi River Board of State Public Health & Environmental Managers), *Recommended Standards for Water Works* (1997 ed.), Health Education Services, Albany, NY, 1997.
8. GL-UMR BSSE (Great Lakes–Upper Mississippi River Board of State Sanitary Engineers). *Recommended Standards for Water Works* (1968 ed., with 1972 revisions), Health Education Services, Albany, NY, 1972.
9. Walton, Graham, *Effectiveness of Water Treatment Processes,* Publication No. 898, Washington, DC: U.S. Public Health Service, 1962.
10. Kawamura, S., ‘‘Hydraulic Scale-Model Simulation of the Sedimentation Process,’’ *JAWWA,* p. 372, July 1981.
11. Hudson, H. E., Jr., *Water Clarification Processes: Practical Design and Evaluation,* Van Nostrand Reinhold, New York, 1981.
12. Stukenberg, J., Rodman, L. C., and Touslee, V. E., ‘‘Activated Sludge Clarifier Design Improvements,’’ paper presented at the 54th Annual Water Pollution Control Federation Conference, Detroit, MI, October 1981.
13. Hazen, A., ‘‘On Sedimentation,’’ *Transactions of the ASCE* 53:45, 1904.
14. Eliassen, R., discussion of (1)^2. *Transactions of the ASCE* 111:947, 1946.
15. Conley, W. R., and Slechta, A. F., ‘‘Recent Experiences in Plant Scale Application of the Settling Tube Concept,’’ paper presented at the Water Pollution Control Federation Conference, Boston, MA, October 1970.
16. Culp, G. L., Hsiung, K.Y, and Conley, W. R., ‘‘Tube Clarification Process: Operating Experiences,’’ *Journal of the Sanitary Engineering Division, ASCE* SA5(October):829, 1969
17. Longhurst, S. J., and Graham, N. J. D., ‘‘Dissolved Air Flotation for Potable Water Treatment,’’ *Public Health Engineer* 14(6):71–76, 1987.
18. Heinänen, J., Jokela, P., and Ala-Peijari, T., ‘‘Use of Dissolved Air Flotation in Potable Water Treatment in Finland,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.
19. Valade, M . T., Edzwald, J. K., Tobiason, J. E., Dahlquist, J., Hedberg, T., and Amato, T., ‘‘Particle Removal by Flotation and Filtration: Pretreatment Effects,’’ *JAWWA* 88(12):35–47, 1996.
20. Johnson, B. A., Gong, B., Bellamy, W., and Tran, T., ‘‘Pilot Plant Testing of Dissolved Air Flotation for Treating Boston’s Low Turbidity Surface Water Supply,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.

21. Ferguson, C., Logsdon, G. S., and Curley, D., ‘‘Comparison of Dissolved Air Flotation and Direct Filtration,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.
22. Gregory, R., and Zabel, T. F., ‘‘Sedimentation and Flotation,’’ in *Water Quality and Treatment,* 4th ed. (edited by American Water Works Association), McGraw-Hill, New York, 1990.
23. Edzwald, J. K., Walsh, J. P, Kaminski, G. S, and Dunn, H. J., ‘‘Flocculation and Air Requirements for Dissolved Air Flotation,’’ *JAWWA* 84(3):92–100, 1992.
24. Takahashi, T., Miyahara, T., and Mochizuki, H., ‘‘Fundamental Study of Bubble Formation in Dissolved Air Pressure Flotation,’’ *J. Chem. Engr. Japan* 12:275–280, 1979.
25. Zabel, T. F., ‘‘The Advantages of Dissolved-Air Flotation for Water Treatment,’’ *JAWWA* 77(5):42–46, 1985.
26. Kitchener, J. A., and Gochin, R. J., ‘‘The Mechanism of Dissolved Air Flotation for Potable Water: Basic Analysis and a Proposal,’’ *Wat. Res.* 15:585–590, 1981.
27. Zabel, T. F., ‘‘Flotation in Water Treatment,’’ pp. 431–454 in *Innovations in Flotation Technology,* Kluwer Academic Publishers, The Netherlands, 1992.
28. Edzwald, J. K., ‘‘Principles and Applications of Dissolved Air Flotation,’’ *Wat. Sci. Tech.* 31(3–4):1–23, 1995.
29. Rykaart, E. M., and Haarhoff, J., ‘‘Behavior of Air Injection Nozzles in Dissolved Air Flotation,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.
30. Edzwald, J. K., and Walsh, J. P., *Dissolved Air Flotation: Laboratory and Pilot Plant Investigations,* American Water Works Association Research Foundation and American Water Works Association, Denver, CO, 1992.
31. Bunker, D, Q., Jr., Edzwald, J. K., Dahlquist, J., and Gillberg, L., ‘‘Pretreatment Considerations for Dissolved Air Flotation: Water Type, Coagulants, and Flocculation,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.
32. Edzwald, J. K., Olson, S. C., and Tamulonis, C. W., *Dissolved Air Flotation: Field Investigations,* American Water Works Research Foundation and American Water Works Association, Denver, CO, 1994.
33. Haarhoff, J., and van Vuuren, L. R. J., ‘‘Design Parameters for Dissolved Air Flotation in South Africa,’’ paper presented at the International Specialized Conference on Flotation Processes in Water and Sludge Treatment, Orlando, FL, April 26–28, 1994.

SUPPLEMENTARY READING

AWWA (American Water Works Association) Committee on Viruses in Water, ‘‘Viruses in Water,’’ *JAWWA* 6(10):491, 1969.

Culp, G. L., Hansen, S. P., and Richardson, G. H., ‘‘High Rate Sedimentation in Water Treatment Works,’’ *JAWWA* 60:681, 1968.

Hansen, S. P., and Culp, G. L., ‘‘Applying Shallow Depth Sedimentation Theory,’’ *JAWWA* 59:1134, 1967.

McMichael, F. C., ‘‘Sedimentation in Inclined Tubes and Its Application for the Design of High-Rate Sedimentation Devices,’’ paper presented at the International Union of Theoretical and Applied Mechanics Symposium on Flow of Fluid-Solid Mixtures, University of Cambridge, England, March 24–29, 1969.

Yao, K. M., ‘‘Theoretical Study of High-Rate Sedimentation,’’ *JWPCF* 42(2):218–228, 1970.

CHAPTER 12

Filtration

INTRODUCTION

In waterworks parlance, filtration most frequently refers to the use of a relatively deep [1½ to 6 feet (0.46 to 1.82 m)] granular bed to remove particulate impurities from water. Water filtration is a physical-chemical process for separating suspended and colloidal impurities from water by passage through a bed of granular material. There are two separate steps:

- Transport, in which suspended particles are transported to the immediate vicinity of the solid filter media
- Attachment, in which particles become attached to the filter media surface or to another particle previously retained in the filter

The transport step is primarily a physical process, while the attachment step is very much influenced by chemical and physical-chemical variables.

THE ROLE OF FILTRATION IN WATER TREATMENT

It is generally acknowledged that most surface water supplies should be filtered; but a significant number of surface water supplies derived from uninhabited, ‘‘protected’’ watersheds still are not filtered. These unfiltered systems are undergoing rigorous review by state and federal agencies to assess the need for filtration to supplement disinfection provided by most facilities.

Some well waters meet water quality standards and goals with no treatment other than chlorination. However, many well supplies require removal of iron, manganese, hardness, color, odor, turbidity, hydrogen sulfide, bacteria, viruses, or other undesirable impurities. In these cases, and in instances where the ground water quality is influenced by surface water, filtration is ordinarily a part of the overall treatment process.

In water treatment, the general practice is to reduce the turbidity of water to below 10 NTU before application to filters, with a desired goal of 1–2 NTU to ensure optimum filter performance. However, some types of filters can handle applied turbidities of up to 50 NTU on a continuous basis and will tolerate occasional turbidity peaks up to 200 NTU. Several public water supplies derived from surface sources having average turbidities of 20 to 50 NTU and peaks of 100 to 200 NTU have operated quite successfully without the use of settling basins ahead of the filters (direct filtration).

However, increasingly more stringent turbidity limits and concern regarding removal of pathogens such as *Giardia* and *Cryptosporidium* may require that pretreatment be provided to ensure compliance with the regulations.

Provisions should be made in plant design to disinfect both filter influent and effluent. Disinfectant applied ahead of a filter is a great aid in maintaining clean filters. Disinfection of the finished water makes the most effective use of the disinfectant because of the reduced disinfectant demand at this point in treatment. Provisions should also be made for application of a filter aid such as alum, activated silica, or a polymer ahead of the filters so that the water can be adjusted to optimize filterability.

Where there is no full plant operating experience with a particular water to be treated, or where the records are not adequate to determine the best treatment processes to be used, pilot plant studies are a valuable aid in producing the most efficient and economical scheme of treatment. Pilot studies can be used to determine the kind and extent of pretreatment that may be required ahead of filtration. A pilot filter can reveal through its operation the characteristics of floc particles and filter media that achieve optimal removal efficiency and length of filter runs. While filter-cleaning requirements are well standardized, it is prudent to check them through pilot plant operation on a particular water to be treated. In conducting pilot plant tests, the necessity to operate under all raw-water conditions must not be overlooked. Unless the pilot plant is operated under the most severe conditions, the full-scale plant may be underdesigned and ill-equipped to produce the desired water quality at all times.

FILTRATION TECHNOLOGIES

Filtration systems employed at water treatment facilities utilize a porous medium through which the water passes to remove suspended solids. A number of methods have been developed to accomplish this purpose. Currently, the most commonly used approach to filtration in water treatment plants is rapid rate gravity filtration. However, developments in filtration technology have increased the popularity of other types of filter systems.

Types of Filters Available

A number of types of filtration systems are technically feasible, depending on the application. System types that have been identified for possible consideration for most water treatment facilities are:

Gravity Filtration Systems

- Rapid rate gravity filtration
- Slow-sand filtration

Pressure Filtration Systems

- Rapid rate pressure filtration
- Diatomaceous earth filtration
- Membrane filtration
- Cartridge filtration

Gravity filters operate with relatively low hydraulic headlosses through the unit, while the pressure filters are more practical where higher headlosses are not detrimental. Pressure filters allow pumping through the unit at relatively high pressures, and are commonly used in small plants where they tend to be more economical. In larger plants, gravity filters are more prevalent, as pressure systems are limited by available vessel sizes. A significant concern with pressure filters is the inability to observe the filter during routine operation, particularly during cleaning of the filter.

Rapid Rate Gravity Filtration Rapid rate gravity filtration is the most widely used technology for removing turbidity and microbial contaminants from pretreated surface water and groundwater. Flow is normally downward, with the pretreated water passing through a granular bed. Solids accumulate within the voids and on the top surface of the filter bed. Plugging of the filter bed gradually occurs, resulting in either increased headloss or a reduction in flow through the filter. After a period of operation, the filter bed is cleaned using an upward flow of water referred to as *backwashing*. Treatment prior to the filters, consisting of chemical precipitation and sedimentation, is often used to reduce the quantity of filter influent particulates and ensure a more efficient removal of particulates by the filter.

Slow-Sand Filtration Slow-sand filters are similar to single-media, rapid-rate filters in some respects, yet they differ in a number of important characteristics. In addition to slower flow rates (by a factor of 50 to 100 versus rapid-rate filtration for example), slow-sand filters also:

- Function using biological mechanisms instead of physical-chemical mechanisms
- Have smaller pores between sand particles
- Do not require backwashing
- Have longer run times between cleaning.
- Require a ripening period at the beginning of each run

Although applicable to medium- to large-size plants, they are most commonly used for small communities because of the operational simplicity and large space requirements. Because of the revived interest in this filtration method, it is in some instances being classified as new technology. Slow-sand filters are effective for removal of *Giardia* and *Cryptosporidium*, providing over 3-log removal.[1]

Rapid Rate Pressure Filtration This filter type is similar to the gravity rapid rate filters, except that the flow enters and exits the filter under pressure. The filter bed is normally installed in a cylindrical steel pressure vessel, with connecting pressurized piping. The pressure vessel can be a vertical or horizontal cylinder. The filtration process is essentially the same as the gravity system, except for the elimination of concerns associated with negative pressures in the filter bed. Pressure filters are normally used in small water systems and industrial applications for iron and manganese removal from groundwater. Their use for filtration of surface waters or lime-softened water is not normally permitted.

Low-Head Continuous Backwash Filters A variation of the traditional gravity filter is the low-head continuous backwash filter. The filter usually consists of a shallow bed of sand about 1 ft (0.3 m) deep, with an effective size of 0.4 to 0.5 mm. The bed is divided into multiple compartments with the filtrate flowing to a common effluent channel. A traveling-bridge backwash system, equipped with a backwash pump and a backwash collection system, washes each cell of the filter in succession as it traverses the length of the filter. The backwash system is capable of traversing the filter intermittently or continuously.

Because the filter is washed frequently at low headloss, solids are removed primarily near the top surface of the fine sand. This facilitates removal of the solids during the short backwash period, which lasts only about 15 seconds for each cell.[1] An advantage of this filter is that it is backwashed frequently at low headloss and does not require as deep a filter box as a conventional gravity filter. This type of filter also eliminates the need for a filter gallery, a large backwash pump, and the associated large backwash piping.

There are some concerns over use of this type of filter in drinking water applications. Historically, designs are limited to shallow filters operating at rates of 2 gpm/sf or less and have not included filter-to-waste capability. The backwash is very brief and unassisted by auxiliary scour. Therefore, if the filter is blinded by short-term influent flow containing high levels of solids, cleanup of the filter may be difficult. The flow rate and filtrate quality from individual cells are also not monitored. During the backwashing operation, the cells that are washed first immediately return to service while the traveling bridge continues to backwash the remaining cells. The cells backwashed first operate at higher rates of filtration for a brief period during completion of the entire filter wash—leading to concern over the passage of solids through the newly cleaned cells during this period of higher filtration rate.

Diatomaceous Earth Filtration Diatomaceous earth (DE) filtration, also known as *precoat* or *diatomite filtration,* is applicable to direct treatment of surface waters for removal of relatively low levels of turbidity. Diatomite filters consist of a layer of DE about ⅛ inch thick supported on a septum of filter element. The problems inherent in maintaining a perfect film of DE between filtered and unfiltered water have restricted the use of diatomite filters for municipal purposes, except under favorable conditions. Although diatomaceous earth filtration has been used for a number of years, it is receiving renewed interest in light of the need to filter surface waters of low turbidity previously only chlorinated. Since it can remove (greater than 3-log removal) *Giardia* and *Cryptosporidium* cysts, it can be a viable filtration technology.

Membrane Filtration Membrane filtration has historically been utilized for the removal of salts or organic materials on high-quality untreated raw waters. There has been an increasing interest in membrane filtration for removal of particulates and microorganisms. Low-pressure membranes, in the form of either ultrafiltration (UF) or microfiltration (MF), have become more economical both in capital and operating costs and have received increased attention in drinking water applications. Please refer to Chapter 15, ‘‘Membrane Treatment,’’ for detailed information.

Cartridge Filtration Cartridge filters traditionally have been applied for point-of-use systems and for pretreatment prior to membrane treatment systems. Composed of membrane, fabric, or string filter mediums, the filter material is supported by a filter element and housed in a pressure vessel. Cartridge filters using microporous ceramic filter elements with pore sizes as small as 0.2 μm may be suitable for producing potable water from raw-water supplies containing moderate levels of turbidity, algae, and microbiological contaminates. Virus and bacteria can pass through most of these filters. Source waters should have little virus or bacteria contamination, as disinfection will be relied upon to control these microorganisms. Single-filter elements may be manifolded in a pressurized housing to produce higher flow capacities from a single assembly. The clean filter element pressure drop is about 45 psi at maximum capacity; filters are cleaned when the pressure drop reaches 90 psi.

The application of cartridge filters using either cleanable ceramic or disposable polypropylene cartridges appears to be a feasible method for removing modest levels of turbidity. The importance of selecting a proper pore size is complicated by the capability of cysts and oocysts to deform somewhat and to squeeze through pores seemingly small enough to prevent their passage. The efficiency and economics of the process must be closely evaluated for each application. Pretreatment in the form of roughing filters (rapid-sand or multimedia or fine-mesh screens) may be needed to remove larger suspended solids that could quickly foul the cartridges, reducing capacity. Prechlorination is recommended to prevent microbial growth on the cartridges and to inactivate any organism that might pass through the filter elements.

As water is filtered through a cartridge filter, the pressure drop eventually becomes so great that terminating the filter run is necessary. When this is done, the filter is not backwashed, but rather the cartridge is thrown away and replaced by a clean one. Filter run length has an important influence on cartridge filtration economics.

Filtration System Capabilities

Each of the above noted filtration technologies has varying particle removal capability. Figure 12–1 illustrates the range in particle size removal for several filtration technologies relative to several common contaminants found in water supplies.

THE EVOLUTION OF GRAVITY FILTRATION

Slow Sand

In waterworks history, three basic types of gravity filters have dominated the field at different times. In the 1800s, the slow-sand filter was dominant. It incorporated sand with an effective size of about 0.2 mm. Effective size is that size for which 10 percent of the grains are smaller by weight. This very fine sand produced good-quality water from applied water of low turbidity at rates on the order of 0.05 to 0.13 gpm/sq ft (0.12 to 0.32 m/h) of bed area. The filter was cleaned by scraping a thin layer of media from the surface of the filter, washing it, and returning the washed sand to the bed. Because of the low surface rates, slow-sand filters required large areas of land and were costly to install. They were also expensive to operate because of the laborious method of bed cleaning by surface scraping.

Rapid Sand

Beginning in the early 1900s, under the stimulus of epidemic waterborne disease, the rapid-sand filter came into general use, largely replacing the slow-sand filter. Rapid-

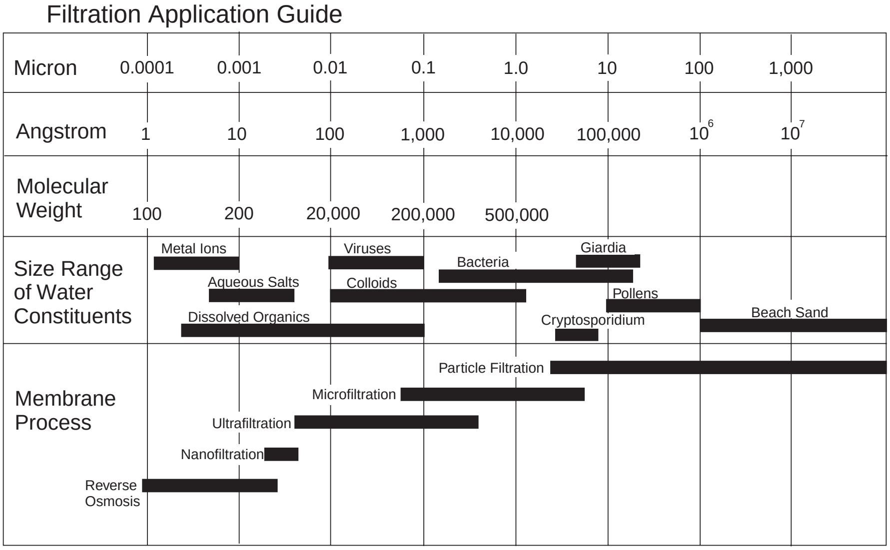


Fig. 12–1. Particle sizes removed by various filtration technologies (Courtesy of Dept. of Interior, Bureau of Reclamation, Water Quality Improvement Center, Yuma, AZ)

sand filter media vary in effective size from 0.35 to 1.00 mm, with a typical value being 0.5 mm with a uniformity coefficient of 1.3 to 1.7. Uniformity coefficient is a measure of the size range and is the ratio of the size for which 60 percent of the grains are smaller by weight to the size for which 10 percent are smaller. This type of fine media has demonstrated the ability to handle applied turbidities of 5 through 10 NTU at rates up to 2 gpm/sq ft (4.88 m/h). The introduction of prior chemical coagulation and hydraulic backwash made possible the use of rapid rather than slow-sand beds.

Rapid sand filters are cleaned by reversing the flow through the filter and backwashing the trapped particles from the bed. In backwashing single-medium sand beds, hydraulic grading of the sand grains occurs. The very finest sand accumulates at the top of the bed, and the coarser particles lie below. More than 90 percent of the particulates removed are taken out in the top few inches of the bed. Once a suspended particle has penetrated this top layer of fine sand, its chances are greatly increased for passing through the entire bed because the void spaces become larger and the opportunities for contact decrease as the particles travel downward. This is a well-recognized limitation of the rapid sand filter.

The pore openings in a rapid-sand filter are made up of 0.5 mm sand range from 0.1 to 0.2 mm in size. In the water applied to the filter, floc size ranges from 2 mm to less than 0.1 mm.[1] It follows from these dimensions that the larger floc particles can be removed by simple straining at the filter surface, but that much of the flocculated matter will pass into the filter and lodge within it. Unless the floc particles are exceptionally strong, those that initially lodge in the filter by simple straining will subsequently break as the hydraulic gradient increases.

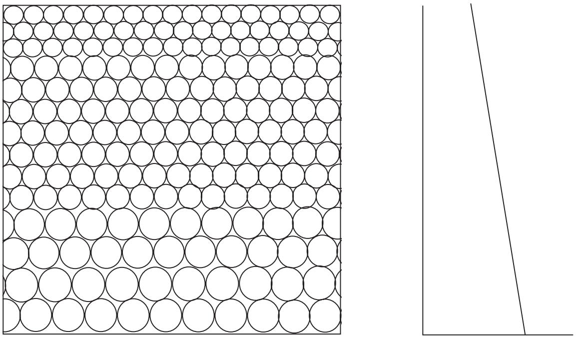
The flocculated material in the water passes into the filter through thousands of openings in each square foot of the filter surface and, by the end of a filter run, ordinarily lodges largely in the top 1 to 4 inches (25.4 to 101.6 mm) of the filter. The probability of a floc particle striking the surface of the filter medium is influenced by sand size, porosity, filter rate, temperature, and density and size of the floc particles. The adherence of the particles is affected by colloidal forces, including the age of the floc, temperature, coagulant concentration, type and concentration of anions, and pH.

Multimedia

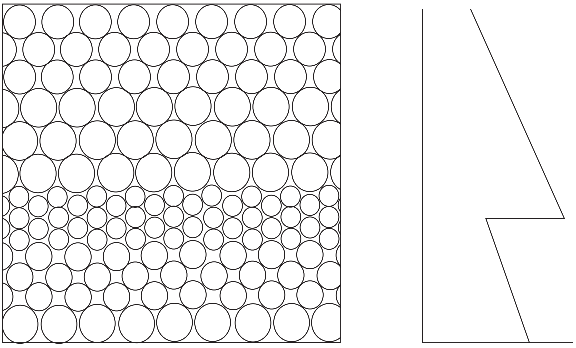
It was not until the early 1940s, under the stimulus of a critical wartime need to produce greatly improved water quality (basically much lower turbidity water) for processing radioactive materials at Hanford, Washington, that coarse-to-fine filtration had its beginnings under the leadership and direction of Raymond Pitman and Walter Conley.[2–7] Development of the coarse-to-fine principle of filtration took place in two major steps. The first step was the development of the dual-media filter, which typically uses 18–24 inches (0.46–0.61 m) of anthracite coal above 8–12 inches (0.20–0.30 m) of silica sand. This provides a two-layer filter in which the coarse upper layer of anthracite acts as a roughing filter to reduce the load of particulates applied to the sand below (Fig. 12–2b). Because of the different specific gravities of the two materials (anthracite 1.4, sand 2.65), the coarser anthracite media will remain above the finer sand media during backwashing. With applied turbidities of less than 5 NTU, dual-media filters can operate under steady-state conditions at 5 to 8 gpm/sq ft (12 to 20 m/h) with the production of high-quality water. Dual-media filters can retain more material removed from the water than a sand filter, but they have a lower resistance to turbidity breakthrough with changing flow rates. This shortcoming is due to the low total surface area of media particles, which is actually less than that for a rapid-sand bed. Dual-media beds in which there is controlled intermixing of the two materials near the interface perform better and wash more easily than dual-media beds designed for more distinct layering.[8]

In designing a dual-media bed, it is desirable to select anthracite as coarse as possible for particle removal to prevent surface blinding, and to have the sand as fine as possible to provide maximum particle removal. However, if the sand is too fine in relation to the anthracite, it will actually rise above the top of the anthracite and remain there when the filter is returned to service. For example, if 0.2 mm sand were placed below 1.0 mm anthracite, the materials would reverse during backwash, with the sand becoming the upper layer and the anthracite the bottom. Although the sand has a higher specific gravity, its small diameter in this case would result in its rising above the anthracite. The only way to enable very fine silica sand to be used in the bottom filter layer would be to use finer anthracite, but this would defeat the purpose of the upper filter layer because the fine anthracite would be susceptible to surface blinding. Experience has shown that it is not feasible to use silica sand smaller than about 0.4 mm because smaller sand would require anthracite small enough to result in unacceptably high headloss at flow rates above 3 gpm/sq ft (7 m/h).

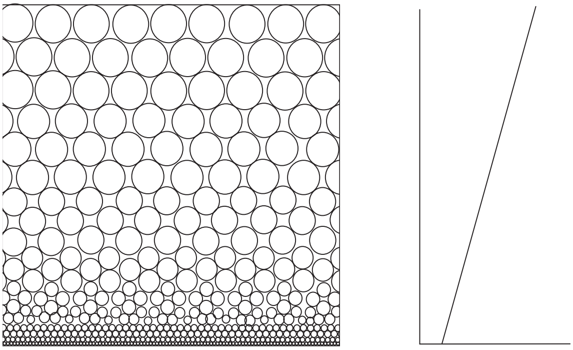
To overcome the above limitation and to achieve a filter that closely approached an ideal one (Fig. 12–2c), the mixed-media concept was developed. The problem of keeping a very fine medium at the bottom of the filter is overcome by using a third, very fine, heavy material (garnet, specific gravity of about 4.2, or ilmenite, specific gravity of about 4.5) beneath the anthracite and sand. The garnet (or ilmenite), sand, and anthracite particles are sized so that controlled intermixing of these materials occurs, and no discrete interface exists between them. This eliminates the stratification illustrated for the dual-media filter in Figure 12–2b, and results in a filter that very



(A)
Single-media bed distributes
coarse particles at the bottom



(B)
Dual-media bed provides
coarse coal above fine sand



(C)
An ideal filter passes water first
through coarse then fine media

Fig. 12–2. Graphical representation of various media designs closely approximates the ideal of a uniform decrease in pore space with increasing filter depth, as shown in Figure 12–2c.

The term ‘‘coarse-to-fine’’ refers more accurately to the pore space or size rather than to the media particles themselves, as is illustrated in Figure 12–2. By selecting the proper size distribution of each of the three media, it is possible to construct a bed that has an increasing number of particles at each successively deeper level in the filter. A typical mixed-media filter has a particle size gradation that decreases from about 1–2 mm at the top to about 0.15 mm at the bottom. The uniform decrease in pore space with filter depth allows the entire filter depth to be utilized for floc removal and storage. Figure 12–3 shows how particles of the different media are actually mixed throughout the bed. At all points in the bed there is some of each component, but the percentage of each changes with bed depth. There is steadily increasing efficiency of filtration in the direction of the flow. In some cases, anthracite of different densities can be used to further extend the coarse-to-fine concept. By using lighter-weight anthracite in addition to the normal anthracite-sand-garnet, it is possible to increase the top grain size to 2.4 mm without changing the grain size of the rest of the filter.[9]

Unfortunately, the literature is replete with comparisons of different filter media where inadequate attention was devoted to the proper sizing of the media. The use of three media rather than two can provide a superior filter *only* if the three media are properly sized. The reader must critically examine the literature to ascertain the basis for the design of the filter media being compared.

The total surface area of the grains in a mixed-media bed is greater than for a sand or dual-media bed, so that it is much more resistant to breakthrough and more tolerant

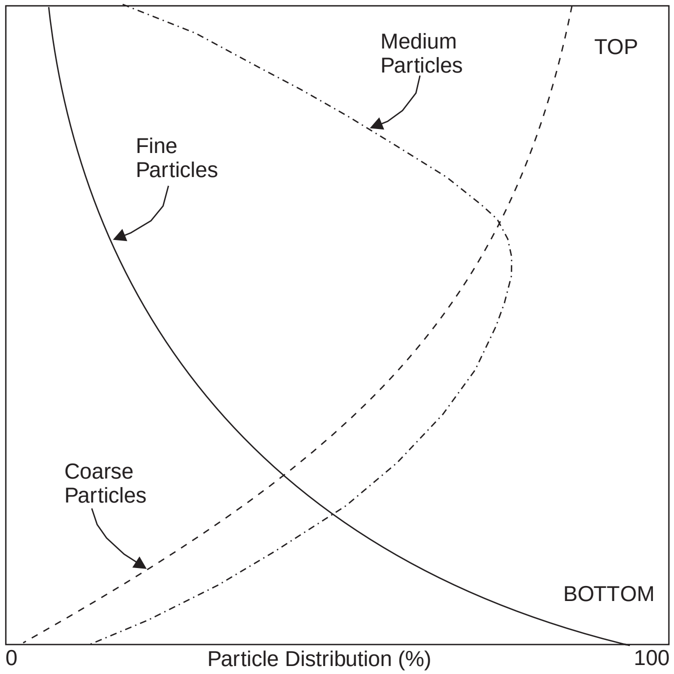


Fig. 12–3. Distribution of media in properly designed mixed-media filter

to surges in flow rates. This provides a great factor of safety in filter operation. Despite the greater total surface area of grains, the initial (clean filter) headloss in the two types is comparable. At 5 gpm/sq ft (12 m/h) throughput, the initial headloss in either a 0.50-mm sand or a mixed-media bed is about 1.5 feet (0.46 mm).

Filter rates of 5 gpm/sq ft (12 m/h) are commonly used for design and operation of dual and mixed-media beds, compared to 2 gpm/sq ft (5 m/h) for sand filters. At the same time, there is improvement in product water quality, which was the original purpose behind the development of the multimedia bed. Along with development and acceptance of the multimedia filter has come a recognition that the rate of filtration is only one factor (and a relatively unimportant one) affecting filter effluent quality. Chemical dosages for optimum filtration, rather than maximum settling, as well as other variables are much more important than filtration rate to production of good water quality.

In the modern concept of water treatment, coagulation and filtration are inseparable. Actually, they are each very closely related parts of the liquid–solids separation process. Because most water plants utilize sedimentation for a preliminary gross separation of settleable solids between coagulation and filtration, the crucial direct relationship of coagulation to optimum filterability often has been overlooked. Coagulant dose should be optimized not to provide the optimum settled water quality but to produce the optimal filtered water quality.

Deep Bed Mono Media Filter

There has been increasing interest in and use of deep (up to 8 feet) beds of relatively coarse (1.2–1.6 mm) monomedia (sand or anthracite) for direct filtration. The single-medium deep-bed filter differs from the conventional sand filter in two ways.[1] First, because the medium is coarser, a deeper bed is required to achieve comparable removal of particulates. Second, because excessive wash rates would be required to fluidize the coarse medium, it is washed without fluidization by the concurrent upflow of air and water. The air/water wash causes mixing of the medium, and little or no stratification by size occurs. Deep bed monomedia filters have been used for coagulated and settled waters. In these cases, a finer (0.9–1.0 mm) media is typically used than in direct filtration applications.

L/d Ratio

The ratio of bed depth (*L*, in millimeters) to the effective size of the media (*d*, in millimeters) has been used to characterize the filtering ability of various media designs. The concept is that equivalent L/d ratios will produce equal filtrate quality when filtering the same influent water at the same filtration rate. According to Kawamura,[56] the value of the L/d ratio should be $>$1,000 in rapid-sand filters and standard dual-media filter beds, $>$1,250 in regular multimedia filter beds, $>$1,300 in most coarse deep beds in which d is 1.2–1.4 mm and L is 1.8–2.0 m, and $>$1,500 in most coarse deep beds in which d is $\geq$1.5 mm.

When the diameter of the media exceeds 1.5 mm, spacing between the grains becomes large compared with the void space in regular filter beds. The void space triples when the diameter of the grain is doubled. Thus, the L/d ratio should be used as an estimate only when the media is $>$1.5-mm diameter.

If a filter aid is not used, then the above L/d ratios need to be increased by a minimum of 20 percent. Additional increases in L/d are not cost-effective because of the deeper filter structure, the higher initial headloss, and a longer filter washing time, which do not appreciably improve filter performance.

For dual and multimedia filters, the sum of the L/d for each layer is calculated (i.e., the weighted average L/d for the bed). For a dual-media filter with 2.0 ft (0.6 m) of anthracite with 1.0-mm ES over 1.0 ft (0.3 m) of sand with 0.5-mm ES, the total L/d would be $600/1.0$ plus $300/0.5 = 1200$.

Filter Classifications

There are several ways to classify filters. They can be described according to the direction of flow through the bed—that is, downflow, upflow, biflow, radial flow, horizontal flow, fine-to-coarse, or coarse-to-fine. They may be classed according to the type of filter media used, such as sand, anthracite, anthracite-sand, multilayered mixed-media, or diatomaceous earth. They may be classed by the number of media, such as monomedia, dual-media, or multimedia. Filters are also classed by flow rate. Slow-sand filters operate at rates of 0.05 to 0.13 gpm/sq ft (0.12 to 0.32 m/h), rapid-sand filters operate at rates of 1 to 2 gpm/sq ft (2.5 to 5 m/h), and high-rate filters operate at rates of 3 to 15 gpm/sq ft (7 to 40 m/h). Filters may also be classified by the type of system used to control the flow rate through the filter, such as constant rate, declining rate, constant level, equal loading, and constant pressure. Constant rate filtration is the most popular control system in the United States.

Another characteristic is pressure or gravity flow. Gravity filter units are usually built with an open top and constructed of concrete or steel, while pressure filters are ordinarily fabricated from steel in the form of a cylindrical tank. The available head for gravity flow usually is limited to about 8 to 12 feet (2.5 to 4.0 m), while it may be as high as 150 psi (1,000 kPA) for pressure filters. Because pressure filters have a closed top, it is not easy to inspect the filter media. Further, it is possible to disturb the media in a pressure filter by sudden changes in pressure. These two factors have tended to limit municipal applications of pressure filters to treatment of relatively unpolluted waters, such as the removal of hardness, iron, or manganese from well waters of good bacterial quality. The susceptibility to bed upset and the inability to see the media in pressure filters have been compensated for, to some extent, by the use of quick-opening manholes and by the use of recording turbidimeters and particle counters for continuous monitoring of the filter effluent quality. The introduction of a 3-inch (75-mm) layer of coarse (1 mm) high-density (specific gravity 4.2) garnet or ilmenite between the fine media and the gravel supporting bed has virtually eliminated the problem of gravel upsets, which is another of the concerns about the use of pressure filters for production of potable water.

WATER FILTER OPERATION

Mechanisms

Many publications describe filtration theory and mechanisms.[15–29] Several mechanisms are involved in particle removal by filtration, some of them physical and others chemical in nature. To fully explain the overall removal of impurities by filtration, the effects of both the physical and chemical actions occurring in a granular bed must be combined. Efficient filtration involves particle destabilization and particle transport similar to the mechanisms of coagulation; good coagulants are often also efficient filter aids.

The processes of coagulation and filtration are inseparable, and their interrelationships must be considered for best treatment results. One important advantage of filtration over coagulation, relative to the removal of very dilute concentrations of colloidal particles, is the much greater opportunity for contact afforded by the granular bed as compared to the number afforded by mixing the water. The removal efficiency of a filter bed is independent of the applied colloidal particle concentration, whereas the time of mixing depends upon colloidal particle concentration.

There are two basic approaches to achieving optimum filterability of water. One is to establish the dosage of primary coagulant needed for maximum filterability rather than for production of the most rapid-settling floc. Another approach is to add a second coagulant as a filter aid to improve the filterability of the settled water as it enters the filter. Filter aid must be added continuously (to coat the filter media rather than precoat). Excessive dosages of filter aids cause filters to clog too rapidly.

For effective filtration, the objective of pretreatment should be to produce small, dense floc, so that the particles are small enough to penetrate the bed surface and migrate partially down into the filter bed. Removal of floc within a bed is accomplished primarily by contact of the floc particles with the surface of the grains or previously deposited floc, and adherence thereto. Contact is brought about principally by the convergence of flow at contractions in the pore channels between the grains. Of minor importance are the flocculation, sedimentation, and entrapment of particles that occur within the pores of the bed.

Adsorption of suspended particles on the surface of the filter grains is also an important factor in filter performance. Physical factors affecting adsorption are the nature of the filter and the suspension. Adsorption is a function of the filter grain size, floc particle size, and the adhesive characteristics and shearing strength of the floc. Chemical factors affecting adsorption include the chemical characteristics of the suspended particles, the aqueous suspension medium, and the filter medium. Two of the most important chemical characteristics are the electrochemical forces and van der Waals forces (molecular cohesive forces between particles).

Filtration Efficiency

Filters are highly efficient in removing suspended and colloidal materials from water. Many impurities can be removed by the combination of proper coagulation and filtration. Among the many are: turbidity, bacteria, algae, viruses, protozoa color, oxidized iron and manganese, radioactive particles, chemicals added in pretreatment, and heavy metals. Because filtration is both a physical and a chemical process, there are a large number of variables that influence filter efficiency. These variables exist both in the water applied to the filter and in the filter itself. Knowledge of the factors affecting filter efficiency has increased quite rapidly since the 1960s. Use of this information in the design and control of filters makes possible the water quality needed to meet the continually more stringent filter performance requirements of federal and state regulations.

Filter efficiency is affected by several properties of the applied water: temperature, filterability, and the size, nature, concentration, and adhesive qualities of suspended and colloidal particles. Cold water is notably more difficult to filter than warm water, but for most applications there is no control over water temperature. Filterability, which is related to the size and surface chemistry of the suspended and colloidal impurities in the water, is the most important property. One practical way to measure filterability is to operate a pilot filter in parallel with the plant filters. The pilot filter receives raw water with independent control of treatment chemicals. By monitoring pilot filter effluent turbidities and making appropriate adjustments in chemical treatment, optimum filterability of the water can be obtained. The use of raw water in the pilot filter provides the necessary lead time to anticipate plant requirements, particularly on raw-water supplies that experience rapid changes in water quality. Maximum filterability is much more important to production of a water of maximum clarity (minimum turbidity) than is maximum turbidity reduction prior to filtration.

Some properties of the filter bed that affect filtration efficiency are: the size and shape of the grains, the porosity of the bed (or the hydraulic radius of the pore space), the arrangement of grains (whether from fine-to-coarse or coarse-to-fine), the depth of the bed, and the headloss through the bed. In general, filter efficiency increases with smaller grain size, lower porosity, and greater bed depth. Coarse-to-fine filters contain much more storage space for materials removed from the water than do fine-to-coarse filters, and permit the practical use of much finer materials in the bottom of the bed than can be tolerated at the top of a fine-to-coarse filter.

The total surface area of filter media grains is important because it represents the total area available for adsorption of floc. Figure 12–4 summarizes the total surface area of different sizes and depths of filter media.[14] Because of the greater total surface area of the grains, smaller grain size, and lower porosity, the coarse-to-fine filter is more efficient than the fine-to-coarse filter. The much greater total grain surface area and the smaller grain size provided by mixed media as compared to dual media account for the greater resistance to breakthrough provided by mixed media. A mixed-media

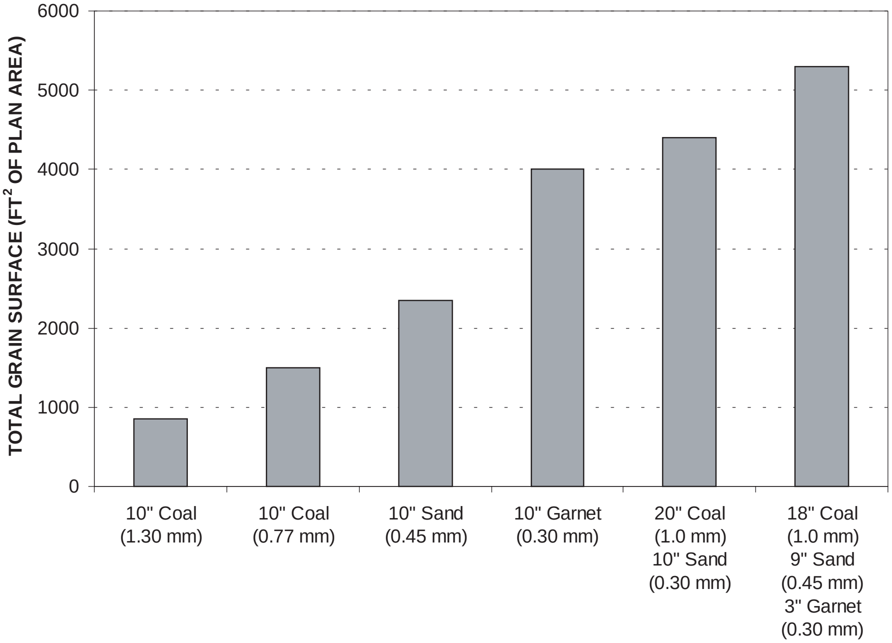


Fig. 12–4. Filter grain surface of different filter media sizes and depths (*Source:* Reference 14)

filter also may permit achievement of lower turbidity at a given coagulant dosage, as shown in Figure 12–5. A dual-media filter is less resistant to breakthrough than rapid-sand filters, whereas a mixed-media filter can be more resistant than either.

Hydraulic throughput rate also affects filter efficiency. However, the rate is not nearly so significant on effluent quality as are other variables. In general, the lower the rate, the higher the efficiency. All other conditions being equal, a filter may produce a slightly better effluent when operating at a rate of 1 gpm/sq ft (2.44 m/h) than when operating at 8 gpm/sq ft (19.5 m/h). However, it is also true that a given filter may operate satisfactorily at 8 gpm/sq ft (19.5 m/h) on a properly prepared water, yet fail to produce a satisfactory effluent at 1 gpm/sq ft (2.44 m/h) when receiving an improperly pretreated water. With good filter design and proper coagulation, the optimum throughput rate is a matter of economics rather than a question of safety.

The efficiency of filters in bacterial removal varies with the applied loading of bacteria but with proper pretreatment should exceed 99 percent. However, bacterial removal by filtration should never be assumed to reach 100 percent. The water must be chlorinated for satisfactory disinfection. Coagulation, flocculation, and filtration will remove more than 98 percent of polio virus at filtration rates of 2 to 6 gpm/sq ft (2.5 to 1.5 m/h), but complete removal is dependent upon proper disinfection.[10]

A properly operating filter should result in an effluent turbidity of less than 0.1 NTU. With proper pretreatment, filtered water should be essentially free of color, iron, and manganese. Large microorganisms, including algae and diatoms, and amoebic cysts, are readily removed by filtration from properly pretreated water. *Giardia* and *Cryptosporidium* can also be effectively removed by the combination of proper coagulation and effective filtration (0.1–0.2 NTU).

The use of sensitive, accurate, and reliable turbidimeters for the continuous monitoring of filter effluent quality enhances the degree of control that can be exercised

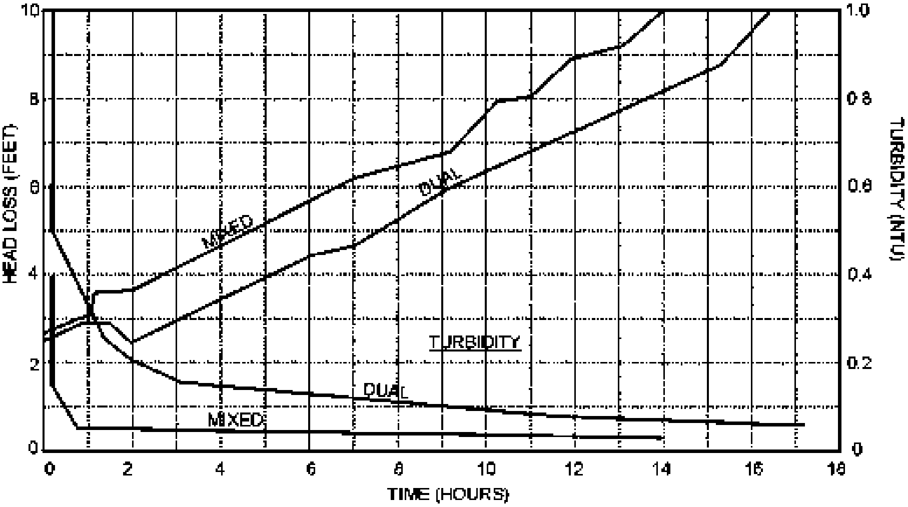


Fig. 12–5. Example of operating characteristic differences between mixed-media and dual-media filters

over filter performance. The turbidity of filter effluent is instantly and continuously determined, reported, and recorded. Turbidimeters are best used on individual filters but can be of value to monitor the performance of a group of filters. When used on individual filters, there is less opportunity for breakthrough in one filter to be masked by the rest of the filters. The signal obtained from the instrument can be used to sound alarms, or, if necessary, to shut down an improperly operating filter unit. This greatly increases the reliability with which filters of all types may be operated, and may broaden the range of safe application for pressure filters. The significance of filter effluent turbidity is reflected in the observation that a turbidity increase of 0.1 NTU was associated with an increase in amoebic cysts by a factor of 10 to 50 when the filter had been operating at an equilibrium condition.[11]

Turbidity standards, while more uniformly adopted than standards for particle counters, have become increasingly stringent. The 1986 SDWA amendments established 0.5 NTU as the benchmark for acceptable filter performance. Pending changes to the regulations will lower the turbidity limit to 0.3 NTU, with the limit expected to eventually be lowered to 0.1 NTU. At these lower turbidities, the accuracy of turbidimeters is challenged, which has drawn renewed interest in particle counters for use in measuring filter performance.

Particle counters can measure the number of particles in selected size ranges (e.g., 2–10 μm, 10–20 μm, etc.), enabling the operator to focus on particle size ranges of particular concern for the filters. In some cases, the particle counter can provide indication of a pending increase in filter effluent turbidity, as the initial filter breakthrough will often occur with the smaller particles. Figure 12–6 shows an example of how filter effluent particle count increases can often proceed increases in turbidity.

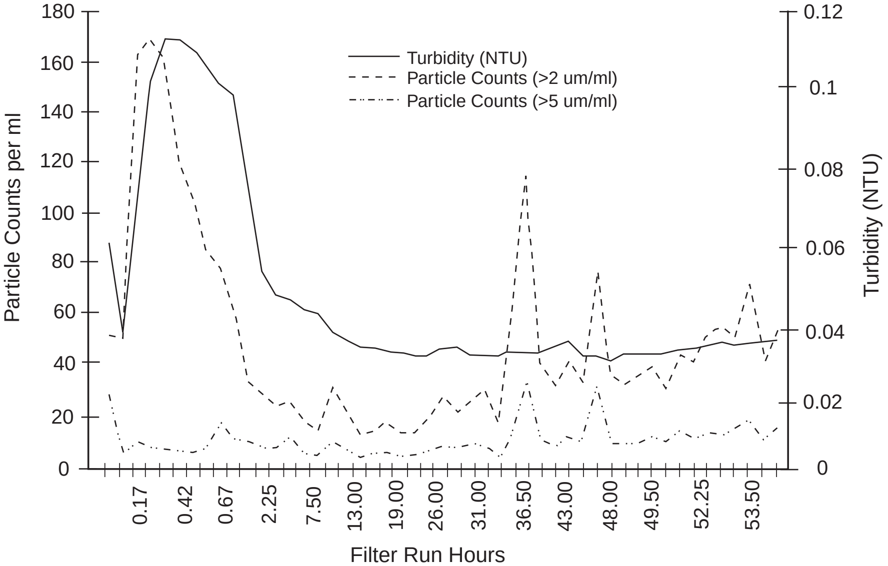


Fig. 12–6. Example of differences in filter effluent particle counts and turbidity during a typical filter run

Standards have not been established yet for acceptable particle counts in filter effluents; however, particle counts in the range of 20–50 per mL for particles larger then 2 μm are considered indicative of a well-performing filter.

Rating Filter Performance

There are many perspectives from which to judge filter performance: throughput rate, quantity of backwash water, length of filter runs, net water production, and finished water quality. When pilot tests are conducted to compare alternative media designs, the amount of data collected during the many runs of each medium can be overwhelming. The amount of water filtered per square foot per filter run has been suggested as a useful parameter for evaluation filter production efficiency.[12] The net water production per run can be readily calculated by subtracting the amount of backwash water required. However, rating filter performance solely on volume of water processed does not reflect the quality of water produced. One approach is the use of the filter performance index (FPI) to express the relative performance of two or more filters operating under similar, but not necessarily identical, conditions:

$$\text{FPI} = \frac{(\text{NTU}_a - \text{NTU}_e)\text{G}}{\text{NTU}_e} \tag{12–1}$$

where:

FPI = filter performance index
NTU_a = turbidity in units applied to filter
NTU_e = turbidity in units in filter effluent
G = gallons filtered between backwashes

Higher FPI numbers indicate better performance than low FPI numbers, with the degree of superiority represented by the percent difference in any two values.

The utility of the FPI is illustrated by the results of an extensive pilot plant test program in Sacramento, California.[13] A total of about 360 individual runs were made, consisting of 90 sets of parallel runs of a mixed-media filter, a sand filter, a dual-media filter, and an anthracite-capped sand filter. The curves and tabulations of all the filter runs made a formidably thick volume. However, the overall comparative performance can be readily grasped from the following averages:

Media Type	Average FPI
Mixed media	74,000
Dual media	58,000
Anthracite capped sand	45,000
Sand	25,000

During the course of the study, it was observed that the mixed-media filters recovered more quickly following backwash than did the other types of media. That is, only a few minutes of operation were required to produce effluent turbidities within the goal of 0.10 NTU with the mixed-media bed. With the other filters, periods of several hours were required for turbidities to reach the goal. This was increasingly apparent at high filter rates.

Filter Operational Parameters

The filtration system must be capable of meeting several general operating parameters. These general operating parameters must be compatible with the treatment processes both upstream and downstream of the filters. The general requirements must address a number of issues, including: number of filter units, length of filter runs, out-of-service limitations, hydraulic limitations, and flow control requirements.

Length of Filter Runs The required length of filter run before backwashing depends upon a number of variables, including: surface loading rate, water temperature, depth of solids penetration, media distribution, and pretreatment provided. In general, the normal filter run lengths should be long enough to avoid hampering overall plant operations due to frequent filter cleaning or excessive use of plant water production for cleaning operations. The filter run lengths should also not be so long as to permit biological growth in the filter or interfere with routine operating shifts at the plant. For rapid rate gravity filters, run lengths of between 12 and 72 hours are not unusual, with run lengths of 24 to 48 hours considered typical.

Out-of-Service Limitations Because of the nature of filter operations and the potential for equipment malfunctions, some allowances should be provided for taking filter units out of service for maintenance. These provisions are in addition to those provided to take the filters off-line for routine cleaning. The capability to remove individual filter units from service for extended periods of time must be provided, without requiring unusual measures to operate the remaining filters still in service. Filter loading rates may need to be increased above the established design parameters, but must be capable of hydraulically handling the increased flow. The reduced filter run lengths and increased frequency of backwash should not overburden the filtration system or waste-handling system at the plant. Some design standards require that the filter system be capable of meeting the plant design capacity at the approved filtration rate with one unit out of service. In large plants, with a large number of filters, it is often a common practice to establish the filtration requirements (acceptable filtration rate) with two filters assumed out of service.

Hydraulic Limitations The flow received by the filtration system at most plants will normally be at atmospheric pressure. If gravity filters are employed, the downstream plant hydraulics will be set to provide sufficient head through the filters without intermediate pumping. Should pressure filters be utilized, booster pumps would take suction from the effluent of the pretreatment system, raising the hydraulic grade line to account for the high headlosses through the pressure filters. The downstream hydraulics of the plant can be set independent of the hydraulics upstream of the pressure filters, making plant layout more flexible. However, for rapid rate gravity filters approximately 10–12 feet of head should be provided for in the plant hydraulics for headloss through the filters. This headloss would provide sufficient allowance for routine operation of gravity filters.

Flow Control Requirements The filter system must automatically adjust to changes in plant flow, maintaining stable conditions within the filter units. Intermediate storage or bypass of the flow is often not normally required. If intermediate storage is utilized, it is usually provided by employing submerged orifice effluent troughs in lieu of overflow weir troughs on the basins preceding the filters. The submerged orifice troughs permit the buildup of water in the basin to avoid temporary changes in filtration rates should a filter be removed from service for a short period.

Another approach to maintaining even flows through the filters during short-term periods when a filter(s) is removed from service for cleaning is to bring previously cleaned filters back on-line at the same time a dirty filter is removed from service. This requires that a filter that has been cleaned sit idle until another filter needs to be cleaned, but it allows maintenance of a steady flow through the filters already on-line. In the event the idle filter is returned to service prior to removing another filter from service, the automatic filter flow controls would need to evenly distribute the change in flow among the active filters.

DESIGN OF FILTER SYSTEMS

Selection of Type, Size, and Number of Units

In designing new filters or rehabilitating existing filters, there are a number of factors to consider. The type, layout, number, and size of the filter units are key elements of the basis of design. For rapid rate gravity filters, a number of factors must be addressed in developing the layout for the filters, including: overall configuration, number of units, unit sizes, and filter depth.

Configuration A number of factors will affect the overall filter configuration. The major factors are: depth of the filter box, backwash method, and method of filter control. Other factors also can impact configuration of the filters (i.e., site constraints, materials of construction). Some design standards also contain several requirements that must be considered in the filter configurations, including provisions for:

- Vertical walls within the filter
- No protrusion of the filter walls into the filter media
- Cover by superstructure as determined necessary for local climate.
- Head room to permit normal inspection and operations.
- Minimum depth of filter box of 8.5 feet.
- Minimum water depth over the surface of the filter media of 3 feet
- Trapped effluent to prevent backflow of air to the bottom of the filters
- Construction to prevent cross-connections and common walls between potable and nonpotable water

One common arrangement for gravity filters is to place the filter units side by side in two rows on opposite sides of a central pipe gallery (see Fig. 12–7). Another arrangement sometimes used is to place the filters in a row opposite the upstream treatment basins, separated by a pipe gallery. These two arrangements are considered the more typical approaches to filter layout.

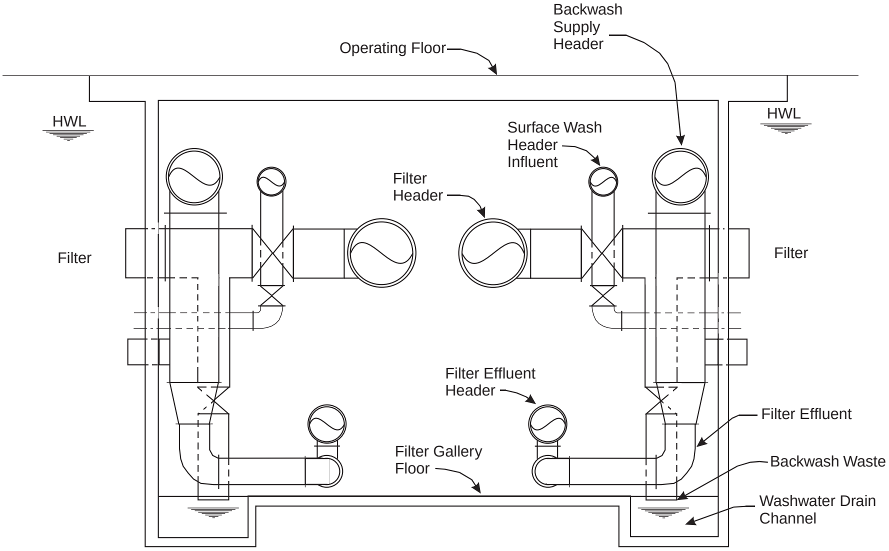


Fig. 12–7. Sample filter pipe gallery cross section for gravity filters

One end of the rows of filters should be unobstructed to allow construction of additional units in future expansion. The best pipe gallery design includes a daylight entrance to provide good lighting, ventilation, and drainage, and improves access for operation, maintenance, and repair.

Some other layouts are possible using less conventional schemes. The valveless filter, often referred to as the ‘‘Greenleaf’’ filter, utilizes clusters of four filters centered around a middle control chamber. Multiple clusters of the filters can be used in a variety of arrangements. This type of filter utilizes the effluent from three filters to backwash the fourth filter, employing the head available in the three producing filters. The primary advantage of the valveless filter concept is the elimination of the complex piping and valving associated with the more traditional gravity filter layouts. However, if a filter must be removed from the service for maintenance, operation of the remaining three filters in the cluster is compromised, as sufficient flow for proper backwashing cannot be provided without auxiliary pumping. In addition, the terminal headloss is limited to 3–4 feet by the effluent weir, and flow splitting between filters is inaccurate due to the use of inlet weirs to control flow to individual filters. Finally, the valveless filter concept does not permit the use of a center gullet to divide the filters into two cells, essentially doubling the number of filters required when compared to the traditional gravity filter layouts.

Filters should be located as close as possible to the source of influent water, the backwash water supply, the filtered water storage reservoir, and the control room. On sites that are subject to flooding, the level of the bottom of the filter boxes should be located above the maximum flood level. This arrangement permits the discharge of filter backwash water during flood periods and avoids possible contamination of the filtered water.

Filter Number/Sizing Multiple filter units must be provided with sufficient number to permit removing the largest unit from service without compromising the design criteria established for the filters. The number of filter units required will depend upon the type of filter technology employed. For example, self-backwashing filters (such as the Greenleaf filter) should have at least four filters in even small plants to be able to produce the volume of backwash water needed at low flows. Each filter type has inherent practical limitations on the size and capacity of each unit. Some design standards have established the minimum number of filter requirements that varies with the type of filter technology employed. In general, these regulations require a minimum of two filter units. For plants of 10–30-mgd capacity, a minimum of four filters is recommended.[56]

Large filters are normally provided with a center gullet to permit backwashing the filter half at a time, using common inlet and outlet piping. This approach permits reducing the number of filters and the overall filter system complexity. When using a central gullet, normal practice is to limit the total filter size to approximately 2,000 square feet to maintain backwash flow rates and filter piping/valve sizes within reasonable limits. This maximum size is also due to difficulties in providing uniform distribution of backwash water over large areas, reduction in filter capacity with one unit out of service for backwashing, and structural design considerations.

To filter a given quantity of water, the capital cost of piping, valves, controls, and filter structures is usually less for a minimum number of large filter units as compared to a greater number of smaller units. In expanding existing plants, it may be better not to increase the size of filter units but rather to match the existing size in order to avoid the need to increase the capacity of wash water supply and disposal facilities.

Pressure filters may be either horizontal or vertical. Horizontal filters offer much larger filter areas per unit and would normally be used when plant capacities exceed 1 to 15 mgd (4 to 60 Ml/d). Figure 12–8 shows a typical horizontal pressure filter vessel. Although an 8-foot (2.44-m) pressure filter vessel is shown, 10-foot (0.31 m) diameters are also commonly used with lengths up to 60 feet (20 m). Figure 12–9 shows a typical vertical filter. Diameters up to 11 feet (3.35 m) are commonly used for vertical filters, with working pressures up to 150 psig (1,000 kPa).

Filter Depth Traditionally, the overall depth of filters from water surface to underdrains has been 8 to 10 feet. Design standards also require a minimum filter box depth of 8½ feet, with a minimum water depth of 3 feet over the filter media. The trend toward deeper filters has a number of advantages over the traditional filter design:

- Increases submergence above media to prevent air binding
- Increases depth to accommodate coarse monomedia
- Provides for higher headlosses on high-rate filters to ensure adequate filter run time

Normally, 10–12 feet of headloss is provided for between the water surface above the filter media and the free water level in the filter effluent conduit. The depth of the filter box does not need to account for the total headloss, but should allow a minimum of 5–8 feet for buildup of headloss during filter runs. As much as 3 feet of headloss may be required to account for losses through the underdrain orifices, underdrains, effluent piping, and valves.

The filter depth should enable the plant operators to take full advantage of the filter system's inherent capabilities. As a result, an increase in the water depth over the filter

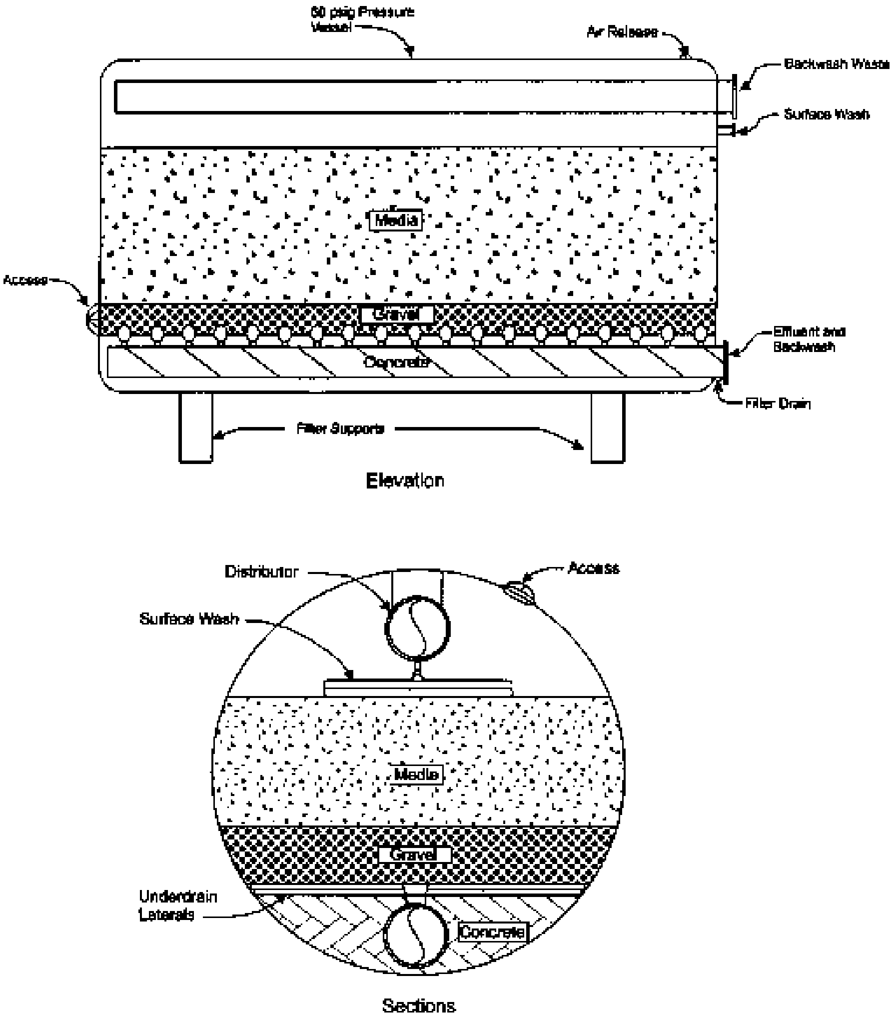


Fig. 12–8. Typical pressure filter (Courtesy of USFilter/Memco)

media is often used. The increase in water depth will provide more available head to lengthen filter runs and enable flexibility to change to alternate filter media in the future should it become necessary. A minimum water depth of 6 feet above the surface of the filter media should be provided, with 2–3 feet of freeboard allowed for in the filter design. The overall depth of the filter box will be affected by the type of underdrain and depth of filter media that are used, so the depth of filters cannot be finalized until those elements are selected.

The use of common walls between filtered and unfiltered water should be avoided so as to eliminate the possibility of contamination of the finished water. For the same reason, it is better not to construct clearwell storage beneath the pipe gallery floor. Many states do not permit these types of construction.

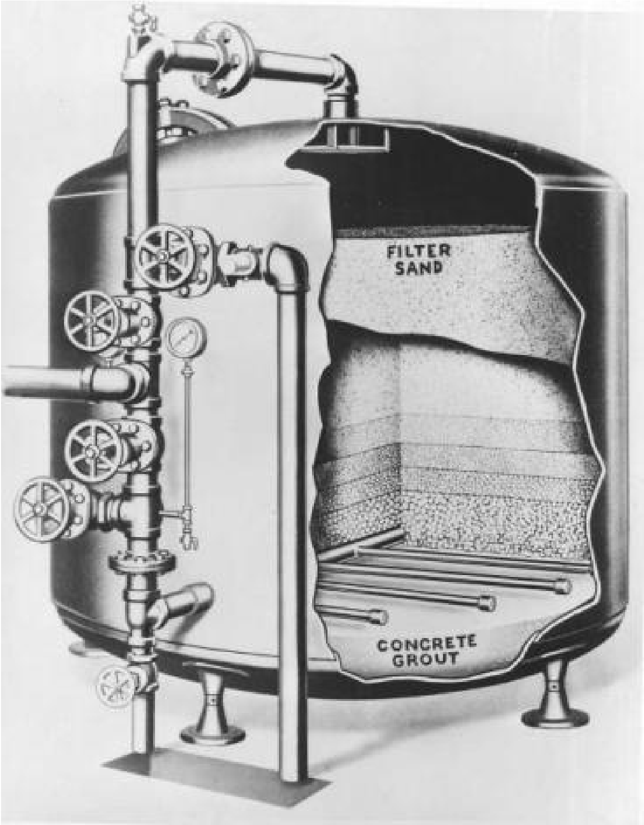


Fig. 12–9. Typical vertical pressure filter with concrete grout fill in the bottom head, pipe headers, lateral underdrains, gravel supporting bed, and filter sand (Courtesy of Infilco Degremont, Inc.)

Filter Underdrains

Filter underdrains have a twofold purpose. The more important is to allow uniform distribution of backwash water without disturbing or upsetting the filter media above. The other is to collect the filtered water uniformly over the area of the bed. There are a wide variety of filter underdrain types available for use in both gravity and pressure filters. All of the systems accomplish the uniform distribution of wash water by introducing a controlling loss of head, usually about 3 to 15 feet (1 to 5 m), in the orifices of the underdrain system. The orifice loss must exceed the sum of the minor (manifold and lateral) headlosses in the underdrain to provide good backwash flow distribution.

The underdrain systems differ basically with respect to the type of filter washing system and whether a gravel layer between the media and underdrain is employed. The filter underdrains are needed to perform a variety of functions, including:

- Supporting and retaining the filter media
- Distribution of backwash supply water and air scour
- Collection of flow through the filter bed

The underdrain design must be capable of uniformly distributing backwash water and air scour across the filter bed and uniformly collecting filtered water over the entire area of the filter. There are three categories of filter underdrain systems available:

- False filter bottom
- Floor-mounted filter bottom
- Pipe lateral

False Filter Bottoms The false filter bottom underdrain system utilizes a false floor within the filter chamber to mount the underdrain system. A plenum is provided under the false bottom to transport water and air to and from the underdrain. The false bottom is normally a precast or cast-in-place concrete system supported by pedestals off the filter chamber floor. Less commonly used are false bottoms constructed of metal plate. The underdrain associated with the false bottoms can take a variety of forms:

- Strainer type
- Inverted pyramidal depression with balls (Wheeler bottoms)
- Porous plate

The strainer-type system employs a number of different strainers, often referred to as *nozzles*, constructed of plastic or stainless steel. The strainers contain orifices to equalize backwash flow over the filter bed and drop pipes to control the inlet of air through the nozzle. The porous plate–type underdrains are made of fused silicas or aluminum oxide particles. They are formed into plates mounted on piers and allow flow to pass through the entire plate structure. The Wheeler bottom is built with interconnected modules, each module containing four inverted pyramidal spaces. Porcelain balls are positioned in the pyramid space to diffuse flow into the gravel. A porcelain thimble at the apex of each pyramid admits water to the plenum beneath the modules. The strainer type and porous plate type of underdrain does not require an overlying gravel layer, whereas the Wheeler bottom requires the gravel to maintain the filter media in place. The strainer-type and porous plate–type underdrains utilize fine openings to prevent the migration of media into the plenum. Normally, strainer-type systems are used for combined air/water backwash systems, while the porous plate and Wheeler bottom underdrains are used for backwash systems not utilizing air scour.

Floor-mounted Filter Bottoms This type of underdrain system can take the form of one of several configurations. The most popular is filter blocks composed of vitrified clay or plastic blocks, mounted directly on the floor of the filter chamber. Originally developed using vitrified clay, the plastic blocks have become more popular in recent years, relegating the clay blocks to retrofit applications. The top surface of the blocks contains closely spaced orifices, with internal orifices to regulate flow. A channel in the floor below the underdrains directs flow to and from the rows of blocks, and connects to the filter gallery piping. The water backwash blocks are designed with dual manifold configuration, with water entering the lower conduit and flowing into the upper conduit along the entire length of the block manifold. The combination air/water backwash blocks utilize triangular passages. As an alternative, filter blocks are available that utilize strainer nozzles mounted in the top of the block or a porous plate, which eliminates the need for support gravel.

Other floor-mounted underdrains are available that distribute flow to and from a common channel below the underdrains, using a variety of shapes and materials for the rows of underdrains. Most of these other underdrains eliminate the need for support gravel.

Pipe Laterals The pipe lateral underdrain uses a main header pipe embedded in gravel in each filter chamber with several laterals in both sides of a center header. The laterals contain perforations on the underside to dissipate flow during backwash against the filter floor and surrounding gravel. The pipe laterals are typically constructed of steel or PVC. The pipe laterals can also be configured using strainer nozzles spaced at intervals along the top of each pipe lateral or using slotted pipes similar to well screens.

Usually, orifice diameters are ¼ to ½ inch (6.4 to 12.8 mm) with spacings of 3 to 8 inches (75 to 200 mm). Fair and Geyer give the following guides to pipe lateral underdrain design:[19]

- Ratio of area of orifice to area of bed served, 0.15 to 0.001
- Ratio of area of lateral to area of orifices served, 2:1 to 4:1
- Ratio of area of main to area of laterals served, 1.5:1 to 3:1
- Diameter of orifices, ¼ to ¾ inch (6 to 20 mm)
- Spacing of orifices, 3 to 12 inches (75 to 300 mm) on centers
- Spacing of laterals, about the same as spacing of orifices

Comparison of Underdrain Types The most commonly used filter underdrain system has traditionally been a block filter bottom used in conjunction with a gravel layer. However, recent renewed interest in air/water backwash has increased the use of strainer-type false filter bottoms and block bottoms that incorporate a porous-plate top to eliminate the need for gravel. In order to determine the most effective underdrain system, a comparison of the advantages and disadvantages for each filter underdrain type suitable for use with air/water backwash is beneficial.

Strainer-Type False Bottom Type

Advantages

- Does not require a gravel layer.
- Plenum allows access to clean out underdrain.

Disadvantages

- Strainer orifices can become plugged.
- Rupture of the false floor is possible.
- Plastic nozzles are fragile and can be easily broken.
- Dead areas can occur between the nozzles.

Floor-Mounted Type

Advantages

- Underdrains are simple to install in filter chamber.
- Underdrain is not susceptible to rupture problems.

Disadvantages

- Requires use of gravel layer above the blocks in some instances.
- Disruption of the gravel layer with air is possible.
- Underdrain interior is not accessible for cleaning.

Pipe Lateral Type

Advantages

- Low installation cost.
- Not susceptible to rupture problems.

Disadvantages

- Experiences relatively high headlosses.
- Flow distribution is not as effective.
- Vertical space required for gravel bed

Filter Gravel

A graded gravel layer, usually 12 to 18 inches (0.31 to 0.46 m) deep, is placed over the pipe underdrain system to prevent the filter media from entering the underdrain orifices and to aid in distribution of the backwash flow. Table 12–1 shows typical gravel bed design criteria for a pipe lateral underdrain.

A weakness of the gravel support system has been the tendency for the gravel eventually to intermix with the filter media. Such gravel ‘‘upsets’’ are caused by localized high velocity during backwash, introduction of air into the backwash system,

TABLE 12–1. Typical Gravel Bed for Pipe Underdrain System

	Number of Layers			
Description	Bottom	2	3	Top*
Depth of layer, inches (mm)	1	3 (76)	3 (76)	4 (122)
Square mesh screen opening, inches				
Passing		¾	½	¼
Retained	¾	½	¼	⅛

Source: Reference 46. (Reprinted from *Water Treatment Plant Design*, 3d ed., by permission. Copyright © 1998, American Water Works Association.)

*Plus coarse garnet.

†Bottom layer should extend to a point 4 inches (100 mm) above the highest outlet of wash water.

or use of excessive backwash flow rates. The gravel layer can be stabilized by using 3 inches (75 mm) of garnet or ilmenite as the top layer of the gravel bed. This coarse, very heavy material will not fluidize during backwash and provides excellent stabilization for the gravel. It also prevents the fine garnet or ilmenite used in a mixed-media filter from mixing with the gravel support bed.

Gravel layers are used with several of the commercially available underdrain systems, such as the traditional block filter bottoms (see Fig. 12–10) and Wheeler bottoms. The gravel serves to prevent the migration of the filter media downward to the underdrains, where the underdrain can become plugged or the filter media can be lost. Gravel depths and gradations vary for these underdrain systems. For example, one manufacturer recommends the gradation listed in Table 12–2.

Gravel should be hard and rounded with an average specific gravity of not less than 2.5. Not more than 1 percent by weight of the material should have a specific gravity of 2.25 or less. Not more than 2 percent by weight of the gravel should consist of thin, flat, or elongated pieces (pieces in which the largest dimension exceeds five times the smallest dimension). The gravel should be free from shale, mica, clay, sand, loam, and organic impurities of any kind. The porosity of gravel in any layer should not be less than 35 percent or more than 45 percent. Gravel should be screened to proper size and uniformly graded within each layer. Not more than 8 percent by weight of any layer should be coarser or finer than the specified limit.

Filter tanks must be thoroughly cleaned before gravel is placed, and kept clean throughout the placing operation. Gravel made dirty in any way should be removed and replaced with clean gravel. The bottom layer should be carefully placed by hand to avoid movement of the underdrain system and to ensure free passage of water from the orifices. Each gravel layer should be completed before the next layer above is started. Workers should not stand or walk directly on material less than ½ inch in diameter, but rather should place boards to be used as walkways. If different layers of gravel are inadvertently mixed, the mixed gravel must be removed and replaced with new material. The top of each layer should be made perfectly level by matching to a water surface at the proper level in the filter box.

Coarse Garnet

A 3-inch (75-mm) layer of high-density gravel (garnet or ilmenite) can be used between the gravel bed and the fine media. This coarse, dense layer prevents disruption

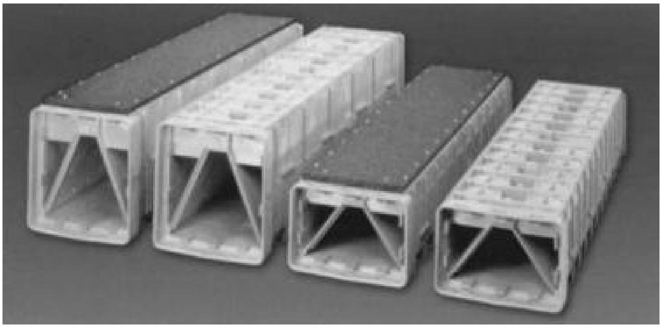


Fig. 12–10. Leopold Type S Technology Underdrain can be used for air-scour backwash with a porous-plate or gravel underdrain (Courtesy of The F. B. Leopold Company Inc.)

TABLE 12–2. Gravel Size and Layer Thickness for Use with Block Bottoms

	Layer Thickness		Size Limit	
Gravel Layer	Inches	(mm)	Inches	(mm)
Bottom	2	50	¾ × ½	20 × 12.7
Second	2	50	½ × ¼	12.7 × 6.4
Top*	2	50	¼ × ⅛	6.4 × 3.2

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*Plus coarse garnet.

of the gravel. The specific gravity of the material should not be less than 4.2. The garnet or ilmenite particles in the bottom 1½ inch (38 mm) layer should be 3⁄16 inch (4.8 mm) .4 by 10 mesh (2 to 5 mm) silica gravel could be added as the top layer to the gravel layers indicated in Tables 12–1 and 12–2. Otherwise, there may be migration of fine media down into the gravel supporting bed.

Mixed Media

There is no one mixed-media design that will be optimum for all water filtration problems. Conley and Hsiung have presented techniques designed to optimize the media selection for any given filtration application.[7] Their work clearly indicates the marked effects that the quantity and quality of floc to be removed can have on media selection, as shown in Table 12–3. Pilot tests of various media designs can be more than justified by improved plant performance in most cases.

Certainly, use of mixed media does not in itself ensure superior performance, as illustrated by the experiences of Oakley and Cripps.[47] Using the anthracite, sand, and garnet materials readily available to them resulted in a bed with no significant advantages over other filter types. Oakley reports the mixed-media bed was made up of 8 inches (203 mm) of 0.7 to 0.8 mm garnet, 8 inches (203 mm) of 1.2 to 1.4 mm sand, and 8 inches (203 mm) of 1.4 to 2.4 mm anthracite.[48] The authors' experience

TABLE 12–3. Illustrations of Varying Mixed-Media Design for Various Types of Floc Removal

	Garnet		Silica Sand		Anthracite	
Type of Application	Size	Depth, Inches (mm)	Size	Depth, Inches (mm)	Size	Depth, Inches (mm)
Very heavy loading of fragile floc	−40 + 80*	8 (200)	−20 + 40	12 (300)	−10 + 20	22 (560)
Moderate loading of very strong floc	−20 + 40	3 (75)	−10 + 20	12 (300)	−10 + 16	15 (380)
Moderate loading of fragile floc	−20 + 80	3 (75)	−20 + 40	9 (225)	−10 + 20	8 (200)

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*−40 + 80 = passing No. 40 and retained on No. 80 U.S. sieves.

indicates that this bed was too shallow and too coarse, and that a better media selection for the particular application would have been 3 inches (76 mm) of 0.4 to 0.8 mm garnet, 9 inches (229 mm) of 0.6 to 0.8 mm sand, and 24 inches (610 mm) of 1 to 2 mm anthracite.

A key factor in constructing a satisfactory mixed-media bed is careful control of the size distribution of each component medium. Rarely is the size distribution of commercially available materials adequate for construction of a good mixed-media filter. A common problem is failure to remove excessive amounts of fine materials when the media is installed. These fines can be removed by placing the media in the filter, backwashing it, draining the filter, and skimming the upper surface. The procedure is repeated until field sieve analyses indicate that an adequate particle size distribution has been obtained. The second medium is then added and the procedure repeated. The third medium is then added and the entire procedure repeated. Sometimes, 10 to 20 percent of the materials may have to be skimmed and discarded to achieve the proper particle-size distribution.

Dual Media

As compared to mixed-media, the dual-media (anthracite-sand) filter has lower headlosses through the media and less resistance to breakthrough because it is made up of coarser particles and has less total surface area of particles. The mixed-media filter is capable of producing lower finished water turbidities than the dual-media. These differences are greater and become more pronounced when the difficulty of the filtration application increases. In polishing highly pretreated waters, the differences are not so great, and some designers prefer to use anthracite-sand media.

Typically, anthracite-sand filters consist of a coarse layer of anthracite about 18–24 inches (500–700 mm) deep above a fine layer of sand about 8–12 inches (200–300 mm) thick. Some mixing of anthracite and sand at their interface is desirable to avoid excessive accumulation of floc, which occurs at this point in beds graded to produce well-defined layers of sand and anthracite. Such intermixing reduces the void size in the lower portion of the anthracite layer, forcing it to remove floc that otherwise might pass through the anthracite layer. Typical gradations of sand and anthracite for use in dual-media filters are given in Table 12–4.

“Capping” Sand Filters with Anthracite

One inexpensive method to improve rapid sand filter performance is to remove several (4–8) inches of sand from a bed and replace it with an equal depth of anthracite. This produces a layered-type bed that has some of the advantages of a dual-media bed. The design provides for some intermixing at the interface, and is superior in performance to a single medium.[49]

At Sheboygan, comparisons were made between anthracite-capped sand filters and sand alone, under various raw-water conditions, including comparisons made during the algae season. The capped filters were operated at 3 gpm/sq ft (7 m/h) and the sand at 2 gpm/sq ft (5 m/h). It was found that “the more adverse the applied water conditions, relative to algae and floc, the more dramatic are the results obtained with anthracite capped filter runs.” With good water conditions, the anthracite cap improved filter runs by a ratio of 2 to 1. The worst water conditions may give 10-to-1 improvement in filter runs. Through the use of capped filters, short filter runs can be eliminated.

TABLE 12–4. Typical Anthracite and Sand Distribution by Sieve Size in Dual-Media Bed

Anthracite Distribution by Sieve Size

U.S. Sieve No.	Percent Passing Sieve
4	99–100
6	95–100
14	60–100
16	30–100
18	0–50
20	0–5

Sand Distribution by Sieve Size

U.S. Sieve No.	Percent Passing Sieve
20	96–100
30	70–90
40	0–10
50	0–5

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At Kenosha, residual aluminum tests showed less alum passing through the capped filter than through the sand-only filter, except in cases of breakthrough. Bacteriologically, all tests showed safe water at all times. At Racine, it was found that the capped filters removed ‘‘more amorphous matter than sand alone.’’ At Evanston, capped filter operation at 4 gpm/sq ft (10 m/h) used less washwater than sand filters.

Rapid Sand Filters

Virtually all new water purification plants are now being designed to use mixed-media or dual-media filters, and many existing rapid sand filter plants are being converted to use these materials. However, some single-medium (sand or anthracite) filters are being installed in the expansion of existing plants to match existing facilities and to avoid the cost of converting all of the old units.

For practical purposes, the size of sand grains is determined on a weight basis from sieve analysis, even though the resulting diameters may be 10 to 15 percent less than those determined by the count and weight method, which should be used for more accurate results. The majority of rapid sand filters in use today contain sand with an effective size of 0.35 to 0.50 mm, although some have sand with an effective size as high as 0.70 mm. The uniformity coefficient is usually not less than 1.3 or more than 1.5.

Sand passing a 50-mesh (U.S. series) sieve is generally too fine for use in a rapid-sand filter, as it stratifies at the surface and shortens filter runs by sealing off the top quite rapidly. Sand retained on a 16-mesh sieve is too coarse to be useful in filtration within the depths normally used in filter plants. Therefore, filter sand usually ranges in size from that passing a 16-mesh to that retained on a 50-mesh sieve. Table 12–5 shows a typical sand specification.

TABLE 12–5. Suggested Size Specifications for Filter Sand

Sieve No.			Retained on Sieve, Percent	
Series Tyler $\sqrt{2}$	U.S. Series	Opening, mm	Minimum	Maximum
65	70	0.208	0	1
48	50	0.295	0	9
35	40	0.417	40	60
28	30	0.589	40	60
20	20	0.833	0	9
14	16	1.168	0	1

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Sand filters should have a hydrochloric acid solubility of less than 5 percent when tested in accordance with AWWA Standard B11-53. Sand should have a specific gravity of not less than 2.5, and be clean and well graded. After placement in the filter, sand should be backwashed three times at not less than 30 percent expansion, and then the top ¼ inch (6.4 mm) of very fine material should be carefully scraped off and discarded.

Anthracite Media

Crushed anthracite coal may be used in lieu of sand as a fine granular filter medium. It has a specific gravity of 1.5, compared to 2.65 for silica sand and crushed quartz. Anthracite is often used in deep (6 to 8 feet) monomedia beds with 1.2–1.6 mm media used for direct filtration and 0.9–1.1 mm used for coagulated and settled water. Because of the lower specific gravity of anthracite, only about half the backwashing velocity is needed for equal expansion (not necessarily equal washing) compared to sand. An anthracite bed of the same effective size as a sand bed has a greater bed porosity than the sand bed. Anthracite coal filter media should be clean and free of long, thin, or scaly pieces, with a hardness of 2.0 to 3.55 mm on the Moh scale and a specific gravity not less than 1.5.

Filter Control System

There are two basic methods of operating gravity filters that differ primarily in the way that the flow is applied across the filter. These methods are referred to as constant rate filtration (CRF) and declining rate filtration (DRF). CRF is the most widely employed method in the water industry, with either the influent or effluent flow controlled through each filter. This approach usually limits the maximum flow through the filter beds, minimizing rapid flow changes to prevent surges that can disrupt the filter bed. With DRF, the filter configuration is similar to CRF, except all filters discharge to a common effluent pipe or channel without use of flow control. Water levels in the filters are at a common level, with flow allowed to vary depending on how clean the individual filter beds are at the time. With CRF, there are three ways a filter can be operated:

- Influent flow splitting with constant water level
- Influent flow splitting with varying water level
- A rate-of-flow controller in the filtered water piping

Figure 12–11 illustrates these three methods of control and that of DRF. Following is a brief description of the CRF and DRF filters.

Constant Rate: Influent Flow Splitting with Constant Level This method of control generally uses individual weirs in the header channel entrance to each filter. The channel hydraulics and weir lengths must be carefully designed and generously sized to ensure that equal flow splitting occurs. The constant level is maintained by means of a level element in each filter. The level element sends signals to a modulating valve in the filter effluent line to control the total system headloss through the filter, and thus maintain a constant level. At the start of the filter run, the valve is only partially open so that the total headloss through the filter media, underdrain, gullet, piping, and control valve is equal to the total available filter head at the set water level in the filter. As the headlosses increase due to accumulation of particulate matter, the level tends to rise to maintain the flow through constant total headloss, thereby maintaining a constant level in the filter. When the flow to the filters increases due to backwashing other filters or increased plant flow, the level element signals the modulating valve to adjust accordingly.

The advantages of this type of filter operation are:

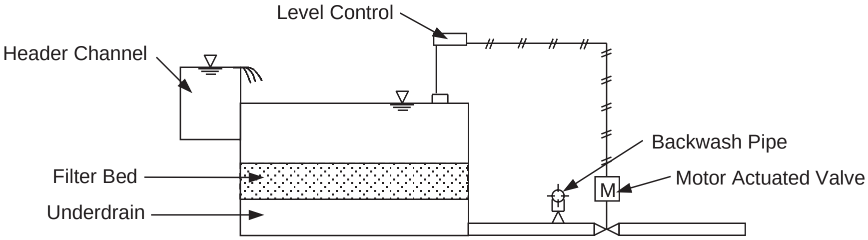
- Operator has control of the system.
- Filter flow element is not needed.
- Can respond gradually to flow variations.
- Filter aid is easy to apply.

The disadvantages are:

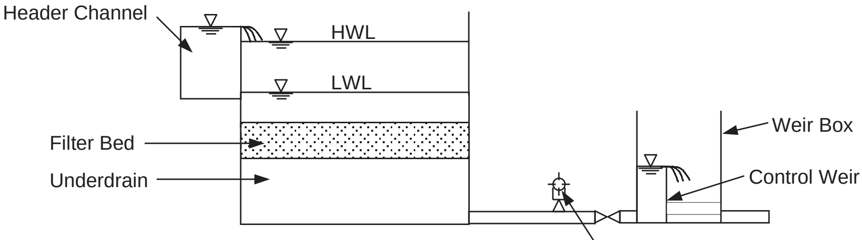
- Influent header channel and weir requiring careful hydraulic design to ensure equal flow split
- Added cost of channel width and weir
- Headloss over weir
- Level element and modulating control valve required in each filter
- Operator not knowing flow through each filter (i.e., if flow is split equally)
- Floc potentially breaking up at weir

Constant Rate: Influent Flow Splitting with Varying Water Level This control method is similar to the other influent flow splitting method except that there are no level elements, controllers, or modulating valves. At the start of a filter run, the water level is above the filter media. A device such as an effluent weir must be used to ensure that the minimum water level above the media when the filter is in a clean condition is acceptable.

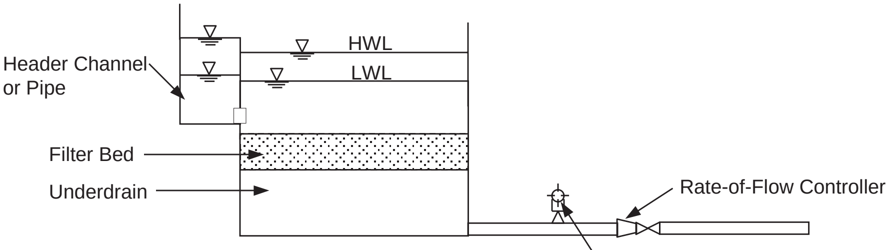
During a filter run, as the filter clogs, the water rises to overcome the headloss created by the clogged filter media. Changes in plant flow due to backwashing other



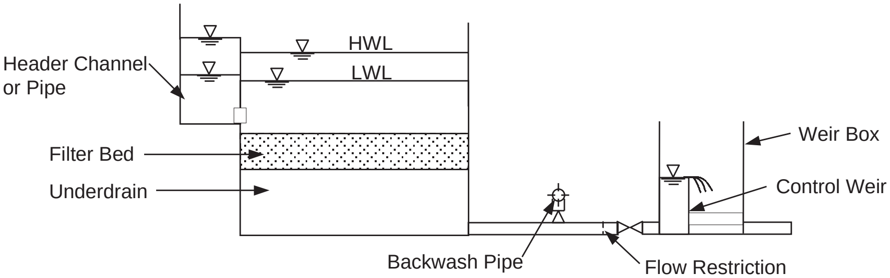
Constant Rate Filtration (Constant Level Influent Flow Splitting)



Constant Rate Filtration (Varying Level Influent Flow Splitting)



Constant Rate Filtration (Rate-of-Flow Controlled)



Declining Rate Filtration

Fig. 12–11. Potential options for operating gravity filters

filters or changing flow in the plant will cause the water level in the filters to rise or fall accordingly.

The advantages of this type of filter operation are:

- Filter effluent valve is a simple on–off type.
- Controls are simple.
- Filter flow or level element is not necessary.
- Filter aid is easy to apply.

The disadvantages are:

- Headloss over weir.
- Channel and weir costs can be high.
- Filter box must be deeper.
- Floc may break up at weir.

Constant Rate: Rate of Flow Controlled With this type of control, the water level in all filters and, generally, the filter header channel are maintained at a constant level. Flow in all filters is controlled by means of a Venturi or other type of meter and control valve in the filter effluent piping. A level element monitors the water level in the influent channel. A controller trims the control valves to maintain each filter flow to a set point.

During a filter run, the rate-of-flow controller maintains the flow through the filters at a set rate. As the filter clogs, the headloss through the filter increases, which reduces the flow. The controller senses the decrease in flow and opens the control valve to adjust for the increased head in the filter to maintain the set flow rate. The master level control in the influent channel senses variations in the water level in the channel, due to increased plant flow or the backwashing of a filter, and increases the set point flow through the rate-of-flow controllers.

The normal operating level range of this type of filter is about 6 inches. This range is wide enough so as not to shock the filter with surges.

The advantages of this type of filter operation are:

- Operator has full manual and/or automatic control of the filter system.
- Operator can vary the flow through any filter.
- Filter box can be designed shallower.
- Controls can be fully automated.
- No weir head and free-fall allowance needed.
- Can ramp flow at start of filter run.
- Responds gradually to plant flow variation.
- Filter aid is easy to apply.

The disadvantages are:

- Added cost of a rate-of-flow controller/meter
- Additional metering and control equipment needed

Declining Rate With DRF, the rate of filtration varies in each filter during a filter run, between each backwash. The filtration rate usually varies between 70 percent and 130 percent of the average filtration rate to the filter. When a freshly backwashed filter in a group of filters is put back into service, it will operate at highest average filtration rate of all the filters until the next filter is backwashed and put back into service. When the backwashed filter is placed back in service, the filtration rate in the filter under consideration will decline in a step-wise manner, to a new lower filtration rate. This rise and fall motion of the water level over the filters will continue along with the stepped decline in the filtration rate until the filter is the last filter to be backwashed, at which time it will be filtering at its lowest rate. During this cycle, the water level in all other filters will rise in unison.

The advantages of this type of filter operation are:

- Simpler operation
- Less filtering head is required for filter runs of equal length
- No weir head and free-fall allowance are needed
- No rate-of-flow controller/meter is needed
- Hydraulically responds well to plant flow variations

The disadvantages are:

- Continual manual surveillance or controls must be added.
- Difficult to effectively use a filter aid.
- Filter box must be deeper.
- Less likely to be effective at higher filtration rates.
- Influent header channel and valves must be proportionally larger, and therefore more expensive.
- The initial high rate of flow into the filter box can cause media migration.
- Operator has little control flexibility.

Conclusions Each of the four options for filter control logic is potentially feasible for overall plant treatment scheme. However, the constant rate rate-of-flow controlled scheme is considered the most suitable type of control because it:

- Provides ability to control flow through each filter.
- Does not cause floc breakup on filter influent water.
- Allows gradual changes of flow through filters.
- Easily accommodates conditioning of influent water with filter aid.

Filter Backwashing

During the service cycle of filter operation, particulate matter removed from the applied water accumulates on the surface of the grains of fine media and in the pore spaces between grains. With continued operation of a filter, materials removed from the water and stored within the bed reduce the porosity of the bed. This has two effects on filter operation: it increases the headloss through the filter, and it increases the shearing stresses on the accumulated floc. Eventually the total hydraulic headloss may approach or equal the head necessary to provide the desired flow rate through the filter, or there may be a leakage or breakthrough of floc particles into the filter effluent. Just before either of these outcomes can occur, the filter should be removed from service for cleaning. In the old slow-sand filters, the arrangement of sand particles is fine to coarse in the direction of filtration (down); most of the impurities removed from the water collect on the top surface of the bed, which can be cleaned by mechanical scraping and removal of about ½ inch (10 mm) of sand and floc. In rapid-sand filters, there is somewhat deeper penetration of particles into the bed because of the coarser media used and the higher flow rates employed. However, most of the materials are stored in the top few inches of a rapid-sand filter bed. In dual-media and mixed-media beds, floc is stored throughout the bed depth to within a few inches of the bottom of the fine media.

Rapid-sand, dual-media, mixed-media, and deep monomedia, filters are cleaned by hydraulic backwashing (upflow) with potable water. Backwashing of filters that have become clogged with particulate matter captured on the grains of fine media and in the pore spaces between grains can be accomplished in a variety of methods.

Types of Backwash Systems The traditional backwash system uses treated water wash, with the water introduced into the bottom of the filter bed through the underdrain system to obtain full-bed fluidization. Three basic approaches are normally employed for gravity filter backwashing:

- Treated water wash
- Treated water wash with surface wash
- Treated water wash with air scour

Surface wash and air scour greatly reduce the potential for formation of mud balls that endanger filter effectiveness.

The surface wash assisted wash water system is the most commonly used backwash system for gravity filters. Although fixed nozzles can be used, normal practice is to employ a rotary arm mechanism for the surface wash system, with nozzles located directly above the filter bed so that they are in the expanded bed during backwash. Fixed-nozzle assemblies are not normally used, as they restrict access to the filter media and are costly to maintain. The advantages of a rotary surface wash system are that the:

- System is simple to implement and operate, as a source of high-pressure water is the only auxiliary system needed.
- Equipment is easily accessible for maintenance.

The disadvantages of a surface wash system are that the:

- Rotary arm can stop rotation, preventing proper cleaning of the filter media.
- Corner areas of the filter chamber are difficult to effectively clean, as the rotating nozzles cannot pass directly over the filter surface.
- Surface wash mechanism cleans only at one horizontal plane in the filter, providing no benefit to cleaning of the bed area further below the mechanism.

- Surface wash system requires approximately 1–2 gpm/sf of water flow to operate, increasing plant water production requirements.

An air scour backwash system is used in a number of methods to improve the effectiveness of the wash water or to reduce the backwash water flow rates. The air may be used prior to the water backwash or concurrently with the water backwash. Air is introduced through orifices located beneath the filter media. Potential advantages of an air-scour auxiliary system are that the:

- Air scour evenly covers the entire surface area of the filter bed.
- Full depth of the filter media is agitated.
- Volume of wash water required may be reduced by shortening the duration of the wash.

The air-scour auxiliary disadvantages are:

- The potential for loss of filter media is greater, particularly if simultaneous air scour and wash water are used.
- Movement of the support gravel is a potential concern if the air is introduced below the gravel.
- The auxiliary system is more complicated as an air blower, piping system, and control system are required.

The most effective backwash is provided by simultaneous air scour and wash water fluidization. This method is used in deep monomedia beds made of media that is too coarse to fluidize. However, this method is very susceptible to media loss problems if the media is fluidized during the backwash, as in traditional filter design. Using air scour prior to wash water fluidization is considered to have about the same effectiveness as the use of surface wash with wash water fluidization. Reductions in the volume of backwash waste water can be accomplished with simultaneous air/water backwash.

Water/Air Source The supply of water for the filter backwashing operations is filtered water. There are a number of options for supplying wash water that are commonly used, including:

- Interconnection with the high service pump discharge
- Gravity flow from an aboveground wash water tank
- Gravity flow from a higher elevation storage reservoir off-site
- Pumping from the on-site storage reservoir

The most common approach provides backwash by direct pumping from the plant storage.

The supply of air for the air scour can be generated by on-site blowers, with the air piped to the filters. The high-flow/low-pressure air requirements can be handled by a single blower connected to the filters through a header system. A standby blower should be provided to ensure reliability of the backwash system.

Wash Water Troughs Normal practice is utilizing evenly distributed wash water troughs above the filter media to collect backwash water, and direct flow to the wash water gullet and drain system. The troughs serve to equalize head on the underdrains and provide uniform upflow of wash water through the media. The bottom of the trough must be positioned above the expanded zone of the filter media during backwashing to prevent media loss. Traditional design is to space the troughs at 1½ to 2 times the distance between the trough weir and filter media surface. Most design standards require that the horizontal travel of suspended particulates not exceed 3 feet. The spacing must also allow each trough to serve the same amount of filter surface area. The troughs are usually constructed of concrete or fiberglass-reinforced plastic, and span the filter chamber to avoid interior filter supports. The troughs should be designed to permit field leveling of the trough weirs after installation to assure even upflow during backwashing. Some freeboard should be allowed to prevent flooding of wash water troughs and uneven distribution of wash water.

The dimensions of a filter trough may be determined by use of the following equation:

$$Q = 2.49bh^{3/2} \tag{12–2}$$

where:

Q = the rate of discharge, cu ft/sec (m^3/h)
b = width of trough, ft (m)
h = maximum water depth in trough, ft (m)
2.49 = constant for English units (4950 is the constant for metric units shown)

Sequential air-scouring systems do not use wash troughs across the filter bed area. Instead, they use a single overflow wall along a central gullet and a single V-shaped trough along the opposite wall. In these systems, the width of each filter cell must be limited to about 14 feet (4.5 meters) for the wash water to be effectively removed.

Backwash Hydraulics The required backwash water upflow through the filters is variable and dependent upon a number of factors. These factors include water temperature, filter media type, and washing method. The rate of flow must be sufficient to fluidize the filter media in order to ensure proper cleaning of the filter. Research has shown that optimum cleaning is obtained with 25–35 percent expansion.

Water viscosity increases with decreasing temperature, increasing friction with the media and lowering the wash water rates needed for bed expansion. As a result, the backwash system must be designed for the warmest water temperature to ensure adequate backwash water flow under all operating conditions. For example, a backwash rate of 18 gpm/sq ft (45 m/h) at 68°F (20°C) equates to 15.7 gpm/sq ft (38 m/h) at 41°F (5°C) and 20 gpm/sq ft (50 m/h) at 95°F (35°C). The time required for complete washing varies from 3 to 15 minutes.

The proper backwash rate should be based on the grain size of the filter media, its specific gravity, and the high and low water temperature of the region (Fig. 12–12). The adjustments of backwash water for water temperature can be calculated using the following equation:[56]

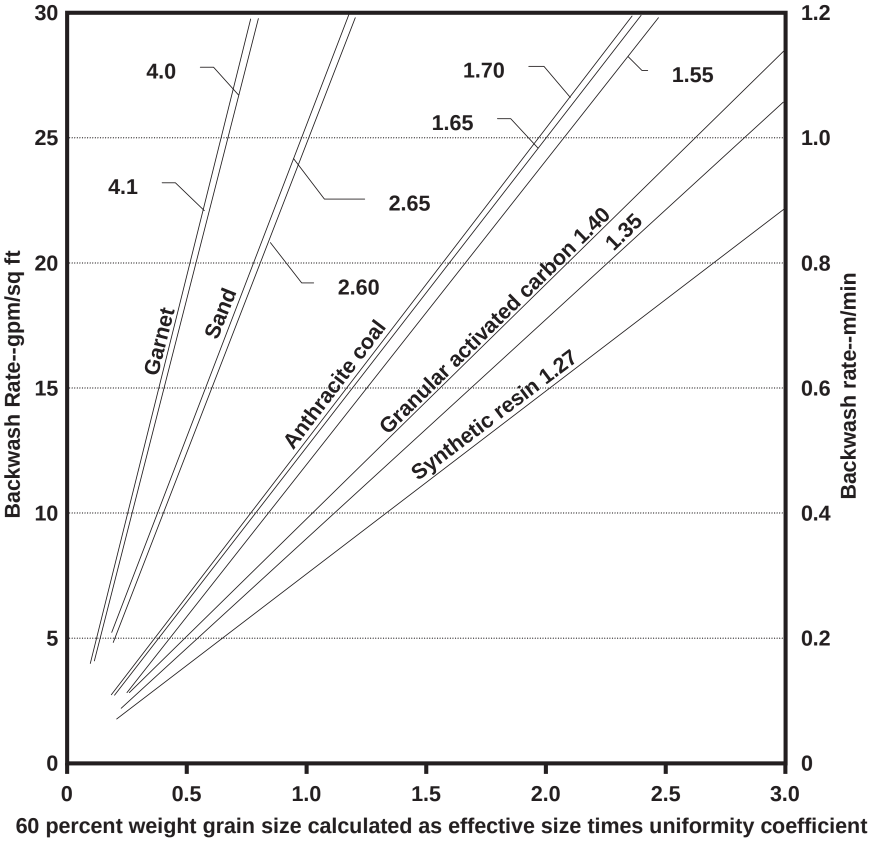


Fig. 12–12. Backwash rates based on filter media grain size (Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association and American Society of Civil Engineers)

$$v_{b-t} = v_{b-20} \times \mu t^{-t/3} \tag{12–3}$$

in which t is the temperature in °C, v_{b-t} is the backwash rate at temperature t, v_{b-20} is the backwash rate at 20°C, and μ is the water viscocity in centipoise at temperature t. Figure 12–12 applies only for a media with a uniformity coefficient ≤1.5 and only if fluidization of the filter bed is required (fluidization air-scouring filter wash). A backwash rate of at least 18 gpm/sq ft (45 m/h) is necessary to purge air bubbles trapped in coarse deep filter beds after air scouring.[56]

Media characteristics impact the wash water flow, increasing with larger media size and higher media density. If more than one type of media is employed, the backwash rate must provide for proper stratification of the filter media. Each method of filter washing has characteristic upflow rates and durations. The design of the underdrain and auxiliary scour systems largely affect the flow rate required.

The required head for backwashing gravity filters typically ranges from 4 to 7 feet, with additional head required to compensate for headlosses in the piping system between the backwash pump and the filters and the required static lift.

Air Scour/Fluidization There are three general approaches to use of air and water filter backwashing:

- Air scour alone followed by low-rate water wash
- Air scour alone followed by high-rate water wash
- Simultaneous air scour and water wash

Typical air and water rates are shown in Table 12–6. One manufacturer offers an air-scour system that can readily be retrofitted to an existing filter. The system consists of a grid of air diffuser pipes that is installed from above with the filter remaining in service. The grid rests just above the surface of the filter media and is connected to an air blower.

Filter-to-Waste The purpose is to waste filter effluent at the start of each filter cycle after the completion of backwashing until the filter effluent turbidity level drops to an acceptable level. The presence of high turbidity levels in the filter effluent at the start-up of a filter after backwash is considered more than just a ‘‘ripening’’ process. Instead, it is viewed as a flushing out of remnant backwash solids and material released during particle collisions during the closure of the backwash valves. A 2- to 20-minute period may be required for filter-to-waste depending on the pretreatment effectiveness. The use of filter-to-waste has gained renewed interest in the past several years on surface

TABLE 12–6. Typical Water and Air–Scour Flow Rates for Backwater Systems Employing Air Scour

Filter Medium	Backwash Sequence	Air Rate, scfm/sf (m/h)	Water Rate,* gpm/sf (m/h)
Fine sand 0.5 mm ES	Air first Water second	2–3 (37–55)	 15 (37)
Fine dual and multimedia 1.0 mm Es anthracite	Air first Water second	3–4 (55–73)	 15–20 (37–49)
Coarse dual media 1.5 mm ES anthracite	Air first Air + water on rising level Water third	4–5 (73–91) 4–5 (73–91)	 10 (24) 25 (61)
Coarse sand 1.0 mm ES	Air + water 1st Simultaneously Water second	3–4 (55–73)	6–7 (15–17) Same or double rate
Coarse sand 2 mm ES	Air + water 1st Simultaneously Water second	6–8 (110–146)	10–12 (24–29) Same or double rate
Coarse anthracite 1.5 mm ES	Air + water 1st Simultaneously Water second	3–5 (55–91)	8–10 (20–24) Same or double rate*

(Courtesy of Water Quality Improvement Center, Yuma, AZ)

*Water rates for dual and multimedia vary with water temperature and should fluidize the bed to achieve restratification of the media. See Equation 12–3.

water supplies that contain *Giardia* and *Cryptosporidium* cysts. During normal start-up of a filter, turbidity passage is small when averaged with the effluent turbidity of the other filters. However, *Giardia* and *Cryptosporidium* are infective even at low levels, so pose a health risk when passed through only one filter. Filter-to-waste piping typically discharges to the backwash waste water pipe header. An air gap is needed on the filter-to-waste connection to the backwash waste water piping to avoid creating a cross-connection.

The addition of coagulant or polymer to the backwash water as an alternative to filter-to-waste has been used in some cases. The goal is to condition the filter before filtration resumes. It is not always successful. Some instances of floc formation and sludge deposition in clearwells have been reported.[56]

Filter Backwash Concerns

Filters can be seriously damaged by slugs of air introduced during filter backwashing. The supporting gravel can be overturned and mixed with the fine media, which requires removal and replacement of all media for proper repair. Air can be unintentionally introduced to the bottom of the filter in a number of ways. If a vertical pump is used for the backwash supply, air may collect in the vertical pump column between backwashings. The air can be eliminated without harm by starting the pump against a closed discharge valve and bleeding the air out from behind the valve through a pressure air release valve. The air release valve must have sufficient capacity to discharge the accumulated air in a few seconds.

Also, air or dissolved oxygen, released from the water on standing and warming in the washwater supply piping, may accumulate at high points in the piping and be swept into the filter underdrains by the inrushing washwater. This can be avoided by placing a air release valve at the high point in the line, and providing a ½-inch (12.7-mm) pressure water connection to the washwater supply header to keep the line full of water and to expel the air.

The entry for washwater into the filter bottom must be designed to dissipate the velocity head of the washwater in such a manner that uniform distribution of washwater is obtained. Lack of attention to this important design factor has often led to difficult and expensive alterations and corrective repairs to filters.

The use of a high-pressure—above 15 psi (100 kPa)—source of filter backwash water through a pressure-reducing valve is not advised. Numerous failures of systems using pressure-reducing valves have so thoroughly upset and mixed the supporting gravel and fine media that these materials have had to be completely removed from the filter and replaced with new media.

Filter Agitators

Practice in the United States leans heavily toward installation of the essential, but misnamed, ‘‘surface wash’’ on all new filters. ‘‘Auxiliary scour’’ or ‘‘filter agitation’’ better describes the function of this device, as it aids in cleaning much more than the filter surface. Rotary surface washers are the most common, but fixed jets also have been used successfully.

Adequate surface wash improves filter cleaning and prevents mudball formation and filter cracking. Conventional rotary surface wash equipment consists of arms on a fixed swivel supported from the washwater troughs about 2 inches (50 mm) above the surface of the unexpanded filter media. In dual- or mixed-media filters, dual-arm devices have also been used to provide better cleaning at the anthracite–sand interface. In these systems, a second set of arms is located about 6 inches (150 mm) above the anthracite–sand interface in a dual-media filter or about 18 inches (450 mm) above the top of the gravel in a mixed-media filter. The arms are fitted with a series of nozzles, and revolve because of the water jet reaction. Water pressures of 40 to 100 psi (275 to 700 kPA) are required for the operation of the rotary surface wash, depending upon the diameter of the arms. The volume required is about 0.5–1.0 gpm/sq ft (1.2–2.4 m/h). Surface washers are usually started about 1 minute before the end of the backwash period. It is recommended that the nozzles be equipped with rubber caps that act to prevent entry of fine filter media and plugging of the nozzles.

Baylis designed a fixed-jet surface wash system consisting of a grid of distributing pipes extending to within a couple of inches of the top surface of the bed. Nozzles with five ¼-inch (6-mm) holes are spaced at about 24- to 30-inch (600- to 750-mm) centers each way. The required flow is about 2 gpm/sq ft (5 m/h) at a head of 20 to 60 feet (6 to 18 m).

Surface wash piping is a direct cross-connection between filtered and unfiltered water. Normal practice is to bring the surface wash header into the filter over the top of the filter box and to fit it with a vacuum breaker and a check valve at the high point to prevent backsiphoning. A single vacuum breaker on the surface wash header will suffice. An alternate is to use settled water through a separate surface wash pump. The filter valves must be in the proper position to ensure proper cleaning of the filters. Table 12–7 shows valve positions for various cycles of filter operation.

Loss of Head Monitoring

The loss of head through a filter provides valuable information about the condition of the bed and its proper operation. An increase in the initial loss of head for successive runs over a period of time may indicate clogging of the underdrains or gravel, the need for auxiliary scour, or insufficient washing of the beds. The rate of headloss increase during a run yields considerable information concerning the efficiency of both the pretreatment and the filtration operation.

The determination of headloss through a filter is a very simple matter, involving only the measurement of the relative water levels on either side of the filter. The simplest form of headloss device for gravity filters is made up of two transparent tubes

TABLE 12–7. Valve Positions During Various Treatment Operations

	Valve Position		
Valve	Filtering	Backwashing	Filtering to Waste
Influent	Open	Closed	Open
Effluent	Open	Closed	Closed
Filter-to-waste	Closed	Closed	Open
Washwater supply	Closed	Open	Closed
Washwater drain	Closed	Open	Closed
Surface wash or air scour	Closed	Open	Closed

(From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

installed side by side in the pipe gallery with a gauge board between, graduated in feet. One tube is connected to the filter or filter influent line, the other to the effluent line. The headloss is the observed difference in water levels. More sophisticated methods for measuring the difference in water levels include float-actuated and differential-pressure-cell-actuated indicating and recording devices. The headloss may be indicated and recorded at some remote point if desired. Headloss equipment may be used in connection with control systems for automatic backwashing of filters, as one means of initiating the backwash cycle when the headloss reaches some preset maximum value.

Use of Polymers as Filtration Aids

Polymers are high-molecular-weight, water-soluble compounds that can be used as primary coagulants, settling aids, or filtration aids. They may be cationic, anionic, or nonionic in charge. Generally, the doses required as a filtration aid are less than 0.1 mg/L. Used as a filtration aid, polymer is added to increase the strength of the chemical floc and to control the depth of penetration of floc into the filter. For maximum effectiveness as a filtration aid, the polymer should be added directly to the filter influent and not in an upstream settling basin or flocculator. However, if polymers are used upstream as settling aids, it may not be necessary to add any additional polymer as a filtration aid.

Figure 12–13 illustrates the effects of polymers as filter aids. The conditions represented in part A of the figure illustrate the results of a fragile floc shearing and then penetrating the filter, causing a premature termination of its run due to breakthrough of excessively high effluent turbidity. If the polymer dose is too high (part B), the floc is too strong to permit penetration into the filter, causing a rapid buildup of headloss in the upper portion of the filter and premature termination due to excessive headloss. The optimum polymer dose will permit the terminal headloss to be reached simultaneously with the first sign of increasing filter effluent turbidity or particle counts.

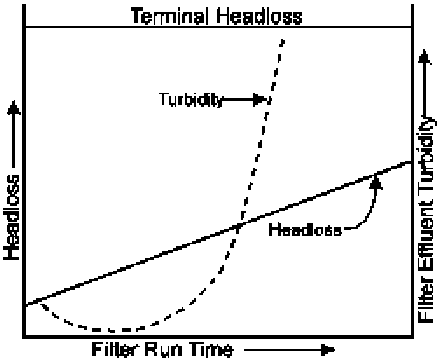
Many polymers are delivered in a dry form. They are not easily dissolved, and special polymer mixing and feeding equipment is required. Many polymers are biodegradable and cannot be stored in dilute solution for more than a few days without suffering significant degradation and loss of strength. (Chapter 24 contains detailed information on feeding and handling systems for polymers.)

Monitoring Filter Effluent Quality

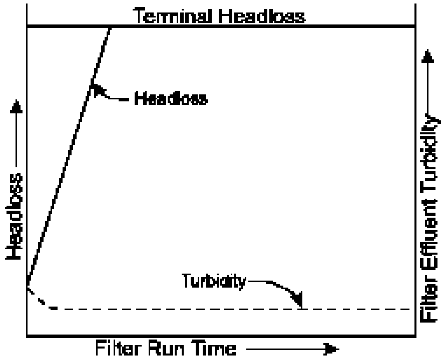
Turbidimeters and particle counters are available to monitor the quality of the filter effluent continuously. Should either exceed the desired level, a signal from the turbidimeter or particle counter can be used to sound an alarm or to initiate the backwash program. Recording of the turbidimeter or counter output provides a continuous record of filter performance.

Pilot Filters

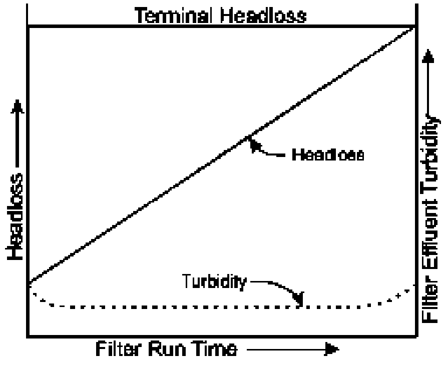
A small filter receiving raw water dosed with the treatment chemicals is often operated in parallel with the plant filters. The pilot filter directly measures the filterability of the water under actual plant operating conditions. By fitting the pilot filter with a recording turbidimeter, chemical dosages can be adjusted to obtain the desired quality of filter effluent before water in the full-scale plant reaches the filters.



(A) POLYMER DOSE INADEQUATE



(B) POLYMER DOSE EXCESSIVE



(C) OPTIMUM POLYMER DOSE

Fig. 12–13. Effects of polymers as filter aids (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Filter Control Sequencing

Filter runs are usually terminated when either the headloss reaches a predetermined value, the filter effluent turbidity or particle count exceeds the desired maximum, or a certain amount of time passes. Each of these events is adaptable to instrumentation that can be used to signal the need for backwashing or to trigger a fully automated backwash system. Automatic control of filter backwashing may be provided, which is interlocked so that the necessary prerequisites for each step are completed before the next step is begun. At the receipt of a backwash start signal, the following events typically occur in the sequence listed in this illustrative program: Filter influent and effluent valves close. Any chemical feed to the filter being backwashed stops. Plant chemical feeds adjust to the new plant flow rate, to maintain proper chemical feed to the filters still in service. The waste valves starts to open. When the waste valve reaches the fully open position and actuates a limit switch, the surface wash pump starts, and the surface wash valves open. Surface wash flow to waste continues for a period of time adjustable up to 10 minutes.

At the end of the initial surface wash period, usually 1 to 2 minutes, the main backwash valve opens. The backwash and surface wash both continue for a period of time, usually 6 to 7 minutes, adjustable up to 30 minutes. The backwash flow rate is indicated on a controller and is controlled automatically to a manual set point. At the end of the combined wash periods, the surface wash valves close, and the surface wash pump stops. The backwash continues without surface wash for a time, usually 1 to 2 minutes, adjustable up to 30 minutes. At the completion of the backwash period, the backwash has closed, influent and effluent waste valves open, and the bed filters to waste for a period of time, usually 3 to 7 minutes, adjustable up to 30 minutes. The backwash delay timer resets and begins a new timing cycle, adjustable up to 12 hours. The bed selector switch steps to the next filter. Chemical feed to the clean filter is reestablished. At the end of the filter-to-waste period, the effluent waste valve closes, and the effluent valve opens to restore the cleaned bed to normal filter service. Provision should be made for optional manual operation of all automatic features.

It may be desirable to set alarms for certain functions that affect filter operation, on a conveniently located annunciator panel. These alarm functions include high turbidity, high headloss, low plant flow, low backwash flow rate, and excessive length of backwash.

Filter Piping, Valves, and Conduits

Ductile-iron pipe and fittings or cement mortar–lined welded steel pipe and fittings are the most widely used materials for filter piping. The layout of filter piping must include consideration of the ease of valve removal for repair and easy access for maintenance. Flexible pipe joints should be provided at all structure walls to prevent pipeline breaks due to movement. Color coding of the filter piping is a valuable operating aid. The filter piping is usually designed for the flows and velocities shown in Table 12–8. The filter influent should enter the filter box so that the velocity of the incoming water does not disrupt the surface of the fine media. This is often done by directing the influent stream against the gullet wall, thus dissipating the velocity head within the gullet. It can also be done by locating the influent pipe below the top of the filter troughs so that the water enters the filter through the troughs. A further

TABLE 12–8. Filter Piping Design Flows and Velocities

Description	Velocity ft/sec (m/s)	Maximum Flow, gpm/sq ft (m/h) of Filter Area
Influent	1–4 (0.305–1.22)	8–12 (20–30)
Effluent	3–6 (0.92–1.83)	8–12 (20–30)
Washwater supply	5–10 (1.52–3.05)	15–25 (35–60)
Backwash waste	3–8 (0.92–2.44)	15–25 (35–60)
Filter-to-waste	6–12 (1.83–3.66)	4–8 (10–20)

precaution is to install an influent valve with throttling control for use in slowly refilling the beds. The filter-to-waste connection to the filter should have positive air gap protection against backsiphoning from the drain to the filter bottom. The filter-to-waste, effluent, and washwater supply lines usually are manifolded for common connection to the filter underdrain system.

In the design of pipe galleries, reinforced concrete flumes and box conduits and concrete-encased concrete pipe may be used for washwater drains or other service when located adjacent to the pipe gallery floor, but should not be installed overhead because of difficulties with cracks and leaks. Invariably, pipe galleries with overhead concrete conduits are drippy, damp, unsightly places with a humid atmosphere that discourages good housekeeping by making it difficult to maintain. Instead, pipe galleries should be provided with positive drainage, good ventilation, sufficient light, and dehumidification equipment (if required by the prevailing climate). Filter influent and effluent lines should be provided with sample taps.

The rubber-seated, electrically or pneumatically actuated and operated butterfly valve has almost entirely replaced the hydraulically actuated and operated gate valves that were formerly used extensively as filter valves. Of the two types, the butterfly valve is smaller, lighter, easier to install, and better for throttling services, and it can be installed and operated in any position. The valves should be factory-equipped with the desired valve stops, limit switches, and position indicators because field-mounting of these devices is often unsatisfactory.

Each filter unit, except split beds, should have six valves for its proper operation: influent, effluent, washwater supply, washwater drain, surface wash or air scour, and filter-to-waste. The positions of these valves during the three cycles of filter operation were given in Table 12–7.

Existing Plant Expansion and Conversion

Because multimedia filters operate more efficiently, safely, and reliably at 5 gpm/sq ft (12 m/h) than do conventional rapid-sand filters at only 2 gpm/sq ft (5 m/h), there obviously is great potential for expanding the capacity of existing plants at least up to double with only the nominal expense of replacing sand with dual or mixed media. This, of course, has been done in a great many instances. Because of the ability of multimedia filters to remove and store solids from high-turbidity waters, often it is not necessary to add settling basin capacity in plant expansion. In other cases, this must be done, or, as an alternative, settling tubes may be installed in existing basins because this change will allow increasing basin throughput without loss of settling efficiency.

Filter Design Checklist

In designing a rapid rate filter, a number of design considerations must be addressed to ensure proper filter operation. Following is a checklist of some of the more critical design issues:

1. Filter media sizing and selection should be based on pilot tests. If this is not possible, data should be obtained from similar applications to determine the suitability of the media design.
2. In dual- and mixed-media and high-rate, deep monomedia filter systems, provisions should be made for the addition of polyelectrolytes directly to the filter influent.
3. The turbidity and/or particle count of each filter unit should be monitored continuously and recorded.
4. The flow and headloss through each filter should be monitored continuously and recorded.
5. Provisions should be made for the addition of disinfectant directly to the filter influent.
6. Pressure filters must be equipped with pressure and vacuum air release valves.
7. Provisions should be made to divert any filter effluent of unsatisfactory quality (i.e., provide a filter-to-waste cycle).
8. Provisions should be made for automatic initiation and completion of the filter backwash cycle.
9. The filter controls and pipe galleries should be housed. Pipe galleries should have humidity control systems.
10. Filter piping should be color-coded.
11. The filter system layout must enable easy removal of pumps and valves for maintenance.
12. The backwash rate must be based upon the specific filter media used and the backwash water temperature variations expected.
13. Filter backwash supply storage should have a volume at least adequate to complete two filter backwashes.
14. Adequate surface wash or air-scour facilities must be provided.
15. There should be adequate backwash and surface wash pump capacity available with the largest single pumps out of service.
16. Backwash supply lines must be equipped with air release valves.
17. A means should be provided to indicate the backwash flow rate continuously and to enable positive control of the filter backwash rate. A means should also be provided to limit the filter backwash rate positively to a preset maximum value.
18. The filter design must incorporate underdrains and backwash wastewater collection devices that ensure uniform distribution of backwash water and filter influent.
19. The filter system should be equipped with an alarm system that will indicate major malfunctions.
20. Construction details must prevent cross-connections and backflow.

FILTER PROBLEMS AND POTENTIAL SOLUTIONS

Problems in filter operation and performance can be caused by poor design or poor operation. However, advances in the engineering design of filters and filter controls and appurtenances have made water filtration an inherently stable, extremely efficient, and highly reliable unit treatment process. With proper design and good operation, all the problems are easily solved.

Some potential filter problems are:

- Surface clogging and cracking
- Short runs due to rapid increases in headloss
- Short runs due to floc breakthrough and high effluent turbidity
- Variations in effluent quality with changes in applied water flow rate or quality
- Gravel displacement or mounding
- Mudball formation
- Growth of filter grains, bed shrinkage, and media pulling away from sidewalls
- Sand leakage
- Loss of media
- Negative head and air binding
- Air leakage into the system

Surface Clogging and Cracking These conditions are usually caused by rapid accumulations of solids on the top surface of the fine media. This is not normally a problem in dual- or mixed-media filters because of the greater porosity of their top surface, compared to sand. Also, when a filter aid is used with dual- or mixed-media filters, the dosage can be reduced as necessary (or eliminated) to allow particulates to penetrate deeper into the bed. In other words, regulation of the polymer dosage to the filter influent gives some control over the effective porosity of the filter, to accommodate changes in incoming floc characteristics.

Rapid Increases in Headloss This is related to the problem just discussed. Dual- and mixed-media beds collect particulates throughout the depth of the bed, rather than mostly at the surface of the bed as with a sand or other surface-type filters, and are much less susceptible to this problem than are the surface-type filters. Also, the flexibility provided by use of a polymer as a filter aid allows control of the rate of headloss buildup through dosage changes.

Floc Breakthrough Floc breakthrough can be avoided by increasing filter aid dosages or converting rapid-sand filters to dual-media filters or mixed-media filter units. As mentioned earlier, this is one important point of superiority of mixed-media and dual-media over sand filters. It arises because of the much greater surface area of the grains in a mixed-media or dual-media filter compared to sand. The finest medium is 40 to 80 mesh in a mixed-media bed (10 percent of the total bed), 40 to 50 mesh (9 percent of total) in a sand bed, and 40 to 50 mesh (5 percent of total) in a dual-media filter. The finest medium (garnet) in a mixed-media bed has the additional advantage not only of being finer but also of being located at the very bottom of the filter where the applied load is lightest, and where it can serve its intended purpose as a polishing agent. Each type of filter media is capable of storing floc in the bed, with most of the floc retained in the upper ⅓ of each filter media layer. Floc storage depths available above the finest media in various typical beds are:

- Single media, 0 inches (a single media bed has no storage above it)
- Dual media, 18 inches (450 mm)
- Mixed media, 27 inches (700 mm)

A further advantage of mixed-media and dual-media filters in this regard is the greater total number of media particles contained in an equal volume of bed. This tremendously increases the number of opportunities for contact between media and colloids in the water, which greatly enhances removal of these colloids. It is these superior properties of mixed media and dual media in resistance to leakage of particulates and much greater removal of colloids that make the use of single-media beds debatable under any circumstances. When coupled with the proper use of polymers as a filter aid, effective filter media design can eliminate short runs due to floc breakthrough.

Mounding Gravel displacement or mounding can be eliminated by use of a 3-inch (75-mm) layer of coarse garnet or ilmenite between fine media and the gravel supporting bed as previously recommended, and by limiting the total flow and head of water available for backwash. Avoid using washwater from a high-pressure source through a pressure-reducing valve that could fail.

Mudball Formation This can be eliminated by providing an adequate backwash flow rate [up to 20 gpm/sq ft (50 m/h)] and a properly designed system for auxiliary scour (surface wash or air scour). The successful use of beds employing filter aids and in-depth filtration is dependent on provision and operation of a good system of auxiliary scour. Small media particles enhance mudball formation. It has been shown that the size of sand particles in mudballs is much smaller than the effective size of the sand media.[25] Thus, proper removal of the fines when placing media will assist in mudball control.

Growth of Filter Grains Growth of filter grains, bed shrinkage, and media pulling away from filter sidewalls are related problems. Again, the provision and use of adequate backwash facilities, including surface wash or air scour, are the keys. It is the compressibility of filter grains that are heavily coated with materials filtered, deposited, or absorbed from the water that is the root of these difficulties. These problems usually can be avoided by proper backwashing. The growth of particles refers to a macroscopic increase in size and not to the development of a microscopic film of polymer and other chemicals. The microscopic film results from proper use of a filter aid that is beneficial in adsorption and retention of particulates for the period of a single operational cycle. This microscopic layer is not nearly thick enough to create a problem by increasing the compressibility of the bed. An alum or polymer film on filter grains may actually be an aid in reducing the adherence of calcium carbonate and facilitating its removal during backwashing. However, in filtering lime-softened waters it is important to adjust the pH of the filter influent by addition of carbon dioxide or acid to a level at which calcium carbonate deposition does not occur.

Sand Leakage This can be prevented by using the coarse garnet layer between the fine media and gravel supporting bed as recommended earlier. The garnet layer prevents the downward migration and escape of media fines.

Loss of Media The loss of filter media, particularly anthracite during backwashing, is one problem for which there is no complete solution. Losses can be reduced by increasing the distance between the top of the expanded bed during maximum backwash flows and the washwater troughs. It also can be helped by cutting off air wash or auxiliary scour 1 or 2 minutes before the end of the main backwash. A loss of 1–2 inches per year of anthracite is not unusual, and should be replaced with new media periodically.

Negative Head and Air Binding These can be avoided in most cases, but there may be a few extreme situations, usually of short duration, where they cannot be entirely eliminated. In any case, it is a good idea to provide a water depth of at least 5 feet (1.5 m) above the surface of the unexpanded filter bed. The more depth the better, at least as far as negative head and air binding are concerned. The filter should not be operated to terminal headlosses that are greater than the depth of submergence of the filter media. in order to minimize the potential for air binding.

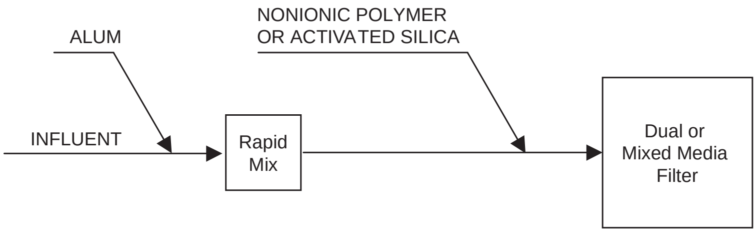
When filter influent water contains dissolved oxygen at or near saturation levels, and when the pressure in the filter media is reduced by siphon action to less than atmospheric at a point below the surface of the fine media, the oxygen comes out of solution, and gas bubbles are released. They may accumulate within the bed and tremendously increase the resistance to flow, or headloss. When flow through a filter is stopped and the water level is lowered in preparation for backwashing, bed pressures are reduced and more oxygen is released. Even further release of bubbles occurs during backwash, which may lead to loss of media in the waste backwash water when bubbles adhere to anthracite or sand particles and carry them into the washwater troughs. More frequent filter backwashing may alleviate the problem to some extent, as there is then less time for bubbles to accumulate. However, when the problem is acute—as it may be in the spring when surface water is warming, and oxygen solubility is decreasing at the higher temperatures—it may only be endured and not solved. Maintaining maximum water depths above the beds and frequent backwashing may help resolve, but may not completely eliminate, the difficulties.

Air Air can leak into the system by a variety of means. If the filter rate control valve is located above the hydraulic gradient, air may enter the system through the stuffing box between the valve and the valve operator. The problem is aggravated by the aspiration effect of a control valve located in a reduced cross section of pipe. A similar problem can occur during the filtration cycle through the stuffing boxes on surface wash systems, where one of the surface wash arms is located in the media. If there is a negative head in the filter, air may be pulled through the stuffing box into the bed during the filtration cycle.[50] Such leakage can be prevented by locating the stuffing box at an elevation that is always submerged during the filter cycle.

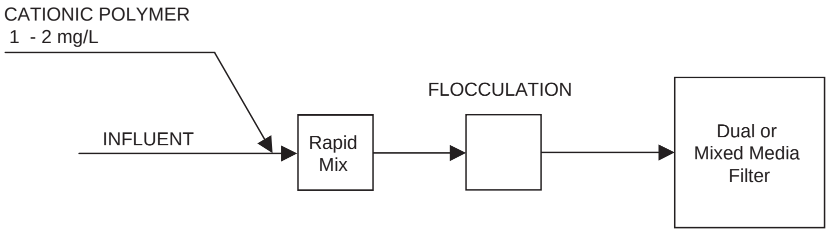
DIRECT FILTRATION

The ability of dual- and mixed-media filters and deep, relatively coarse monomedia to tolerate higher applied turbidities has resulted in several applications where coagulated water is filtered directly without sedimentation.[30–45] Figure 12–14 presents approaches to direct filtration now in use.

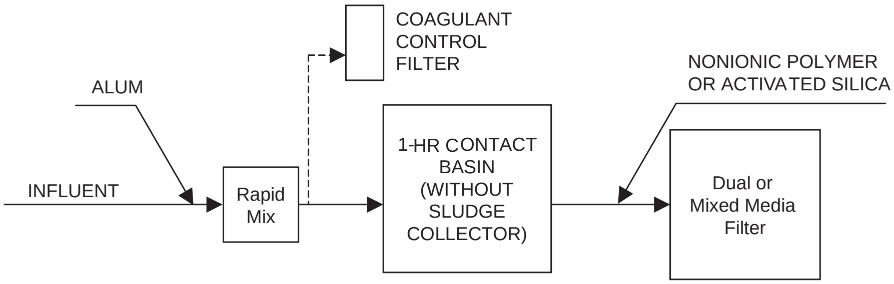
As shown in Figure 12–14, all three direct filtration schemes require some form of pretreatment to ensure that a filterable floc is formed prior to the filters. The direct filtration arrangement shown in Figure 12–14a uses the addition of alum with rapid mixing followed by application of a nonionic polymer or activated silica to the influent



SCHEME A
DIRECT FILTRATION USING ALUM AND NONIONIC POLYMER OR ACTIVATED SILICA



SCHEME B
DIRECT FILTRATION USING FLOCCULATION



SCHEME C
DIRECT FILTRATION USING A CONTACT BASIN

Fig. 12–14. Approaches to using direct filtration (Adapted from *Journal AWWA*, Vol. 69, No. 7 (July 1977), by permission. Copyright © 1977, American Water Works Association.)

of dual- or mixed-media filters. An alternative would replace the alum and nonionic polymer with the use of a cationic polymer in the raw water entering the rapid mix. In some cases, preozonation has been reported to enhance the removal of turbidity in the direct filtration process.[33] Potential mechanisms are microflocculation and the production of more polar compounds.

Figure 12–14b is a direct filtration scheme utilizing a flocculation basin in which the chemical dosage alternates are the same as for Figure 12–14a. The flocculation basin provides additional detention time and mixing to ensure good floc formation on raw waters that are more difficult to treat.

The Figure 12–14c flow scheme is a direct filtration arrangement with a 1-hour contact basin between the rapid mix and the filter. The purpose of the contact basin is to increase the reliability of the process by adding a lead time of 1 hour between turbidity readings, showing the results of coagulation from the coagulant-control filter and the entry of water to the filters. In this process, the purpose of the contact basin is not to provide for settling. There is no flocculation basin, and the contact basin is not equipped with a sludge collector. The coagulant options are the same as for Figure 12–14a.

Following the proper mixing of the coagulant with raw water, a number of complex reactions take place with colloidal turbidity and color. These coagulation reactions take place in less than 1 second. The rapid mixing process for direct filtration usually does not differ from that for conventional plants.

At this point in the process, the particles formed are very small, and the colloids are destabilized. When the destabilized particles collide, they stick together, with the rate of agglomeration of these microscopic destabilized particles to form visible floc depending principally upon the number of opportunities for contact they are afforded. In a still body of water, agglomeration takes place at a slow, almost imperceptible, rate; the rate can be increased by agitation or stirring of the water. In a well-designed flocculator, agglomeration of all particles might be completed in times varying from 5 to 45 minutes, when enough collisions will have occurred for the floc particles to become large enough to settle rapidly.

If settling is omitted from the plant flow scheme, as in a direct filtration plant, and if a properly designed rapid mix is provided, then usually there is no reason to include flocculation in the direct filtration process. Rather than spending money on flocculation, it may be better to improve the rapid mixing, provide finer filter media, or increase the depth of the fine filter media. Water containing the destabilized particles can be taken directly from the rapid-mix basin to a granular filter where contact flocculation takes place as part of the filtration process. The flocculation rate is greatly accelerated because of the tremendous number of opportunities for contact afforded in the passage of the water through the granular bed. The floc particles become attached or absorbed to the surface of the filter grains. The smaller the filter grains, the greater are the opportunities for contact, and the more rapid is the removal of particulate matter. Small filter grains also have a greater surface area per unit volume, which provides more area for attachment of floc particles to the filter grains than is available with larger grains.

This contact and the surface attachment or adsorption of particles to filter grains account for particulate removal beyond that of any simple mechanical straining action of the fine media. The pores of the filter gradually fill with floc as particles are sheared off the surfaces of filter grains. As a filter run progresses, the upper pores of the filter cannot retain any more floc, and the particles move down into the filter to find a resting place. Finally, either the headloss through the bed increases to the point where the filter must be cleaned by backwashing, or, if the floc strength is inadequate, floc particles may appear in the effluent, requiring filter backwashing.

For many raw waters, direct filtration will produce water of a quality equal to that obtained by flocculation, settling, and filtration. The limitation of direct filtration is the inability to handle high concentrations of suspended solids. At some point, the amount of suspended solids will be too high for reasonable filter runs, and settling before filtration will be necessary.

The possibilities of applying direct filtration to municipal plants are good if one of the following conditions exists:

- Raw-water turbidity (NTU) and color (color units) are each less than 25
- Color is low and the maximum turbidity does not exceed 200 NTU
- Turbidity is low and the maximum color does not exceed 100 color units

The presence of paper fiber or of diatoms in excess of 1,000 areal standard units per milliliter (asu/ml) requires that settling be included in the treatment process chain. Diatom levels in excess of 200 asu/ml may require the use of special coarse anthracite on the tope of the bed in order to extend filter runs.

The suitability of a raw water for direct filtration cannot be determined from numerical values alone; such values provide only a preliminary indication. Pilot plant tests must be performed in each case to find out whether or not direct filtration will provide satisfactory treatment under the prevailing local circumstances of raw-water quality.

The chief advantage of direct filtration is the potential for capital cost savings from the elimination of sludge-collecting equipment, settling basin structures, flocculation equipment, and flocculation-basin structures. This cost reduction may make possible the provision of needed filtration for some communities that could not otherwise afford it. Operation and maintenance costs are reduced because there is less equipment to operate and maintain.

With direct filtration there may also be a savings of 10 to 30 percent in chemical costs because generally less alum is required to produce a filterable floc than to produce a settleable floc.[36] The costs for polymer may be greater than in conventional plants, but these higher costs are more than offset by the lower costs for coagulants.

Direct filtration produces less sludge than conventional treatment, and a denser sludge.[36] The collection of waste solids is simplified. Waste solids are all contained in a single stream, the waste filter-backwash water.

Pilot plant tests and plant-operating experience show that high-quality filtrate can be produced in direct filtration at filter rates of 5 to 15 gpm/sq ft (12 to 37 m/h).[36] A usual design rate is 5 gpm/sq ft (12 m/h). This provides for flexibility of operation and affords a margin of safety against variations in raw-water quality. Filter influent and effluent piping should be designed for flows of 10 gpm/sq ft (25 m/h).

BIOLOGICAL FILTRATION

The increased interest in applying ozone to potable waters in the United States has, in turn, increased interest in the use of biological filtration of water supplies. When ozone is applied to water, the size and structure of the organic material in the water are significantly altered. There is typically a dramatic increase in the assimilable (biodegradable) organic compounds (AOC), which is undesirable because it can lead to bacterial regrowth in the distribution system. Also, several ozone by-products, such as formaldehyde and acetaldehyde, have been identified as potential human health concerns and may be regulated in the future. Many of these ozonation by-products can be readily removed by biologically active filters. By-products, such as aldehydes, carboxylic acids, aldo acids, and keto acids, are all relatively easily biodegradable, and removals in excess of 75 percent are normally achieved by biological filtration.[51]

Biologically active filters have been used in Western Europe for many years. Common European practice involves two-stage filtration with the biological filters located downstream of the main filtration process designed for particulate removal. The trend in the United States has been to use a single filter that integrates particulate removal and biological treatment in a single unit. When considering the retrofitting of the biological step into an existing plant, it is often not practical to add another filtration step. Experience has shown that it is possible in a single filter to achieve the needed biological treatment without sacrificing particulate removal effectiveness.[52] This section summarizes the key design considerations in single-stage filtration.

Filter Media The most commonly used media for single-stage filtration are granular activated carbon (GAC)-sand, anthracite-sand, and monomedia beds of GAC, anthracite, or sand. GAC offers some potential advantages in that it offers a macroporous structure and irregular surface for bacterial attachment sites that are more resistant to shear stress. In addition, GAC can remove some contaminants by adsorption, at least until its adsorptive capacity is exhausted. GAC can adsorb some slowly degradable components that can be degraded by the attached bacteria. The micropores of GAC are too small (1–100 nm) to allow bacterial growth within the micropores. As a result, the specific surface area available for biomass attachment might actually be higher in a sand filter than in a GAC filter, because the effective size of the sand is usually smaller than that of the GAC. Whether or not the potential advantages of GAC offset the higher cost must be determined on a case-by-case basis. In several cases, anthracite and GAC filters have been reported to provide similar, average BOM removals.[52] GAC is reported to provide better aldehyde removal at colder water temperatures and provide faster reestablishment of BOM removal after periods when the filters have been out of service. New York City conducted extensive comparative tests of GAC and anthracite for biological filtration following ozone and dissolved air flotation treatment of its Croton Lake Supply.[53] Water temperatures varied from 2°C to 22°C with raw-water total organic carbon (TOC) values of 2–3 mg/L. Both the GAC and anthracite filter media were tested at filter loading rates of 8–20 gpm/sf with depths of 72–96 inches. After months of tests, it was concluded that the differences in finished water quality between the GAC and anthracite were not significant enough to justify the higher cost of the GAC media. Both media produced excellent turbidity and particle removals, with finished water turbidities typically in the 0.05 range and particle counts ($>2\mu$m) of less than 20/ml. Biologically degradable organic carbon (BDOC) and biostability tests of the finished water showed no significant difference between the GAC and anthracite media. An anthracite media of 1.1 mm size, depth of 72 inches, and filtration rate of 12 gpm/sf was selected for the full-scale Croton Lake biological filtration facility.

Contact Time Several studies have shown that contact time affects the removal of BOM within biological filters. Contact time is usually expressed in terms of empty bed contact time (EBCT). Several researchers have shown that it is contact time and not hydraulic loading rate that is the key parameter for BOM removal.[52] BOM removal can be approximated by a first-order model, so increasing EBCT will improve BOM removal but less than proportionately. The incremental benefit of using long EBCT values is small. Acceptable removals of ozonation by-products can occur with EBCTs in the range of 2–4 minutes, although site-specific tests should be conducted to determine the required EBCT. The table below shows the filter depths required to obtain 4 minutes of EBCT at various filter loading rates:

Filter Loading Rate	Filter Depth for 4 minutes EBCT
5 gpms/sf	32 inches
8 gpm/sf	51 inches
12 gpm/sf	77 inches

The design criteria (12 gpm/sf and media depth of 72 inches) for the New York Croton Lake biological filters described in the preceding filter media section is equivalent to an EBCT of 3.75 minutes. The removals of chlorination by-product precursors require considerably longer times.

Temperature Not surprisingly, the removal capabilities of a biological filter are affected by temperature. Several studies have shown that AOC removals increase as temperatures increase and the time to reach steady-state removal is shorter as temperatures increase.[52] As a result, it is important to conduct pilot tests of biological filtration during the periods of coldest water temperature.

Backwashing It appears that biological particles adhere more strongly to filter media than do nonbiological particles. Thus, optimum backwashing for removal of nonbiological material from a filter may not cause an excessive loss of biofilm. This is borne out by experience with biological filters that are washed with or without air scouring and does not show a significant decrease in AOC removal when returned to service after backwashing. One study[54] found no statistical difference in AOC removal between biological filters that were backwashed with air scour and those that were backwashed with water only. Backwashing with chlorinated water is another issue of concern with biological filters. Although use of chlorinated backwash water has been shown to decrease the biological mass in the filter, the capability to remove AOC is not greatly affected when dealing with ozonation by-products.[52] However, one study [54] reported that a biological filter backwashed with chlorinated water produced consistently higher AOC values than one washed with nonchlorinated water. If the objective of biological filtration is the removal of chlorination by-product precursors, which are not as readily degradable as ozonation by-products, the presence of chlorine in the backwash water is more critical. Another study found that it is important to backwash the biological filter before returning it to service if it has been shut down.[55] The decay of organic matter within the filter during the shutdown period will adversely affect the finished water quality unless the filter is backwashed before returning it to service.

Oxidants in Filter Influent The effects of oxidants, such as chlorine, hydrogen peroxide, and ozone, in the filter influent is a function of the type of filter media used. GAC decomposes chlorine and other oxidants; thus, biological activity in biofilters using GAC media is not as sensitive to oxidants in the influent as anthracite filters. However, continual presence of oxidants can cause structural deterioration of the GAC. Hydrogen peroxide is sometimes used to accelerate ozone decay and could appear in the biofilter influent. The continuous presence of 1 mg/L hydrogen peroxide in the influent of anthracite-sand filters has been found not to inhibit the removal of ozone by-products, and the hydrogen peroxide was rapidly destroyed in the filter. Chloramine residuals of about 1 mg/L in biological filter influent using anthracite media has been found to inhibit biological removal of ozonation by-products, perhaps because chloramines are relatively stable.[52]

Pilot Testing As noted in the preceding discussion of biological filters, site-specific tests are needed to determine the proper design criteria for filter media type and EBCT for the specific raw-water characteristics involved. There are several issues to consider when planning and conducting such studies.[52] If GAC is tested, care must be taken that the test duration is long enough to exhaust the adsorptive capacity of the GAC. This will require that several tens of thousands of bed volumes be treated. The adsorption provided by the fresh GAC will mask the long-term results that will be obtained from biological treatment. It is also important to test various media sizes and uniformity. In the case of the New York Croton Lake study, GAC and anthracite media ranging in size from 0.9 to 1.4 mm were tested before the l.1-mm size was selected for the full-scale design. At least one of the pilot filters should be backwashed with nonchlorinated water. Backwashing procedures should be identical to those that will be used in the full-scale facility. The tests must include the coldest water temperatures anticipated for the full-scale facility, as biological filter performance is temperature-sensitive. Oxidants in the pilot filter influent should be minimized.

OTHER TYPES OF FILTERS

Diatomite Filters

Diatomite filters consist of a layer of diatomaceous earth about ⅛ (3 mm) thick supported on a septum or filter element. The thin precoat layer of diatomaceous earth is subject to cracking and must be supplemented by a continuous body feed of diatomite. The problems inherent in maintaining a perfect film of diatomaceous earth between filtered and unfiltered water have restricted the use of diatomite filters for municipal purposes, except under certain favorable conditions.

Municipal experience with turbidity removal has been principally where the turbidity is relatively low and the bacteriological quality is good; thus, little pretreatment is provided. The largest installation is a 20-mgd system in California.

The process can obtain high removals of *Giardia* and *Cryptosporidium* without operator expertise in coagulation. Another advantage of a diatomite filtration plant for potable water is the lower first cost of such a plant. On waters containing low suspended solids, the diatomite filter installation cost should be somewhat lower than the cost of a conventional rapid-sand filtration plant. Diatomite filters will thus find application in potable water treatment under the following conditions:

- In cases where the diatomite plant will be found to produce water at a lower total cost than any practical alternative
- In cases where the lower first cost of a diatomite filter installation may be the major factor in the final choice of the plant
- For emergency or standby service at locations experiencing large seasonal variations in water demand, when the lower first cost of the diatomite filter may prove to be economical

Some of the important operating parameters of diatomite filters have been summarized:[11]

- A precoat of 2.45 to 4.89 lb/sq ft (12 to 25 kg/m^2) is applied to prepare the filter.
- A continuous feed of filter aid as body feed is necessary to prevent the cake from clogging with the particles being filtered out.
- Acceptable cleaning of the filter will maintain at least 95 percent of the septum area available for flow after 100 filter run cycles.
- Because of the precoat, DE filters do not require filter-to-waste upon start-up.
- If the flow to the filter is disrupted, the filter cake drops off the septum. When the filter is restarted, clean diatomite and filtered water should be used to recoat the filter to reduce the potential for passage of pathogens.
- It may be necessary to adjust the body feed rate in proportion to the raw-water turbidity to prevent a short run.
- Although filter runs can be 2 to 4 days long, decomposition of organic matter trapped in the filter cake may necessitate shorter runs to avoid taste and odor problems.
- Vacuum DE filters offer the advantages of not requiring pressure vessels and being visible during backwash; however, they have the disadvantage of an increased potential for release of gases, which can cause shorter runs.
- DE filters can provide very effective removal of cysts, algae, and asbestos. In some cases, alum coating of the DE improves performance.
- The rate of body feed and size of diatomite used are critical variables affecting the length of the run.

Upflow Filters

Upflow filtration has an obvious theoretical advantage, because coarse-to-fine filtration can be achieved with a single medium, such as sand, with almost perfect gradation of both pore space and grain size from coarse to fine in the direction of filtration (upward). Since the bed is backwashed in the same direction but at higher flow rates, the desired relative positions of fine media are maintained or reestablished with each backwash. The inherent advantage of upflow filtration has long been recognized, and, under laboratory conditions, the anticipated high filtration efficiency has been verified by several workers.

The difficulty with upflow filtration comes when the headloss above a given level exceeds the weight of the bed above that level, at which time the bed lifts or partially fluidizes, allowing previously removed solids to escape in the effluent. In Russia, bed depths up to 6 feet (1.82 m) are used in an attempt to minimize bed lifting. In the United States, parallel plates or a metal grid is placed at the top of the fine media. The spacing of the plates or the size of the openings in the grid is such that the media grains arch across the open space to restrain the bed against expansion. These restraining bar systems have about 75 percent open area in the best designs developed to date. Figure 12–15 illustrates an upflow filter with a restraining grid system. Even with the use of a restraining grid or a deep bed, there may be problems with excessive pressures or sudden variations in pressure that break the sand bridge or cause the bed to expand and lose its filter action.

The frequency of breakthrough is rare, but the fact that it can occur at all—say, with poor operation—has been sufficient to raise questions concerning public health implications and to limit the use of upflow filters for potable water applications. In areas that are free of health considerations, upflow filters have found wide application

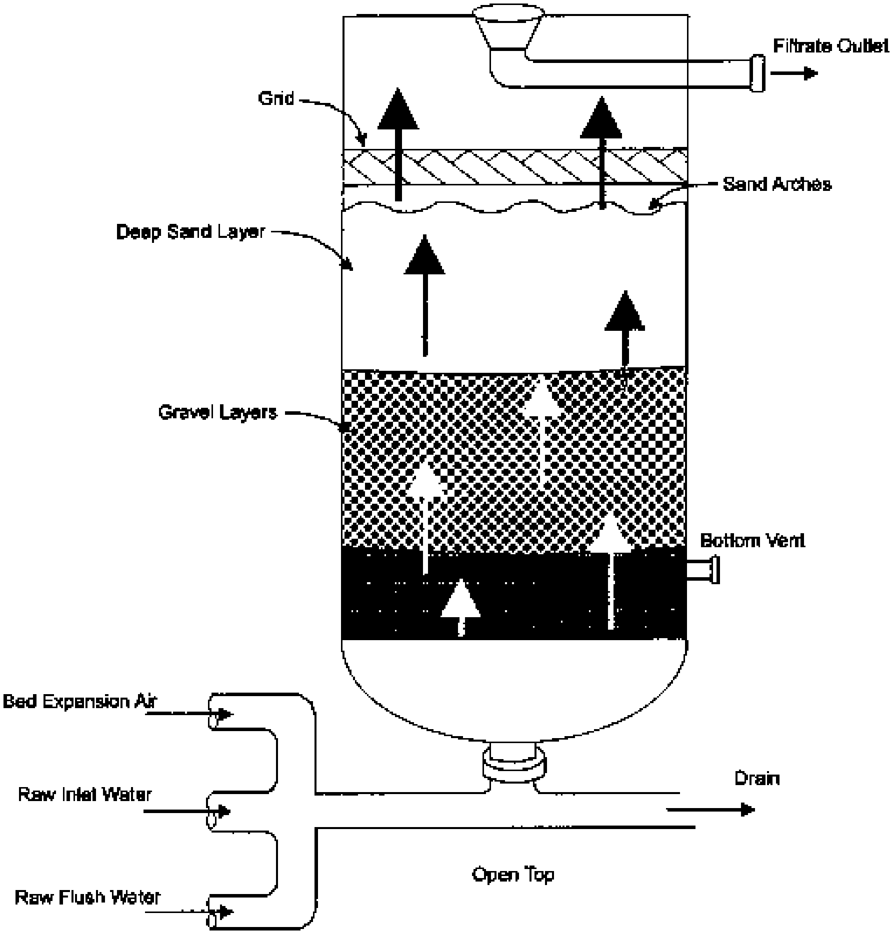


Fig. 12–15. Upflow filter with restraining grid

and have given excellent service. These areas include process water, wastewater treatment, deep-well-injection water, API separator effluent, cooling water, and other similar applications. Until a reliable means of restraining a dirty upflow bed is developed, it appears likely that potable water applications of the upflow filter in the United States will continue to be limited.

Biflow Filters

Biflow filters are an outgrowth of upflow filters, in that the divided flow (downward from the top and upflow from the bottom (see Fig. 12–16) is an attempt to restrain the bottom upflow portion of the bed by placing a downflow filter above it. Biflow filters are used in Holland and Russia, but not to any extent in the United States. They permit filtration in two opposite directions at the same time. Essentially, the top and bottom halves are completely independent filters of equal capacity, which results in some savings in structure and underdrains.

Unfortunately, the biflow filter has an inherent limitation that seems to preclude development of a unit that will produce an exceptionally high-quality effluent. First consider a single-medium biflow bed. The finest materials are at the top of the upper downflow bed. This arrangement makes the top half of the bed a rapid-sand or surface-type filter, and the quality of water produced at best cannot exceed that from a rapid-sand filter. The bottom half of this same filter is a coarse-to-fine filter, but unfortunately the finest material at the top outlet from the bed is somewhat coarser than the finest material that can be successfully used in a rapid-sand filter. Obviously, the effluent from this bed will be of lesser quality than that from the rapid-sand downflow filter above. This situation has been recognized by researchers and revealed by pilot tests,

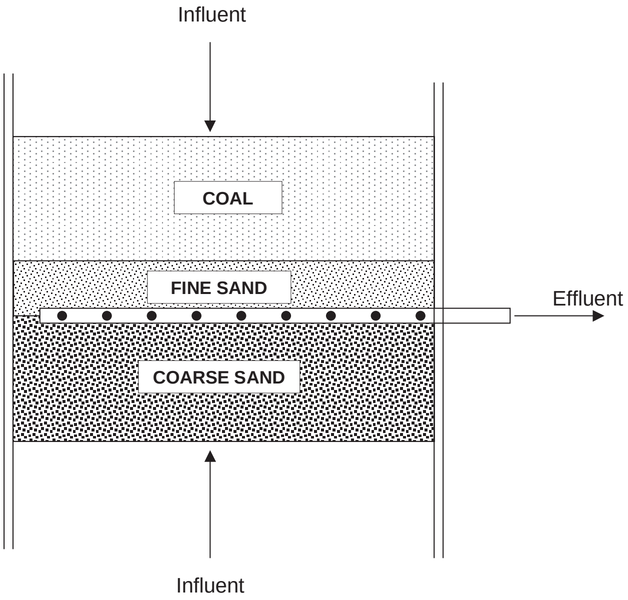


Fig. 12–16. Cross section of dual-media biflow filter

and led to consideration of the dual-media biflow filter, illustrated in Figure 12–16. The idea here, and the advantage over the single-medium biflow bed, is that this arrangement places the fine sand closer to the mid-collector. It provides a dual-media (anthracite-sand) downflow bed above a coarse-to-fine single-medium (sand) upflow bed. Again, there are practical limits on the coarseness of the sand. If the sand is finer than that ordinarily used in a rapid sand filter, as it must be to build the best upflow filter in the bottom half of the bed, then it will be so fine that excessive amounts of it will rise into the anthracite bed during backwashing. The gradation of a sand that is suitable for the dual-media bed in the upper half of the bed will be too coarse to provide the best possible filtration in the upflow bottom half of the bed. The quality of effluent from either half will not approach that from a mixed-media filter. This problem is so basic that there does not appear to be a solution to the dilemma unless resolved by further research and development.

REFERENCES

1. *Water Quality and Treatment. A Handbook of Public Water Supplies,* American Water Works Association, McGraw-Hill, New York, 1999.
2. Conley, W. R., Jr., and Pitman, R. W., ‘‘Innovations in Water Clarification’’ *JAWWA,* p. 1319, 1960.
3. Conley, W. R., Jr., ‘‘Experiences with Anthracite Sand Filters,’’ *JAWWA,* p. 1473, 1961.
4. Rice, A. H., and Conley, W. R., ‘‘The Microfloc Process in Water Treatment,’’ *Tappi,* p. 167A, 1961.
5. Conley, W. R., Jr., ‘‘Integration of the Clarification Process,’’ *JAWWA,* p. 1333, 1965.
6. Pitman, R. W., and Wells, G. W., ‘‘Activated Silica as a Filter Conditioner,’’ *JAWWA,* p. 1167, 1968.
7. Conley, W. R., Jr., and Hsiung, K., ‘‘Design and Application of Multimedia Filters,’’ *JAWWA,* p. 97, 1969.
8. Cleasby, J. L., and Sejkora, G. D., ‘‘Effect of Media Intermixing on Dual Media Filtration,’’ *Journal of the Environmental Engineering Division, ASCE,* p. 503, August 1975.
9. *Mixed Media,* Neptune MicroFloc Bulletin No. 4206.
10. Robeck, G. G., Clarke, N. A., and Dostal, K. A., ‘‘Effectiveness of Water Treatment Processes in Virus Removal,’’ *JAWWA,* p. 1275, 1962.
11. Logsdon, G. S., and Fox, K., ‘‘Getting Your Money’s Worth from Filtration,’’ *JAWWA,* p. 249, May, 1982.
12. Trussell, R. R., et al., ‘‘Recent Developments in Filtration System Design,’’ *JAWWA,* p. 705, December 1980.
13. Culp/Wesner/Culp, *Pilot Filtration Tests at the American River Water Treatment Plant,* prepared for the City of Sacramento, CA, May 1982.
14. Hsiung, A. K., Conley, W. R., and Hansen, S. P., ‘‘The Effect of Media Selection on Filtration Performance,’’ presented at the Hawaii section, American Water Works Association, April 1976.
15. American Water Works Association, *State of the Art of Water Filtration,* Committee Report, p. 662, 1972.
16. Camp, T. R., ‘‘Theory of Water Filtration,’’ *Proceedings American Society of Civil Engineers,* p. 1, 1964.
17. Cleasby, J. L., ‘‘Approaches to a Filterability Index for Granular Filters,’’ *JAWWA,* p. 372, 1969.

18. Cleasby, J. L., Williamson, M. M., and Baumann, R. E., ‘‘Effect of Filtration Rate Changes on Quality,’’ *JAWWA*, p. 869, 1963.
19. Fair, G. M., and Geyer, J. C., *Water Supply and Wastewater Disposal,* John Wiley & Sons, Inc., New York, 1954.
20. Diaper, E. W. J., and Ives, K. J., ‘‘Filtration through Size Graded Media,’’ *J. San. Eng. Div. Proceedings American Society of Civil Engineers*, p. 89, June 1963.
21. Hudson, H. E., Jr., ‘‘Physical Aspects of Filtration,’’ *JAWWA*, p. 33, 1969.
22. Hudson, H. E., Jr., ‘‘Theory of the Functioning of Filters,’’ *JAWWA*, p. 868, 1948.
23. Hudson, H. E., Jr., ‘‘Functional Design of Rapid Sand Filters,’’ *J. San. Eng. Div., ASCE*. p. 17, 1963.
24. Ives, K. J., ‘‘Progress in Filtration,’’ *JAWWA*, p. 1225, 1964.
25. Kawamura, S., ‘‘Design and Operation of High-Rate Filters,’’ *JAWWA*, pp. 535, 653, 705, 1975.
26. Minz, D. M., *Modern Theory of Filtration,* Special Subject No. 10, International Water Supply Assoc., London, 1966.
27. Robeck, G. G., Dostal, K. A., and Woodward, R. L., ‘‘Studies of Modifications in Water Filtration,’’ *JAWWA,* p. 198, 1964.
28. Yao, K. M., Hubibjun, M. T., and O’Melia, C. R., ‘‘Water & Waste Filtration—Concepts and Application,’’ *ES&T,* p. 1105, November 1971.
29. O’Melia, C. R., and Stumm, Werner, ‘‘Theory of Water Filtration,’’ *JAWWA*, p. 1393, 1967.
30. Culp, R. L., ‘‘Direct Filtration,’’ *JAWWA,* p. 375, July 1977.
31. Moulton, F., Bedc, C., and Guthrie, J. L., ‘‘Mixed-Media Filters (Clean) 50 MGD without Clarifiers,’’ *Chemical Processing,* March 1968.
32. Willis, J. F., ‘‘Direct Filtration, an Economic Answer to Water Treatment Needs,’’ *Public Works,* p. 87, November 1972.
33. Tate, C. H., and Trussell, R. R., ‘‘Recent Developments in Direct Filtration,’’ *JAWWA*, p. 165, March 1980.
34. Foley, P.D., ‘‘Experience with Direct Filtration at Ontario’s Lake Huron Treatment Plant,’’ *JAWWA*, p. 162, March 1980.
35. McCormick, R. F., and King, P. H., ‘‘Factors That Affect Use of Direct Filtration in Treating Surface Waters,’’ *JAWWA*, p. 234, May 1982.
36. Wagner, E. G., and Hudson, H. E., Jr., ‘‘Low Dosage High-Rate Direct Filtration,’’ *JAWWA*, p. 256, May 1982.
37. ‘‘The Status of Direct Filtration,’’ *JAWWA*, p. 405, July, 1980.
38. Tate, C. H., Lang, J.S., and Hutchinson, H.L., ‘‘Pilot Plant Tests of Direct Filtration,’’ *JAWWA*, p. 379, July 1977.
39. Logsdon, G. S., Clark, R. M., and Tate, C.H., ‘‘Costs of Direct Filtration to Meet Drinking Water Regulations,’’ *JAWWA*, p. 134, March 1980.
40. Hutchinson, W., and Foley, P. D., ‘‘Operational and Experimental Results of Direct Filtration,’’ *JAWWA*, p. 86, February 1974.
41. Sweeney, G. E., and Prendiville, P. W., ‘‘Direct Filtration: An Economic Answer to a City’s Water Needs,’’ *JAWWA*, p. 65, February 1974.
42. Hutchinson, W. R., ‘‘High-Rate Direct Filtration,’’ *JAWWA*, p. 292, June 1976.
43. Spink, C. M., and Monscvitz, J. T., ‘‘Design and Operation of a 200–mgd Direct-Filtration Facility,’’ *JAWWA*, p. 127, February 1974.
44. Monscvitz, J. T., et al., ‘‘Some Practical Experience in Direct Filtration,’’ *JAWWA*, p. 584, October, 1978.
45. Culp, R. L., ‘‘New Water Treatment Methods Serve Richland,’’ *Public Works,* p. 86, July 1964.

46. Culp, R. L., ‘‘Filtration,’’ Chapter 8, *Water Treatment Plant Design Manual,* American Water Works Association, New York, 1969.
47. Oakley, H. R., and Cripps, T., ‘‘British Practice in the Tertiary Treatment of Sewage,’’ *J.WPCF,* p. 36, 1969.
48. Oakley, H. R., personal communication, 1969.
49. Nelson, O. F., ‘‘Capping Sand Filters,’’ *JAWWA,* p. 539, 1969.
50. Hudson, H. E., *Water Clarification Processes. Practical Design and Evaluation,* Van Nostrand Reinhold, New York, 1981.
51. Carlson, K. H. and Amy, G. L., ‘‘BOM Removal During Biolfiltration,’’ *JAWWA,* p. 43, December 1998.
52. Urfer, D., Huck, P. M., Booth, S. D. J., and Coffey, B. M., ‘‘Biological Filtration for BOM and Particulate Removal: A Critical Review,’’ *JAWWA,* p. 83, December 1997.
53. Progress Reports, Pilot Studies at the Croton Lake Gatehouse, unpublished reports, City of New York, Department of Environmental Protection, Bureau of Environmental Engineering, 1997–1999.
54. Ahmad R., Amirtharajah, A., Al-Shawwa, A., and Huck, P. M., ‘‘Effects of Backwashing on Biological Filters,’’ *JAWWA,* p. 62, December 1988.
55. Niquette, P., Prevost, M., Servais, P., Beaudet, J., Coallier, J. and Lafrance, P., ‘‘Shutdown of BAC Filters: Effects on Water Quality,’’ *JAWWA,* p. 53, December 1998.
56. Kawamura, S., ‘‘Design and Operation of High Rate Filters,’’ *JAWWA,* p. 77, December, 1999.

CHAPTER 13

Water Softening

HARDNESS

Water that has high concentrations of calcium and magnesium is considered to be ‘‘hard.’’ Iron, manganese, and a few others also constitute hardness. Iron, manganese, and the other minor hardness minerals are normally present in such low concentrations compared to calcium and magnesium that they can usually be ignored for hardness calculations.

Hardness reacts with soap compounds to precipitate as a curd, which is a nuisance for household uses such as laundry and bathing. In recent years, synthetic detergents have replaced soaps in many applications. These compounds are not as reactive with hardness; consequently, hardness is less of a problem in the home for these applications. Excessive hardness in water is undesirable for other reasons, since it can cause internal scaling of pipes, water heaters, and plumbing fixtures.

The hardness of water is rated according to Table 13–1.

Hardness is usually expressed in mg/L of calcium carbonate. Another measure of hardness that has been used in the past is grains per gallon (gpg). One gpg equals 17.1 mg/L of calcium carbonate.

There are two types of hardness: carbonate hardness and noncarbonate hardness.

Carbonate Hardness

Carbonate hardness is that fraction of the total hardness that is due to the presence of calcium and magnesium bicarbonates: $Ca(HCO_3)_2$, $Mg(HCO_3)_2$, and carbonates ($CaCO_3$ and $MgCO_3$). When carbon dioxide is driven off, insoluble calcium and magnesium carbonates are formed. Because of this phenomenon, carbonate hardness is sometimes referred to as ‘‘temporary hardness.’’

TABLE 13–1. Hardness Ratings

Rating	Hardness, mg/L as $CaCO_3$
Soft	Less than 50
Moderately hard	50 to 150
Hard	150 to 300
Very hard	More than 300

Noncarbonate Hardness and Salinity

Noncarbonate hardness is that fraction of total hardness associated with noncarbonate calcium sulfate, $CaSO_4$, calcium chloride, $CaCl_2$, magnesium sulfate, $MgSO_4$, and magnesium chloride, $MgCl_2$. The compounds that comprise noncarbonate hardness are not precipitated when the water is heated, but form a very hard scale in boilers when the water is evaporated. The term *incrustants* is sometimes applied to the compounds contributing noncarbonate hardness.

SOFTENING GOALS

General

The primary goals of softening are to produce a water that is relatively stable and soft enough for its intended use. Other goals that can be achieved in lime softening are the reduction of certain metal concentrations, removal of iron and manganese, and reduction of organic compounds in the raw water.

A factor that must be established before a softening project is initiated is the desired level of hardness in the finished water. Local needs and desires are the major consideration, but other factors should also be considered. The perception of acceptable hardness varies considerably from one location to another. In New England, where natural waters are soft, residents might consider a hardness of 100 mg/L excessive, while residents of the Midwest or Southwest with naturally occurring hard water might consider higher levels of hardness to be satisfactory. The cost of water softening is generally not considered justifiable when the source of the water supply is less than 150 mg/L. Public water supplies usually are not softened below 30 to 50 mg/L because:

- The solubility of $CaCO_3$ is 30 mg/L.
- Waters softer than this tend to be corrosive.

One goal of softening should be to reduce the magnesium hardness to less than 40 mg/L, since levels higher than this tend to form magnesium hydroxide scale in domestic water heaters. As a general softening goal guidelines, most lime-soda softening plants are operated to produce the following levels in the softened water:

- Calcium hardness: 30 to 40 mg/L
- Magnesium hardness: less than 40 mg/L
- Total alkalinity: 60 to 70 mg/L

All of the above are expressed as $CaCO_3$.

Health Effects

Some research has indicated that there may be some health benefits associated with the consumption of hard water. Many studies since the 1960s have demonstrated a fairly consistent relationship between soft water and cardiovascular disease. These studies show that people living in soft water areas have somewhat higher cardiovascular disease rates than those living in hard water areas. Although several theories to explain this phenomenon have been advanced, no causative relationships have been established. Other studies have indicated that long-term consumption of demineralized water may result in the lowering of the bone calcium saturation level.[1,2] Again, the causative factors have not been clearly established. These studies suggest that hard water may be healthier than very soft water.

In waters of very high noncarbonate hardness, there has long been a concern that the softening process itself may increase the sodium content of the water enough to have significance for people with high blood pressure. Recent epidemiological studies have raised questions about this previously postulated causative relationship.

Produce Consistent Water Quality

If the quality of the raw water is highly variable, softening of the water to a level that can be maintained year-round may be desirable. Consistency of the quality of the water is often important to consumers, particularly commercial and industrial users. Water that varies in hardness from season to season may result in operational problems.

Produce a Stable Water

Water that is neither excessively corrosive nor excessively scale-forming is ideal for many uses, including domestic uses. The hardness of the water can be an important factor in stability.

Reduce Organic Compound Concentrations

Naturally occurring organic compounds often found in surface waters may combine with chlorine to produce undesirable disinfection by-product such as chloroform and other trihalomethanes. The Stage I Disinfectants/Disinfection By-Products Rule (D/DBP)[3] establishes specific requirements for total organic carbon removal by coagulation and softening processes (see Chapter 1, ‘‘Criteria and Standards for Potable Water Quality’’). Water systems using softening must provide 15–30 percent TOC reduction, depending on the source water TOC. If they cannot achieve these TOC requirements, they may use the following additional criteria for compliance: reduce treated water alkalinity to less than 50 mg/L (annual average), or use softening that removes at least 10 mg/L of magnesium hardness (annual average). A later section of this chapter discusses removal of organics in the softening process.

Reduce Heavy Metals

Lime softening can reduce the concentrations of many heavy metals. Those that can be reduced by 80 to 95 percent include barium, strontium, silver, arsenic, cadmium, cobalt, copper, mercury, nickel, lead, and zinc. Hexavalent chromium is difficult to precipitate; it can be reduced by only 20 to 30 percent. If in the trivalent form, precipitation is much enhanced.

Lime softening is very efficient for arsenic removal [As(V)] ($>$95 percent removal) at pH above 10.5.[4] Softening is not as successful with trivalent arsenic [As(III)]. Lime softening above pH 11 can remove up to 80 percent of As(III).[5] At pH 11, almost 100 percent of the As(III) is present as the anion H_2AsO_3 instead of H_3AsO_3. Increased removal of As(III) can be accomplished by oxidation of the As(III) to As(V) prior to the treatment. Chlorine or potassium permanganate will easily do this.

Reduce Radionuclides

Many radioactive metals can be significantly reduced by precipitation with lime. Radium is a naturally occurring radionuclide often found in groundwater; its concentration can be reduced by 75 to 95 percent by lime softening. Uranium and thorium are also precipitated by lime softening.

Fission products such as Sr-90 were commonly found in surface waters during the period of atmospheric nuclear weapons testing. Its average concentration in surface waters now is a small fraction of the levels occurring during the 1960s. It can be further reduced by approximately 60 percent by lime softening. Other radionuclides that have been found in surface water include Cr-51, Mn-56, and Ba1-30, all of which have been reduced by 60 to 90 percent in lime-softening plants.

Pathogen Inactivation

Lime coagulation has been demonstrated to be capable of effectively removing and inactivating viruses at high pH values. The mechanism of inactivation under alkaline conditions is probably caused by denaturation of the protein coat and by disruption of the virus. In some cases complete loss of structural integrity of the virus may occur under high pH conditions. The pH reached in lime coagulation is a critical factor in determining the degree of virus inactivation, with inactivation rates increasing dramatically as pH increases from 10.8 to 11.1.[6] Similar effects have been noted for *E. coli* and *Salmonella typhosa*. However, it has been reported[7] that little or no inactivation of *Giardia* was caused by high pH at contact times up to six hours.

LIME-SODA SOFTENING

The most common method of softening municipal and large industrial water supplies is lime–soda softening. The lime-soda process is a precipitative softening process in which the soluble hardness components are first converted to insoluble precipitates, then removed from the water by settling and filtration.

Chemical Reactions in the Lime-Soda Process

To remove carbonate hardness, lime, either as $Ca(OH)_2$ (hydrated lime, slaked lime), is used in small plants; or CaO (quick lime, pebble lime) is used in large plants. Quicklime is cheaper than hydrated lime, but it must be slaked to hydrated lime in the plant before it is added to the water. The lime reacts with free carbon dioxide (if any) and calcium bicarbonate in the water to form calcium carbonate, and with magnesium bicarbonate to form magnesium hydroxide. These compounds are quite insoluble, and will precipitate. Lime will also react with the noncarbonate hardness compounds magnesium sulfate and magnesium chloride to form magnesium hydroxide plus calcium sulfate and calcium chloride. Although magnesium hydroxide precipitates, no hardness is removed in these reactions, since calcium sulfate and calcium chloride are soluble. The reactions involved are as follows:

$$CO_2 + Ca(OH)_2 = CaCO_3\downarrow + H_2O \tag{13–1}$$

$$Ca(HCO_3)_2 + Ca(OH)_2 = 2CaCO_3\downarrow + 2H_2O \tag{13–2}$$

$$Mg(HCO_3)_2 + Ca(OH)_2 = MgCO_3 + CaCO_3 + 2H_2O \tag{13–3}$$

$$MgCO_3 + Ca(OH)_2 = Mg(OH)_2\downarrow + CaCO_3\downarrow \tag{13–4}$$

$$MgSO_4 + Ca(OH)_2 = CaSO_4 + Mg(OH)_2\downarrow \tag{13–5}$$

$$MgCl_2 + Ca(OH)_2 = Mg(OH)_2\downarrow + CaCl_2 \tag{13–6}$$

If it is necessary to reduce the noncarbonate hardness to achieve the desired degree of softening, soda ash (sodium carbonate) is added to the water. This reacts with the calcium sulfate and calcium chloride to form calcium carbonate, which precipitates. The reactions are as follows:

$$CaSO_4 + Na_2CO_3 = CaCO_3\downarrow + Na_2SO_4 \tag{13–7}$$

$$CaCl_2 + Na_2CO_3 = CaCO_3\downarrow + 2NaCl \tag{13–8}$$

In estimating the amounts of chemicals to be added, the assumption is made that the reactions are complete. Where magnesium hardness is to be removed, the amount of lime applied must be sufficient not only to convert the free carbon dioxide and bicarbonates to carbonate, but also to produce magnesium hydroxide by adding the required excess of hydroxide. The pH generally must exceed 10.5 to precipitate magnesium hydroxide. The amounts can be calculated stoichiometrically by using the above equations, or it can be estimated by the following formulae:

For carbonate hardness reduction:

$$\begin{aligned}&\text{Lime required in lb CaO per MG}\\&\quad = 10.6 \times (\text{mg/L } CO_2) + 4.7 \times (\text{mg/L alkalinity}\\&\quad + \text{mg/L Mg hardness} + \text{excess OH alkalinity})\end{aligned} \tag{13–9}$$

The CaO and the CO_2 are expressed as the actual chemicals; the other factors in the above equation are expressed as $CaCO_3$. The amount of excess lime required is determined by the amount of magnesium hardness to be removed. In practice, 20 to 50 mg/L (as $CaCO_3$) is normally required to precipitate the magnesium hardness. The lower values apply when a coagulant such as alum is used in conjunction with the lime.

For noncarbonate hardness reduction:

$$\begin{aligned}&\text{Soda ash required in lb } Na_2CO_3 \text{ per MG}\\&\quad = 9 \times (\text{noncarbonate hardness})\\&\quad\ \text{minus (noncarbonate hardness to remain in the water)}\end{aligned} \tag{13–10}$$

This equation states that the pure soda ash required, expressed as itself, in pounds per million gallons softened, is equal to the amount required to precipitate the desired amount of noncarbonate hardness, expressed as $CaCO_3$.

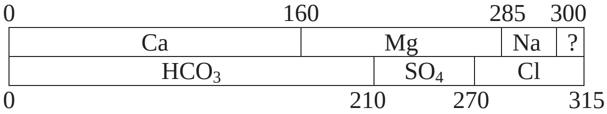
The chemicals are added to the water, normally through a rapid-mix process, then the precipitates are flocculated, settled, and removed as sludge. The addition of 1 pound (0.45 kg) of lime to hard water can lead to the precipitation of up to 3½ pounds (1.59 kg) of mineral solids. The flocculation process may be aided by the use of a coagulant such as aluminum or iron salts, and/or polymer.

To determine how the chemical makeup of the water can be altered by lime–soda softening, it is often useful to prepare a bar chart of the cations and anions in the raw water. Since hardness and alkalinity are always expressed as mg/L $CaCO_3$, it is convenient to express all the other components this way also. Table 13–2 lists conversion factors that can be applied to express several common substances as $CaCO_3$.

Figure 13–1 is a graphical representation of the ion balance of an actual well water that would be amenable to softening. In preparing such a chart, the cations are plotted

TABLE 13-2. $CaCO_3$ Equivalent of Common Substances

Substance	Formula	Molecular Weight	Equivalent Weight	$CaCO_3$ Conv. Factor Multiply by:
Aluminum	Al^{+3}	27.0	9.0	5.56
Aluminum hydroxide	$Al(OH)_3$	78.0	26.0	1.92
Aluminum sulfate (filter alum)	$Al_2(SO_4)_3 \cdot 18H_2O$	666.4	111.1	0.45
Bicarbonate	HCO_3	61.0	61.0	0.82
Calcium	Ca^{+2}	40.1	20.0	2.50
Calcium bicarbonate	$Ca(HCO_3)_2$	162.1	81.1	0.62
Calcium carbonate	$CaCO_3$	100.1	50.0	1.00
Calcium chloride	$CaCl_2$	111.0	55.5	0.90
Calcium hydroxide	$Ca(OH)_2$	74.1	37.0	1.35
Calcium oxide	CaO	56.1	28.0	1.78
Calcium sulfate	$CaSO_4$	136.1	68.1	0.73
Carbon dioxide	CO_2	44.0	22.0	2.27
Carbonate	CO_3^{-2}	60.0	30.0	1.67
Chloride	Cl^-	35.5	35.5	1.41
Ferric chloride	$FeCl_3$	162.2	54.1	0.93
Ferric iron	Fe^{+3}	55.8	18.6	2.69
Ferric sulfate	$Fe_2(SO_4)_3$	399.9	66.6	0.75
Ferrous iron	Fe^{+2}	55.8	27.9	1.79
Magnesium	Mg^{+2}	24.3	12.2	4.12
Magnesium bicarbonate	$Mg(HCO_3)_2$	136.4	73.2	0.68
Magnesium carbonate	$MgCO_3$	84.3	42.2	1.18
Magnesium hydroxide	$Mg(OH)_2$	58.3	29.2	1.71
Magnesium sulfate	$MgSO_4$	120.4	60.2	0.83
Manganese	Mn^{+2}	54.9	27.5	1.82
Sodium	Na^+	23.0	23.0	2.18
Sodium carbonate	Na_2CO_3	106.0	53.0	0.94
Sodium chloride	$NaCl$	58.5	58.5	0.85
Sodium sulfate	Na_2SO_4	132.1	71.0	0.70
Sulfate	SO_4^{-2}	96.1	48.0	1.04



Concentrations are in mg/L as $CaCO_3$

Fig. 13–1. Ion balance-water amenable to softening

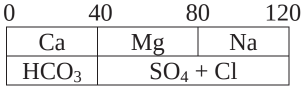
graphically on the top bar, in the following order: Ca, Mg, Fe and Mn, Na. The anions are plotted on the bottom bar, in the following order: HCO_3, CO_3, SO_4, Cl. These will represent the normal mineral constituents of most natural waters. The cations in the top bar can be paired with the anions in the bottom to approximate the actual chemical compounds making up the minerals dissolved in the water. In this example, they are:

$Ca(HCO_3)_2$	160 mg/L as $CaCO_3$
$Mg(HCO_3)_2$	50
$MgSO_4$	60
$MgCl_2$	15
NaCl	15
Unidentified	15
Total	315 mg/L as $CaCO_3$

In the above example and subsequent examples, a question mark indicates that the anions and cations didn't exactly balance, and is assumed to represent the presence of unidentified anions or cations.

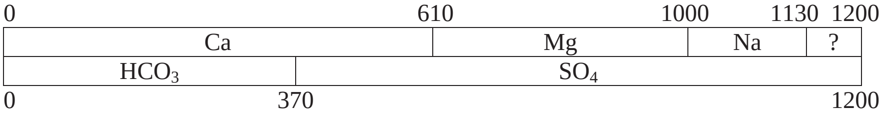
Figure 13–2 shows what the chemical makeup of the same water might be after softening, assuming that the calcium and magnesium have each been reduced to 40 mg/L. Note that the reduction of sulfates and chlorides by 25 mg/L (as $CaCO_3$) has resulted in a corresponding increase in sodium of 25 mg/L (as $CaCO_3$) through the addition of soda ash.

Figure 13–3 is an example of a water that will not benefit very much from lime–soda softening. In this case, the sodium and sulfate concentrations are so high that softening will not yield very acceptable finished water quality. Softening to reduce the sulfate to 40 mg/L (as $CaCO_3$) would require a dose of soda ash sufficient to raise the sodium content of the water by 590 mg/L (as $CaCO_3$). Figure 13–4 represents the ion balance of this water as it might be after lime–soda softening. In this example,



Concentrations are in mg/L as $CaCO_3$

Fig. 13–2. Ion balance after softening



Concentrations are in mg/L as $CaCO_3$

Fig. 13-3. Water with limited softening benefit

the hardness has been reduced to 120 mg/L. The initial sodium plus the added sodium has resulted in a total sodium content of 720 mg/L as $CaCO_3$, or 330 mg/L as sodium. The sulfate is unchanged at 830 mg/L as $CaCO_3$, or 798 mg/L as sulfate. The water is relatively soft, but its total mineral content is still very high. To produce a water of overall better quality, other processes such as reverse osmosis should be considered.

Pretreatment

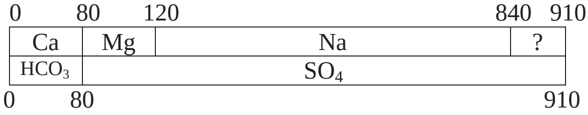
Prior to softening, some pretreatment may be advisable if any of the following conditions exist:

- Raw water turbidities exceed 1,000 NTU at times.
- Raw water has a high concentration of free carbon dioxide (more than 10 mg/L). This much CO_2 is usually found only in ground water. CO_2 can be removed by stripping.
- The raw water is high in organic colloids of a type that impedes crystallization of calcium carbonate.
- Raw water quality is highly variable over short periods of time.
- Recalcining of sludge is to be practiced, in which case the sludge purity is important.

If none of the above is true, the clarification and softening process trains can usually be combined. Basically, the applicable design standards for mixing, flocculation, and sedimentation are the same for the lime-soda process as for conventional clarification.

Use of Caustic Soda

Caustic soda (NaOH) can be used as the primary softening chemical in place of both lime and soda ash. The reactions of caustic soda with carbon dioxide, and carbonate and noncarbonate hardness, are:



Concentrations are in mg/L as $CaCO_3$

Fig. 13-4. Ion balance after softening

$$CO_2 + 2NaOH = Na_2CO_3 + H_2O \tag{13–11}$$

$$Ca(HCO_3)_2 + 2NaOH = CaCO_3\downarrow + Na_2CO_3 + 2H_2O \tag{13–12}$$

$$Mg(HCO_3)_2 + 4NaOH = Mg(OH)_2\downarrow + 2Na_2CO_3 + 2H_2O \tag{13–13}$$

$$MgSO_4 + 2NaOH = Mg(OH)_2\downarrow + Na_2SO_4 \tag{13–14}$$

Caustic soda removes the free carbon dioxide and carbonate hardness, producing calcium carbonate, which precipitates, and soluble sodium carbonate. It also reacts with noncarbonate hardness, producing magnesium hydroxide, which precipitates. The above equations demonstrate that caustic soda can remove both carbonate and noncarbonate hardness. It can not only take the place of soda ash, but can satisfy all or a part of the lime requirement as well. The advisability of using caustic soda as a softening chemical depends on the comparative costs of lime, soda ash, and caustic soda in the area. Usually caustic soda will not be very competitive.

Organic Removal

Lime–soda softening can also remove natural organic matter (NOM). The presence of NOM is important because NOM reacts with disinfectants to form disinfection byproducts (DBPs), which may be harmful to health and which are regulated by federal drinking water standards.

NOM can be divided into hydrophobic and hydrophilic fractions, and this distribution influences NOM removal by softening. Lime softening removes a higher percentage of hydrophobic organic carbon than hydrophilic organic carbon based on bench-scale[8] and full-scale[9] studies. These studies and others have also shown that the high-molecular-weight fraction of NOM is preferentially removed by lime softening.

Magnesium precipitation in the softening process enhances the removal of NOM.[10] Under the pH conditions encountered during lime–soda softening, $CaCO_3$ precipitates as a dense, negatively charged crystalline solid with a low surface area, making it a poor adsorbent for negatively charged NOM. By contrast, $Mg(OH)_2$ is a positively charged gelatinous precipitate with an amorphous structure and a high surface area, making it an excellent adsorbent for NOM. Systems that remove at least 10 mg/L of Mg hardness as $CaCO_3$ are exempt from the enhanced softening requirements of the D/DBP rule. An extensive study of NOM removal was conducted in a series of bench-scale tests conducted at nine U.S. water treatment plants using lime softening.[10] The study found that:

- Softening with lime and soda ash, followed by settling and filtration, can remove modest amounts of DOC from most raw waters, depending on the softening pH. Removal increases as the amount of $Mg(OH)_2$ precipitated increases. A greater amount of DOC can be removed by lime–soda softening for waters with high initial DOC concentrations.
- For the enhanced softening requirements of the D/DBP Rule, for five of the nine water studies, the removal of 10 mg/L of Mg hardness was achieved with lower chemical dosages than those needed to meet the percent TOC removal requirements. The application of a hydrolyzing metal coagulant to assist in solid-liquid separation can offset the need for higher Mg removal.

- The precipitation of $Mg(OH)_2$ greatly enhances the removal of DOC. The amount of DOC removed per unit increase in pH was found to increase considerably at the pH level at which Mg precipitation dramatically increased (pH 10.5–10.8). The amount of DOC removal shows a strong correlation with the amount of Mg precipitated.
- DOC removal by softening was most effective for the raw waters with the highest percentage of hydrophobic organic carbon, and was least effective for the raw waters with the lowest percentage of hydrophobic organic carbon. The Specific UV Absorbance (SUVA) value of the raw water was a good quantitative predictor of the effectiveness of DOC removal by softening.

PROCESS DESIGN CONSIDERATIONS

Process Flow Patterns

Lime–soda softening plants may be divided into four major categories with respect to process flow patterns. As shown in Figure 13–5, they are:

- Single-stage softening
- Split treatment
- Two-stage softening
- Three-stage softening with recovery and reuse of lime by recalcination

Figure 13–5 also shows a number of pretreatment options and variations as previously mentioned. The designer must select from these available processes and treatment elements the particular combination that best suits local raw water conditions in order to develop the optimum softening plant.

Although it is not shown in Figure 13–5, it is possible for waters with limited variations in temperature and physical and chemical quality, which are to be treated at a constant plant throughput rate, to be softened in solids contact basins. In this type of basin, the elements of rapid mixing, flocculation, settling, and contact with previously precipitated calcium carbonate are combined and carried out within a single structure. The potential cost advantages of softening in solids contact basins may be lost if there are variations in water temperature, flow, or quality that require separate individual control.

Each of the above softening flow train options is discussed briefly in the following paragraphs.

Single-Stage Softening This is the simplest and most common flow train. All of the softening is done in one stage of treatment. The water leaving the settling basin is very unstable, and will precipitate scale on all surfaces it contacts, so the softening must be followed by stabilization. This is normally done by adding carbon dioxide to the water (recarbonation).

Split Treatment In this flow train, only a portion of the flow is softened. The balance, usually 20 to 40 percent, is bypassed and serves to stabilize the water by reacting with the excess lime in the softened flow. If additional stabilization is required, it may be accomplished by recarbonation as shown in Figure 13–5.

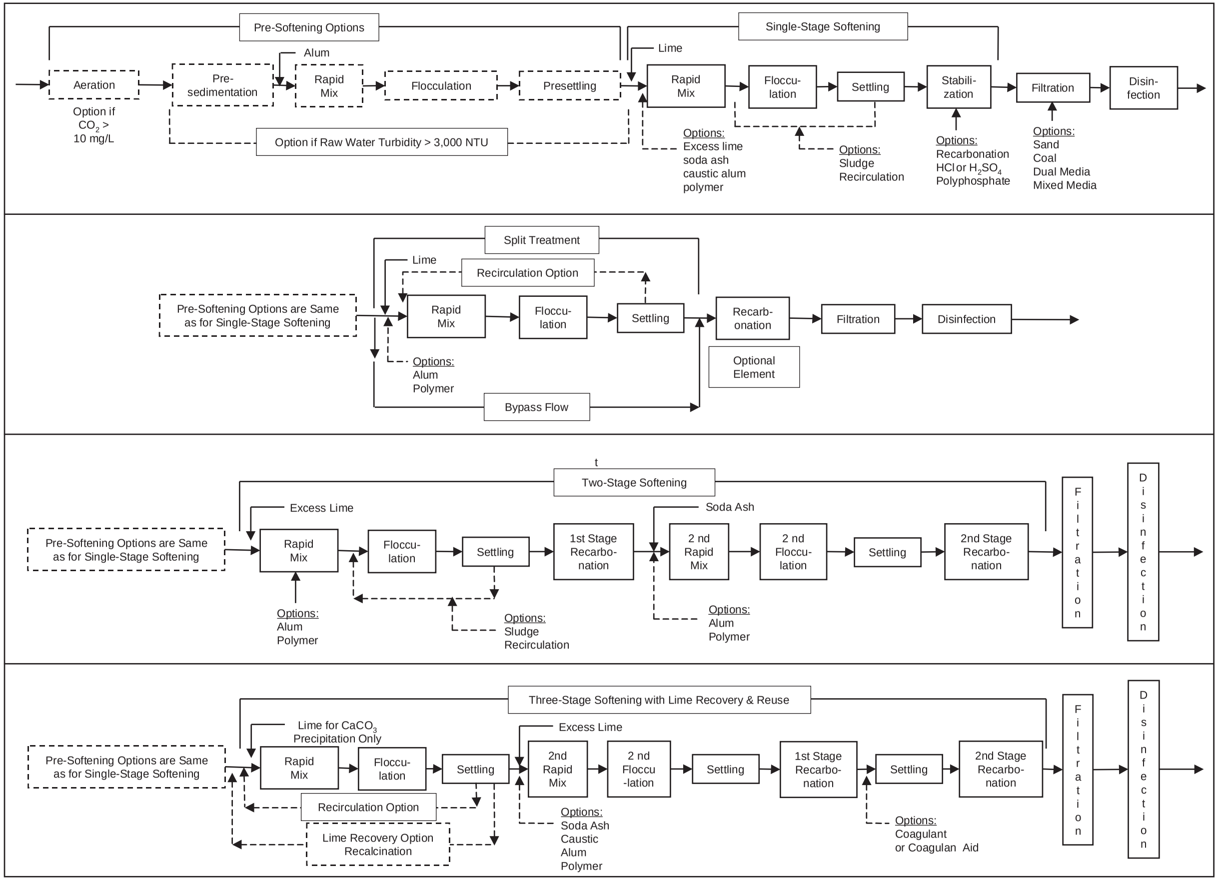


Fig. 13–5. Softening flow patterns (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Two-Stage Softening In this process variation, the soda ash is added in a separate step following excess-lime softening and recarbonation. This may be advantageous in water with a lot of noncarbonate hardness to be removed, especially if there is an unusually high concentration of fluoride or silica. In excess-lime treatment, the magnesium hydroxide floc will adsorb negatively-charged ions such as fluoride and silica. This will usually reduce the noncarbonate hardness. Settling this floc before adding soda ash will have the effect of reducing the soda ash dosage required. Because of the number of additional treatment units required and the consequent high capital cost, this process is not very commonly used.

Three-Stage Softening with Lime Recovery and Reuse Lime sludge can be recalcined, breaking down the $CaCO_3$ to CaO and CO_2, which can then be reused in the softening and recarbonation processes. When the sludge contains $Mg(OH)_2$ as well, the recalcination converts it to MgO, which has no softening value and will build up in the recalcined product with each recycle. This can be avoided by adding just enough lime in the first stage to precipitate only the $CaCO_3$, which can then be recalcined. In the second stage, chemicals are added to precipitate the magnesium and the noncarbonate hardness. The sludge from this stage is wasted. The third stage of softening is the first stage of recarbonation. This process is discussed in the recarbonation section.

Residual Design Considerations

The softening process design should also consider various chemical feed systems, sludge production, and lime handling systems. These auxilliary sytems are described below.

Coagulants Calcium carbonate produced in the softening process is a fine precipitate, which can be difficult to settle. Magnesium hydroxide is a gelatinous floc that can serve as a coagulant if enough is precipitated in softening. It is common to use an added coagulant in the softening process such as alum (aluminum sulfate), ferric chloride, and ferric sulfate to produce a more settleable floc. Polymers are also often used as an aid to coagulation and flocculation.

Lime Feed Lime feeding systems are described in Chapter 24, ‘‘Chemical Storage and Feeding Systems.’’ A trough with an easy-to-remove top is preferable to a pipe for conveying lime slurries. If a trough cannot be used, a rubber hose that is easily accessible for pressing or breaking down the scale in it is preferable to a pipe. Lime in slurries tends to settle. It creates blockages in pipes that are extremely difficult to remove.

Sludge Produced Lime-soda softening produces large quantities of sludge. The amount of solids produced in the process can be estimated by the following formula:[11]

$$S = 8.34 \times Q \times [2(\mathrm{Ca}) + 2.6\,(\mathrm{Mg}) + 0.44(\mathrm{Alum}) + 2.9(\mathrm{Fe}) + \mathrm{SS} + \mathrm{P}] \quad (13\text{–}15)$$

where:

Q = Flow in MGD
Ca = Calcium hardness removed, mg/L as $CaCO_3$

Mg = Magnesium hardness removed, mg/L as $CaCO_3$
Alum = Alum dose, mg/L
Fe = Iron dose, mg/L
SS = Suspended solids in the influent water, mg/L
P = Polymer (or other coagulant aid) dose, mg/L
S = Sludge produced, lb dry solids per day

Lime Recalcining and Reuse Lime recalcining has been used to some extent in water works practice. Long used in industrial applications, it has also been employed in water and wastewater treatment facilities that use lime treatment. The recalcining process is simple, and consists of heating the calcium carbonate sludge to about 1,850° F (1,000°C). This drives off water and carbon dioxide, leaving only the calcium oxide (quicklime) plus some inert material. The off-gases can be used in recarbonation. Recalcining can be done by multiple-hearth furnace, fluid-bed rectors, or rotary kilns. Rotary kilns are used only in very large recalcining operations. Before sludge is recalcined, some pretreatment is required to reduce the water content. This normally includes gravity thickening and dewatering by centrifuge, filter press, or some other means that will produce a cake of satisfactory dryness.

Buildup of inerts can reduce the available lime in the recycled product to a point where it is no longer useful as a softening chemical. To minimize this problem, some lime sludge may be wasted periodically. Removal of lighter solids such as metal hydroxides prior to recalcining will also help. This can be done by pretreating the sludge prior to dewatering with a centrifuge designed for classification (rather than concentration), which will separate the lighter solids from the sludge stream.

In plants treating waters high in calcium hardness, an excess of recalcined lime may be produced. In small water treatment plants (5 to 10 mgd), the cost of reclaimed lime may not be less than the cost of new lime, but when the costs associated with sludge disposal are included in the calculation, recalcining may be economical. As sludge disposal costs rise, recalcining may become an attractive option.

UNIT PROCESS DESIGN CRITERIA FOR LIME SOFTENING

Mixing, Flocculation, and Sedimentation

Two types of softening process trains are described below: conventional mixing, flocculation sedimentation; and solids contact. In addition, laboratory requirements are presented.

In lime–soda softening the applicable design standards are similar to those for conventional clarification. These criteria are outlined below, and are described in more detail in other chapters.

Mixing

Rapid Mixing Rapid mixing provides the rapid dispersal of chemicals throughout the water to be treated, by violent agitation. Mechanical mixers, hydraulic jump, and Parshall flume are recommended methods. Baffled mixing chambers are acceptable only when provisions are made for proper mixing under anticipated variations in flow.

Detention The detention period should be not less than 30 seconds in mechanical mixing chambers. In-line mixers need less detention time.

Flocculation

Basin Design Inlet and outlet design should prevent short-circuiting and destruction of floc. Series compartments are recommended to prevent short-circuiting and to provide decreasing mixing energy with time. A drain should be provided.

Mean Velocity Gradient (G) G Values of 20 to 70 sec^{-1} are recommended. Flocculation in three basins in series is suggested, with G values of 50 to 100 sec^{-1}, 5 to 50 sec^{-1}, and 1 to 10 sec^{-1} in basins 1, 2, and 3, respectively.

Equipment Agitators should be driven by variable-speed drives with the peripheral speed of paddles ranging from 0.5 to 2.0 ft/sec (0.15 to 0.60 m/s). External, nonsubmerged drive equipment is preferred. Cross-flow, axial-flow, propeller, or turbine-type equipment is acceptable.

Piping Flocculation and sedimentation basins should be as close together as possible. The velocity of flocculated water through pipes or conduits to settling basins should be not less than 0.5 or greater than 1.5 ft/sec (0.15 to 0.46 m/s). Allowances should be made to minimize turbulence at bends and changes in direction. Lines with high-pH water are subject to encrustation and ready access for line cleaning should be provided.

Sedimentation The requirements for effective clarification are dependent upon a number of factors related to basin design and the nature of the raw water (turbidity, color and colloidal matter, taste- and odor-causing compounds, or other material to be removed). Also to be considered is the character of floc formed.

Inlet Devices Inlets should be designed to distribute the water equally and at uniform velocities. Open ports, submerged ports, and similar entrance arrangements are recommended. A baffle should be constructed across the basin close to the inlet and should project several feet below the water surface to dissipate inlet velocities and provide uniform flows across the basin.

Outlet Devices Outlet devices should be designed to maintain velocities suitable for settling in the basin and to minimize short-circuiting. The use of submerged orifices is recommended in order to make the volume above the orifices available for storage of any excess water pumped to the plant beyond that passed through the filters.

Overflow Rate The recommended maximum surface overflow rate is 600 gpd/sq ft (1.02 m/h), and the rate of flow over the outlet weir should not exceed 20,000 gpd/ft (250 m^3/d/m) of weir length. Where submerged ports are used as an alternate for overflow weirs, they should be no lower than 3 feet (0.91 m) below the flow line. For softening, detention times of 2 hours or more are recommended.

Drainage Basins should be provided with a means for dewatering. Basin bottoms should slope toward the drain at a slope of not less than 1 in 12 where mechanical sludge collection equipment is not used. Minimum diameter of the drain line is 8 inches (200 mm).

Overflow An overflow weir (or pipe) should be installed that will establish the maximum water level desired on top of the filters. It should discharge with a free fall at a location where the discharge will be seen.

Inlet-to-Outlet Distance The minimum horizontal distance from basin inlet to outlet should be 10 feet (3.0 m).

Safety Permanent ladders or handholds should be provided for safety on the inside walls of basins above the water level. Guardrails should be included. Flushing lines or hydrants should not include interconnection of the potable water with nonpotable water.

Sludge Collection Mechanical sludge collection equipment should be provided.

Sludge Disposal Provision should be made for the operator to observe or sample sludge being withdrawn from the unit.

Solids Contact Units

Units are acceptable for combined softening and clarification where water characteristics are not variable, and flow rates are uniform. The units should be designed for the maximum uniform rate and should be adjustable to changes in flow that are less than the design rate and for changes in water characteristics. A minimum of two units is recommended.

Installation of Equipment A manufacturer’s representative should supervise installation and initial operation of all mechanical equipment. There should be adequate piping with suitable sampling taps so located as to permit the collection of samples of water from critical portions of the units.

Chemical Feed Chemicals should be applied at such points and by such means as to ensure satisfactory mixing of the chemicals with the water.

Mixing Mixing devices employed should be so constructed as to:

- Provide good mixing of the raw water with previously formed sludge particles.
- Prevent deposition of solids in the mixing zone.

Flocculation Flocculation equipment should have the following features:

- Be adjustable for varying energy input.
- Provide for coagulation to occur in a separate chamber or baffled zone within the unit.
- Provide a flocculation and mixing period of not less than 30 minutes.

Sludge Concentrators and Recirculation The equipment should provide either internal or external recirculation and concentrators in order to obtain a concentrated sludge with a minimum of wastewater.

Sludge Removal Sludge removal design should provide that:

- Sludge pipes are not less than 3 inches (75 mm) in diameter and are arranged to facilitate cleaning.
- The entrance to sludge withdrawal piping prevents clogging.
- Valves are located outside the tank for accessibility.
- The operator may observe or sample sludge being withdrawn from the unit.

Cross-Connections Cross-connections must be avoided, and the following should be considered during design:

- Blow-off outlets and drains should terminate at locations that eliminate the possibility of a cross-connection.
- Cross-connection control should be included for the potable water lines used to backflush sludge lines.

Upflow Rates Unless supporting data are available to justify other rates, clarifier upflow rates should not exceed:

- 1.75 gpm/sq ft (4.3 m/h) of area at the slurry separation line, for units used for softeners.

Laboratory Control

Chemical precipitation plants for water softening should provide an electric pH meter; equipment for determining alkalinity, hardness, temperature, and residual disinfectant; and a continuously recording turbidimeter on each filter effluent line. Particle counters may also provide added information of value in controlling filtered water turbidity.

It is also highly desirable to furnish equipment for coagulation control. Pilot filters are preferred, but jar test apparatus, zeta potential meters, streaming current detectors, and colloid titration apparatus may be useful in some situations.

RECARBONATION

Recarbonation is a unit water treatment process that has been in use for many years in numerous municipal and industrial lime–soda softening plants throughout the world. Recarbonation refers to the addition of carbon dioxide to a lime-treated water. When carbon dioxide is added to high-pH, lime-treated water, the pH is lowered, and hydroxides are reconverted to carbonates and bicarbonates. The term *recarbonation* is descriptive of the result of adding carbon dioxide to water.

The purpose of recarbonation is the downward adjustment of the pH of the water to place the water in calcium carbonate equilibrium to avoid problems of deposition of calcium scale. Carbon dioxide is added to the water ahead of the filters in order to avoid coating the grains of filter media with calcium carbonate, which would eventually increase the grain size to the point that filter efficiency would be reduced. It is also important to lower the pH of the lime-treated water to the point of calcium carbonate stability to avoid deposition of calcium carbonate in pipelines.

Lime treatment of waters for softening can raise the pH to a value of 10.4 or more. Primary recarbonation is used to reduce the pH to 9.3, which is near the pH of minimum solubility for calcium carbonate. Primary recarbonation to pH 9.3 results in the formation of a floc that is principally calcium carbonate. If sufficient reaction time, usually about 15 minutes in cold water, is allowed for the primary recarbonation reaction to go to completion, the calcium carbonate floc does not redissolve with subsequent further lowering of pH in secondary recarbonation. However, there is a tendency for the magnesium salts to do so. If lime is to be reclaimed by recalcining and reuse, this settled primary recarbonation floc is a rich source of calcium oxide, and may represent as much as one-third of the total recoverable lime.

The second stage of recarbonation, to pH 7, is beneficial in several ways. It:

- Prepares the water for filtration.
- Lowers the pH to a value that increases the efficiency of carbon adsorption of organics.
- Is an excellent range for effective disinfection by chlorine.
- Stabilizes the water with respect to scale formation in pipelines.

If the pH were not reduced to less than about 8.8 before application to the filters and carbon beds, extensive deposition of calcium carbonate would occur on the surface of the grains. This could reduce filter efficiency, and could also drastically reduce the adsorptive capacity of granular activated carbon for organics. It would produce rapid ash buildup in the carbon pores upon regeneration of the carbon, and would lead to early replacement of the carbon.

SINGLE-STAGE VS. TWO-STAGE RECARBONATION

The pH of a treated water can be reduced from 11 to 7 or to any other desired value in one stage of recarbonation. Single-stage recarbonation eliminates the need for the intermediate settling basin used with two-stage systems. However, by applying sufficient carbon dioxide in one step for the total pH reduction little, if any, calcium is precipitated, with the bulk of calcium remaining in solution, thus increasing the calcium hardness of the finished water. In addition, this causes the loss of a large quantity of calcium carbonate, which could otherwise be settled out, recalcined to lime, and reused. If lime is to be reclaimed or if calcium reduction in the effluent is desired, then two-stage recarbonation is required. Otherwise, single-stage recarbonation may be used, with some savings in initial cost and some reduction in the amount of lime sludge to be handled.

SOURCES OF CARBON DIOXIDE

In some water treatment plants, carbon dioxide for recarbonation may be obtained from stack gas from a nearby incinerator or from a nearby incinerator of from in-plant recalcination furnaces. Other sources include the use of commercial liquid carbon dioxide; or the burning of natural gas, propane, butane, kerosene, fuel oil, or coke.

The stack gas from an incineration furnace that is fired with natural gas will contain about 16 percent carbon dioxide on a wet basis or about 10 percent on a dry basis, the value usually used for design purposes. The burning of 1,000 cubic feet (28.3 m^3) of natural gas produces about 115 pounds (52.2 kg) of CO_2.

The stack gas from a lime recalcining furnace contains not only the CO_2 produced by combustion of the fuel, but also the CO_2 driven off of the calcium carbonate sludge in the recalcining process. For design purposes, a value of 16 percent CO_2 in lime furnace stack gas is a conservative figure.

Kerosene and No. 2 fuel oil will yield about 20 pounds (9 kg) of CO_2 per pound of fuel. Coke will produce approximately 3 pounds (1.4 kg) of CO_2 per pound of coke burned. Commercial liquid CO_2 is 99.5 percent CO_2.

QUANTITIES OF CARBON DIOXIDE REQUIRED

In recarbonation, one molecule of CO_2 is required to convert one molecule of calcium hydroxide (caustic alkalinity) to calcium carbonate. In addition, it takes one molecule of CO_2 to convert one molecule of calcium carbonate to calcium bicarbonate. It follows, then, that two molecules of CO_2 are required to convert one molecule of calcium hydroxide to calcium bicarbonate. These reactions are represented by the following equations:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \tag{13–16}$$

$$CaCO_3 + CO_2 + H_2O \rightarrow Ca(HCO_3)_2 \tag{13–17}$$

Because all forms of alkalinity are expressed in terms of calcium carbonate (molecular weight = 100), the calculations are as follows:

1. Calcium hydroxide to calcium carbonate:

   (molecular weight of CO_2 = 44, and 1 mg/L = 8.34 lb/MG)

$$\begin{aligned} CO_2 \text{ (in lb/MG)} &= \frac{44}{100} \times 8.34 \\ &\quad \times \text{(Hydroxide Alkalinity in mg/L as } CaCO_3) \\ &= 3.7 \times \text{(Hydroxide Alkalinity in mg/L as } CaCO_3) \end{aligned}$$

2. Calcium carbonate to calcium bicarbonate:

$$\begin{aligned} CO_2 \text{ (in lb/MG)} &= \frac{44}{100} \times 8.33 \times \text{(Carbonate Alkalinity in mg/1 as } CaCO_3) \\ &= 3.7 \times \text{(Carbonate Alkalinity in mg/1 as } CaCO_3) \end{aligned}$$

3. Then, for calcium hydroxide to calcium bicarbonate:

$$CO_2 \text{ (lb/MG)} = 7.4 \times \text{(Hydroxide Alkalinity in mg/1 as } CaCO_3)$$

Sample Calculations for the Amount of CO_2 Required

Assume that 400 mg/L of calcium oxide (CaO) has been added to a sample of water and that the stirred and decanted liquor has the following characteristics:

pH = 11.7

Alkalinities (in mg/L as $CaCO_3$): Hydroxide (OH^-) = 380

Carbonate (CO_3^{-2}) = 120

Bicarbonate (HCO_3^-) = 0

After first recarbonation in the laboratory using bottled carbon dioxide, analysis of the lime-treated water shows the following:

pH = 9.3

Alkalinities (in mg/L as $CaCO_3$): OH^- = 0

CO_3^{-2} = 180

HCO_3 = 380

Then, after secondary recarbonation in the laboratory, the water has the following analysis:

pH = 8.3

Alkalinities (in mg/L as $CaCO_3$): OH^- = 0

CO_3^{-2} = 0

HCO_3^- = 750

To change all caustic alkalinity and all carbonate alkalinity to bicarbonates, the amount of CO_2 required is as calculated below:

$$7.4 \times 380 = 2{,}812 \text{ lb/MG (334 kg/ML) of } CO_2 \tag{13–18}$$

$$3.7 \times 120 = \underline{444 \text{ lb/MG (53 kg/ML)}}$$

$$\text{Total} = 3{,}256 \text{ lb/MG (387 kg/Ml) of } CO_2$$

For a 1-mgd (3.8-ML/d) flow, 3,256 pounds (1,478 kg) of CO_2 per day are required. If it is assumed that the CO_2 content of the stack gas to be used for recarbonation is 10 percent, then (100/10) × 3,256 = 32,560 pounds (14,780 kg) of stack gas must be compressed in order to supply the necessary CO_2 to recarbonate 1 mgd (3.8 ML/d) of water.

At standard conditions of 14.7 psia (101.28 kPa) and 60°F (15.5°C), assume that the density of the stack gas is the same as for CO_2, or 0.116 lb/cu ft (1.88 kg/m^3). Then:

$$\frac{32{,}560 \text{ lb}}{0.116 \text{ lb/cu ft}} = 280{,}700 \text{ cu ft/day} \tag{13–19}$$

$$= 195 \text{ cu ft/min } (5.52 \text{ m}^3\text{/min})$$

If it as assumed that the stack gas is cooled in scrubbing to 110°F (43.3°C), then the temperature correction is:

$$\frac{110 + 460}{60 + 460} \times 195 = 1.1 \times 195 \tag{13–20}$$

$$= 214 \text{ cu ft/min/mgd } (1 \text{ m}^3\text{/min/ML/d})$$

With the same gas temperature, for a plant at 6,300 feet above sea level (11.6 psia), the altitude correction is:

$$\frac{14.7}{11.6} \times 215 = 1.26 \times 215 \tag{13–21}$$

$$= 270 \text{ cu ft/min/mgd } (1.26 \text{ m}^3\text{/min/ML/d})$$

Because part of the CO_2 is not absorbed in the water but escapes at the water surface, it is customary to add about 20 percent to the theoretical requirements. If this is done, then at sea level 260 cu ft/min/mgd (1.21 m^3/min/ML/d) of blower or compressor capacity is required, and at 6,300 ft (1,920 m) of altitude 325 cu ft/min/mgd (1.52 m^3/min/ML/d) of plant capacity is needed.

These calculations are based on the following assumptions:

- The water to be recarbonated has a pH = 11.7 with OH^- alkalinity = 380 mg/L as $CaCO_3$ and with a CO_3^{-2} alkalinity of 120 mg/L as $CaCO_3$, all of which is to be converted to bicarbonates.
- There is 10 percent CO_2 in the gas.
- The flue gas temperature is 110°F (43.3°C).
- An excess of 20 percent is added to the theoretical values to allow for absorption losses.

Conditions at each installation undoubtedly will differ from the assumptions used here, and calculations must be based on actual values.

In water-softening plants that add lime to pH = 10.6, the amount of CO_2 required will be much less than in the example above.

NONPRESSURIZED CARBON DIOXIDE GENERATORS

If stack gas from a furnace operating at atmospheric pressure is to be used as a source of CO_2, the gas should be passed through a wet scrubber. Wet scrubbers provide contact between the gas and a flow of scrubbing water. Particulate matter is removed from the gas, and the gas is cooled. Wet scrubbers may be one of three general types: impingement, Venturi, or surface area. Water jet impingement-type scrubbers are efficient in removing potential air pollutants from the exhaust gas, and provide some protection of the CO_2 compression equipment against plugging or scaling by particulates. The scrubbers cool the hot stack gas down to about 110°F (43.3°C).

When stack gas alone is used as the source of CO_2, the stack gas supply must exceed the maximum demands for CO_2. With this situation, control of the amount of CO_2 applied to the water is simple. Air may be admitted through a valve into the suction line leading to the compressor as required to reduce the amount of CO_2 to that desired. Alternatively, part of the compressed gas may be bled off to the atmosphere through a valve in the compressor discharge line. As another method of control, compressed gas may be recirculated from the compressor discharge line back to the suction line through a bypass line and control valve. However, this method has the serious disadvantage of warming the gas by compression, and excessive recirculation can lead to compressor damage by overheating or increased corrosion at the elevated temperatures.

COMPRESSOR SELECTION

Even with thorough scrubbing, stack gas from incineration furnaces of lime recalcining furnaces will contain sufficient particulate matter to cause plugging and seizure problems in some types of blowers and compressors, particularly those with limited clearance between moving metal parts. This problem is less severe with stack gas from atmospheric furnaces, which burn fuel primarily for production of CO_2.

Water-sealed compressors similar to wet vacuum pumps are a good selection for handling dirty, corrosive gases. This type of compressor consists of a squirrel-cage-type rotor that revolves in a circular casing containing water. This is a simple, reliable piece of equipment with only one moving part. It has increased capacity when handling hot, saturated vapors because the vapors are condensed by the cool liquid compressant. The water-sealed compressor is a relatively quiet-running unit, free from pulsations and vibrations.

If the CO_2 distribution grids are submerged a minimum of 8 feet in water as they usually are, the CO_2 compressor must deliver against a differential pressure across the machine of about 6 to 8 psi (41.3 to 55.12 kPa). The exact rating must be determined by calculation, taking into account not only the depth of submergence of the distribution piping but also orifice losses and pipe friction losses. This is discussed in more detail later because it is common to all types of CO_2 systems. The compressors may be of cast-iron construction, or may be supplied with a bronze rotor and cones at considerable extra cost.

The following accessories are commonly required with water-sealed compressor units: water separator with gauge glass and bronze float valve; discharge check valve; expansion joints for inlet and outlet piping; water seal supply line with adjusting cock and orifice union; water line strainer; inlet water spray nozzles; and sealing water line solenoid valve. In addition, the discharge line is usually fitted with an automatic pressure relief valve and a bleed-off valve, both of which would discharge to the free atmosphere. It is not good practice to install shutoff or isolation valves on either the compressor suction or discharge lines because of the possibility of serious damage to the compressor or pipelines in the event that the compressor is operated in error with one or both of the valves closed.

In selecting CO_2 compressor units to meet total capacity requirements, it is a good idea, except in very small installations, to provide at least three compressor units. By properly sizing them, it is then possible to satisfy two needs: to secure a range in output and to provide standby service. If it is assumed that the total CO_2 capacity required is 1,500 cu ft/min (42 m^3/min), then units with individual capacities of 500 cu ft/min (14.15 m^5/min), 1,000 cu ft/min (28.3 m^3/min), and 1,500 cu ft/min (42.45 m^3/min) would be a good choice. This combination gives a range of 500 to 1,500 cu ft/min (14.15 to 42.45 m^3/min) to match plant needs, and it supplies standby with the largest unit out of service by using the two smaller units together.

Figure 13–6 illustrates a typical recarbonation system using stack gas at atmospheric pressure. As indicated in this figure, automatic pH control of the recarbonated effluent can be provided by continuously monitoring an effluent sample for pH. Changes in pH will operate the controller, which in turn positions a bleed-off valve in the CO_2 compressor discharge line to limit the amount of CO_2 to that necessary to maintain the desired pH.

PRESSURE GENERATORS AND UNDERWATER BURNERS

Generators designed specifically for the production of carbon dioxide for recarbonation are usually either pressure generators or submerged underwater burners. Most early installations were of the atmospheric type, in which the fuel is burned at atmospheric pressure and the off-gas is scrubbed and compressed. These systems are expensive to maintain because of the corrosive effects of the hot, moist combustion gases, and atmospheric generators have largely been replaced by pressure generators and underwater burners, except where waste stack gas is available from another source.

Pressure or forced draft generators produce CO_2 by burning natural gas, fuel oil, or other fuels in a pressure chamber. The fuel and excess air are first compressed and injected, and then burned at a pressure that is sufficiently high to allow discharge directly into the water to be recarbonated. The compressors handle only dry gas or dry air at ambient temperatures, and thus the corrosion problems involved in handling the hot, moist stack gases are avoided. One difficulty with this type of pressure generator is its limited capacity range, which may be 3 to 1 or, at best, 5 to 1. This low turndown ratio may necessitate the installation of two or more units in order to secure the required flexibility and process control. A wide range of sizes is commercially available in pressure CO_2 generators. This commercial equipment is well designed and reliable, and includes all auxiliaries and safety controls.

Submerged combustion of natural gas is another method of CO_2 generation. A unit is shown schematically in Figure 13–7. Air and natural gas are compressed and then burned under water at the point of application; that is, in the recarbonation basin. Automatic underwater electric ignition equipment is used to start combustion. Submerged combustion is a simple, efficient means of CO_2 generation that provides good control of recarbonation and requires a minimum of maintenance. The turndown ratio of this type of burner is only about 2 to 1, so it is necessary to provide enough burner assemblies to obtain the desired range of control.

LIQUID CARBON DIOXIDE

Commercial liquid CO_2 has found increasing use for recarbonation in water-softening plants primarily because of its steadily decreasing cost. However, the price of liquid CO_2 depends greatly on the distance from the source of supply, and the first factor to be investigated is the cost of liquid CO_2 delivered to the plant under consideration.

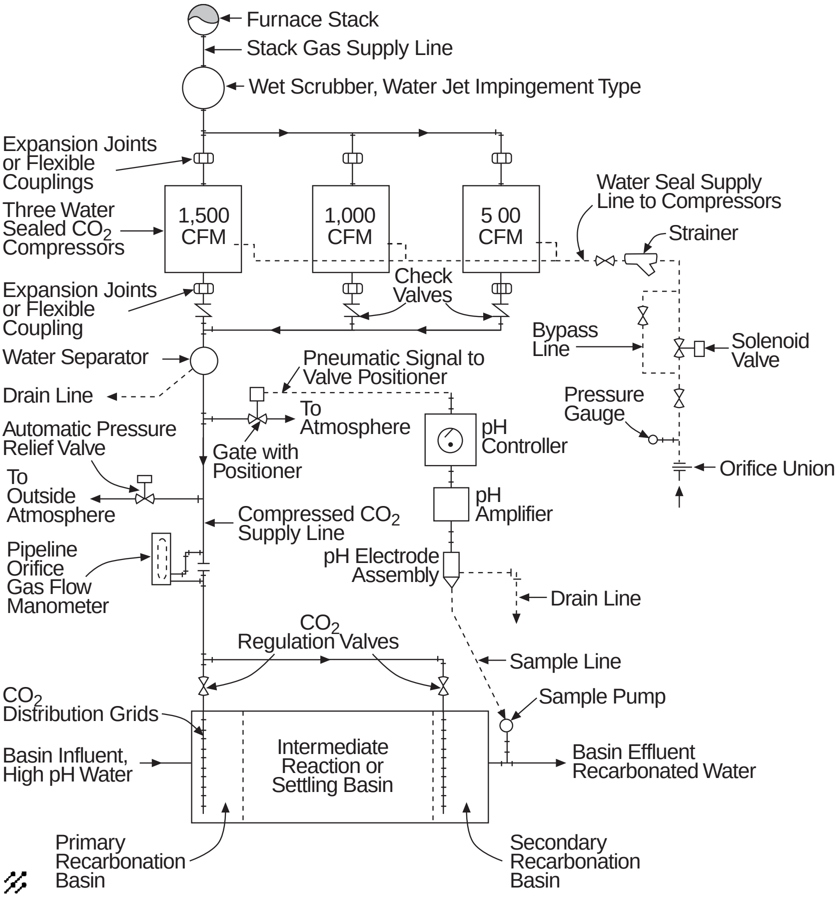


Fig. 13-6. Typical recarbonation system using stack gas (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Even in favorable locations, high cost is still the principal disadvantage of using liquid CO_2. Its advantages include flexibility, ease of control, high purity and efficiency, and the smaller piping required because of its high CO_2 content, 99.5 percent, relative to the 6 to 18 percent obtained from other sources.

Liquid CO_2 may be delivered to customers in insulated tank trucks ranging from 10 to 20 tons (9 to 18 metric tons) in capacity. Rail car shipments of 30 to 40 tons (27 to 36 metric tons) are available to large-volume users. Some manufacturers will lease tank cars so that they may be used for storage at the site, thus eliminating the need and expense of bulk storage tanks and auxiliaries at the plant. For small plants, liquid CO_2 is also available in 20- to 50-lb (9- to 23-kg) cylinders.

Bulk storage tanks may be purchased or leased. Capacities range from 1 to 100 tons (0.91 to 90.7 metric tons), although the common sizes are 4 to 48 tons (3.6 to 43.5 metric tons). Storage tanks must be insulated and equipped with Freon refrigeration and electric or steam vaporization equipment. The working pressure for storage

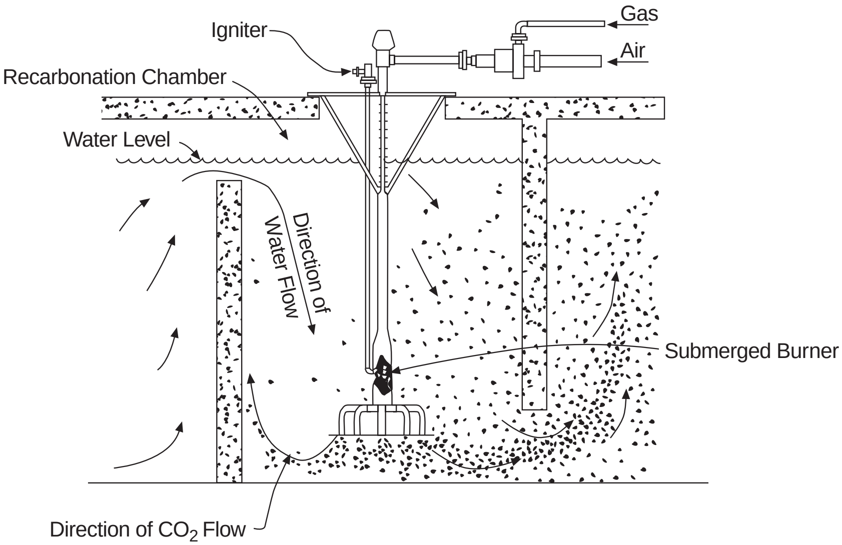


Fig. 13-7. Submerged gas burner schematic (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

tanks is 350 psi (2,400 kPa), and the ASME Code for Unfired Pressure Vessels requires hydrostatic testing to 525 psi (3,600 kPa), or 1.5 times the working pressure. The tanks may be insulated with pressed cork or polyurethane foam. The cooling and vaporizing systems are designed to maintain the liquid CO_2 at about 0°F (−17.8°C) and 300 psi (2,000 kPa). If temperature and pressure rise, the cooling system comes on; and with falling pressure, the vaporizer comes into service. In the event either of these systems fails, or in the event of fire or other accident, the storage tanks are fitted with high and low pressure alarms, two safety pop valves, a manual bleeder relief valve, and a bursting disk.

Either liquid or gas feed systems may be used to apply the liquid CO_2 to the water to be treated. As CO_2 is withdrawn from the storage tank, the pressure is reduced. This pressure reduction cools the CO_2, with the danger of dry ice formation if the expansion is too rapid. Consequently, it is common practice to reduce the pressure in two stages, from the 300 psi (2,000 kPa) tank pressure to the 20 psi (140 kPa) pressure ordinarily required for feeding the CO_2. Vapor heaters may also be used just ahead of the pressure-reducing valves.

For CO_2 gas feed, an orifice plate in the feed line with a simple manometer may be used to measure flow, and a manual valve installed downstream may be used to regulate or control the amount of CO_2 applied. Automatic control to a manual set point could be provided by using a differential pressure transmitter on the feed line orifice, and connecting it to an indicating controller that would operate the control valve. Optionally, the CO_2 feed could be made fully automatic by providing pH control. In this case, an electrode would be installed to measure the pH of the recarbonated water. This signal would be amplified and sent through a controller that would throttle the control valve on the feed line at low pH and open it at high, as set on the controller.

For solution feed of CO_2, equipment similar (except for materials of construction) to solution feed chlorinators may be used. Chlorinator capacity is reduced about 25 percent when feeding CO_2. Approximately 60 gallons of water is required to dissolve 1 pound (0.454 kg) of CO_2 at room temperature and atmospheric pressure. Absorption efficiency with solution feed of CO_2 approaches 100 percent.

CARBON DIOXIDE PIPING AND DIFFUSION SYSTEMS

Because the gas temperature in recarbonation systems is usually in the 70° to 100°F (21.1 to 38°C) range, the pressure is about 6 to 8 psi (40 to 55 kPa). And because CO_2 pipe runs are usually less than 100 feet (30 m), it is convenient to use Table 13–3 to estimate the pipe size required. The pipe sizes obtained from Table 13–3 are, of course, an approximation. For greater accuracy, or for long lines, the following modification of the Darcy-Weisbach formula may be used for pressure loss in air piping:

$$\Delta p = \frac{f}{38{,}000}\,\frac{LTQ^2}{pD^5} \qquad (13\text{–}22)$$

where:

delta p =pressure drop, psi (kPa)

$f = \dfrac{0.048D^{0.027}}{Q^{0.148}}$ (Note: usual values for $f = 0.016$–0.049)
L = pipe length, feet (m)
T = absolute temperature of the gas, °R (°F + 460)
Q = gas flow, cu ft/min (m^3/min)
p = absolute pressure of the gas, psi (kPa) (or line pressure in psi + 14.7)
D = pipe diameter, inches (cm)

The pressure loss in elbows and tees can be approximated by use of the following formulas:

TABLE 13-3. Approximate Gas-Carrying Capacity

Pipe Diameter		Capacity	
Inches	cm	cu ft/min	m^3/min
1	2.54	45	1.27
2	5.08	250	7.1
3	7.02	685	19.4
4	10.16	1,410	39.9
6	15.24	3,870	109.5

$$L = \frac{7.6D}{1 + \frac{3.6}{D}} \tag{13–23}$$

where

L = equivalent length of straight pipe, feet (m)
D = pipe diameter, inches (cm)

The loss in globe valves is about:

$$L = \frac{11.4D}{1 + \frac{3.6}{D}} \tag{13–24}$$

where L and D are as defined above.

Carbon dioxide absorption systems often consist of a grid of perforated pipe submerged in the water. The recommended minimum depth of submergence is 8 feet (2.5 m). With lesser depths of submergence some undissolved CO_2 will escape at the water surface. Properly designed absorption systems will put into solution 85 to 100 percent of the applied CO_2. PVC pipe is suitable for the perforated CO_2 grid pipes. Current practice is to use $\frac{3}{16}$-inch (5 mm)-diameter orifices drilled in the bottom of the pipe at an angle of 30 degrees to the right of the vertical centerline, then 30 degrees to the left, alternating at a spacing of about 3 inches (75 mm) along the centerline of the pipe. Another arrangement is to point the orifices straight up at the top of the pipe and to direct a jet of water from a header down at the CO_2 orifice, in order to form fine bubbles of the gas, which dissolves more readily in the water than larger bubbles. Because PVC does not corrode under acidic conditions, the openings are not subject to plugging as they are in metal pipes.

If $\frac{3}{16}$-inch (0.48-cm) orifices are used, each opening is often rated at 1.1 to 1.65 cu ft/min (0.03 to 0.05 m^3/min), which corresponds to headlosses through the orifice of 3 and 8 inches (75 to 200 mm) of water column, respectively. This is sufficient loss through the orifice to ensure good distribution of the CO_2 to each opening. Carbon dioxide laterals must be laid with the same depth of submergence on each orifice. If the size of the pipe changes, then eccentric reducers should be used to keep the bottom of the pipe level (assuming that the holes are in the bottom of the pipe). Horizontal spacing between CO_2 diffusion laterals should be at least 1.5 feet (0.46 m) in order to get good absorption. To convey cool, dry CO_2, plain steel or cast-iron pipe may be used; but for hot, moist CO_2 gas, the use of stainless steel or other acid-resistant metal is suggested. Special pipe is also required to convey liquid CO_2 in water; a 1.5-inch (38.0-mm) cotton fabric hose with openings of controlled size or porosity has been used successfully. Basin hydraulics must take into account raised water levels caused by CO_2 injection.

CARBON DIOXIDE REACTION BASINS OR INTERMEDIATE SETTLING BASINS

Contrary to many early reports in the literature, the recarbonation reaction is not instantaneous. Although about 90 percent of the applied CO_2 does dissolve in its very short upward journey from the distribution grid through 8 feet (2.5 m) of water to the water surface, the time for complete reaction between the dissolved CO_2 and hydroxide and carbonate ions may be as great as 15 minutes in cold water. In the primary phase of two-stage recarbonation, if the reaction if allowed to go to completion at a pH near 9.3, the calcium carbonate formed is not redissolved in the second phase of recarbonation to a low pH—say, to pH 7.0. Magnesium salts do tend to redissolve under these conditions. In the recarbonation of water, a floc is formed following first-stage recarbonation. This is a rich source of calcium carbonate from which lime (CaO) can be reclaimed and reused by recalcining at temperatures of about 1,850°F (1,000°C). In this case, then, it is desirable to allow not only for reaction time (15 minutes) but for enough time to provide some separation of the calcium carbonate by settling. This will require a settling basin with at least 30-minute detention at maximum flow rate, and a basin surface overflow rate of not more than 2,400 gal/sq ft/day (4 m/h) at peak flow rates. This intermediate settling basin should be fitted with continuous mechanical sludge removal equipment. Figure 13–8 shows a two-stage recarbonation system with intermediate reaction and settling.

Single-stage recarbonation systems should be followed by 15 minutes of detention for completion of the chemical reactions, but no provisions for settling or sludge collection are required. The light, cloudy floc that may be formed at times with single-stage recarbonation is removed quite readily by mixed-media filtration with little effect on filter effluent turbidity, headloss, or length of filter run.

Generally, recarbonated lime-treated water should not be applied directly to beds of granular activated carbon without filtration. Even at low pH (say 7.0), there can be

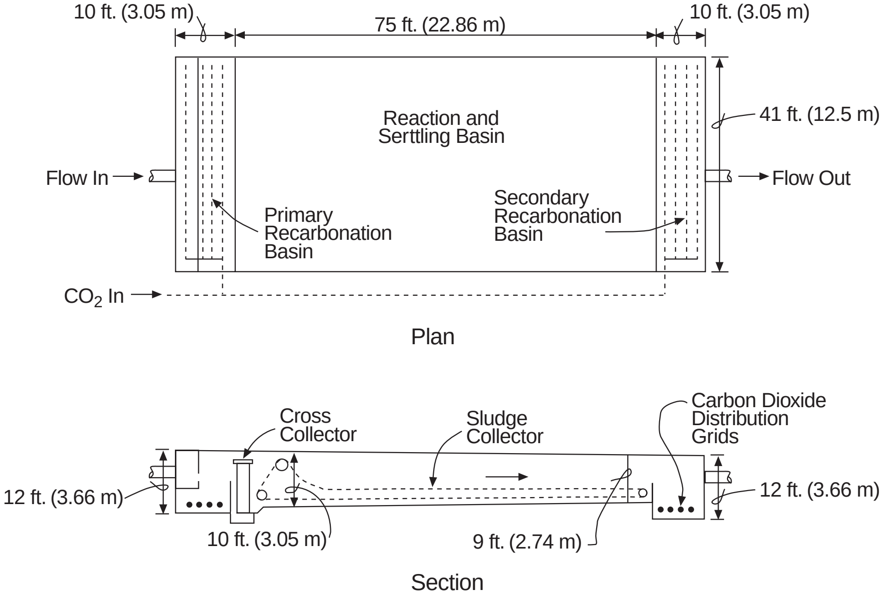


Fig. 13-8. Two-stage recarbonation basin (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

sufficient deposition of calcium carbonate to cause serious problems in the carbon treatment, which can easily be avoided by prior filtration.

A more recent technology has been introduced which uses carbonic acid (formed by aspirating CO_2 into a water via a Venturi-type eductor) to form a liquid-to-liquid transfer of chemical. This system is reported to increase the adsorption efficiency by 10–20 percent and can be diffused into a pipeline, trough, tank, or any location that has a minimum of 18 inches of submergence. The manufacturer indicates the retention is also reduced to less than 10 minutes. This can be particularly valuable in reducing costs associated with plant upgrades, expansions, or new construction.

OPERATION AND CONTROL OF RECARBONATION

The operation and control of recarbonation systems is easy and simple. Automated control systems ordinarily use a single point of pH measurement following the last stage of recarbonation as the basis of control. In two-stage recarbonation systems, the split of total CO_2 flow between the two stages of treatment is fairly constant once it is established for a given flow and the particular set of pH values desired, and control based on the final pH alone is satisfactory without readjustment of the valves supplying the first- and second-stage CO_2 supply headers.

An indirect but more sensitive control of recarbonation is provided by alkalinity measurements. Continuous reliable automatic monitoring and control equipment is available for either the pH or the alkalinity method, but the alkalinity measuring equipment is considerably more expensive than the pH equipment. Manual control is also quite satisfactory, based either on grab sampling and analysis or on observation of continuous automatic monitoring of pH or alkalinity of the recarbonated water. The CO_2 demands do not vary rapidly or widely, and manual control of dosage is better than might be expected.

SAFETY

Under certain conditions carbon dioxide can be dangerous, and there are safety precautions that must be observed. Prolonged exposure to concentrations of 5 percent or more CO_2 in air may cause unconsciousness and death. The maximum allowable daily exposure for a period of 8 hours is 0.5 percent CO_2 in air. Carbon dioxide is 1.5 times as dense as air, and therefore will tend to accumulate in low, confined areas. Filter-type gas masks are not useful in atmospheres containing excess CO_2, and self-contained breathing apparatus and hose masks must be used. Contact of the skin with liquid CO_2 can cause frostbite. Recarbonation basins should be located out of doors, and enclosed structures must not be built above them, because of the danger of excessive amounts of CO_2 accumulating within the structures. Before repairmen enter recarbonation basins, the CO_2 supply should be turned off and the space thoroughly ventilated. In the use of liquid CO_2 there are many other safety considerations too numerous and detailed to be covered completely here. Complete, published information can be obtained from liquid CO_2 suppliers.

FILTER MEDIA REHABILITATION

Should unstable, high pH water be applied to granular filters, encrustation of the media may occur. A variety of techniques to acidify the media to remove calcium carbonate deposits have been reported.[12] Chemicals used have included sulfuric acid, hydrochloric acid, chlorine, citric acid, muriatic acid (dilute hydrochloric acid), and glacial acetic acid.

The St. Louis County Water Company in Missouri has used citric acid to removed $CaCO_3$ encrustation from filter media for several years. The citric acid is obtained in granular form and applied as a 5 percent solution, with the media allowed to soak in the citric acid solution for approximately 4 h prior to backwashing.

Selection of an appropriate acidifying agent must take into account each acid's potential deleterious effects on concrete structures and filter underdrains and equipment. The city of Decatur, Illinois, concluded that a 10:1 dilution of glacial acetic was best suited to their needs.[12] Public relations concerns were minimized because the acid used was a food grade product (dilute acetic acid is vinegar). The filters were backwashed, drained to within 5–6 inches of the media surface, acid added over the entire surface of the filter bed, air-scour applied for 30 minutes, and the filter backwashed for 10 minutes.

REFERENCES

1. Sidorenko, G. I., et al., *Guidelines on Health Aspects of Water Desalination,* World Health Organization, 1981.
2. *Health Effects of the Removal of Substances Occurring Naturally in Drinking Water, with Special Reference to Demineralized and Desalinated Water*. Report of a working group, WHO Regional Office for Europe, Brussels, 1979 (EURO Reports and Studies No. 16).
3. Pontius, F. W., and Diamond, W. R., ''Complying with the Stage 1 D/DBP Rule,'' *JAWWA,* p. 16, March 1999.
4. Montgomery Consulting Engineers, ''Water Treatment Principles and Design,'' John Wiley & Sons, New York, 1985.
5. Sorg, T. J., and Logsdon, G. S., ''Treatment Technology to Meet the Interim Primary Drinking Water Regulations for Inorganics: Part 1 and Part 2,'' *JAWWA,* April and July 1978.
6. Berg, G., *Removal of Viruses from Water and Wastewater*. Dept. of Civil Engineering, University of Illinois, 1971.
7. Logsdon, G. S., Frey, M. M., Stefanich, T. D., Johnson, S. L., Freely, D. E., Rose J. B., and Sobsey, M., The Removal and Disinfection Efficiency of Lime Softening Process for *Giardia* and Viruses. *AWWA Research Foundation,* Denver, CO, 1994.
8. Semmens, M. J., and Staples, A. B., ''The Nature of Organics Removed During Treatment of Mississippi River Water,'' *JAWWA,* p. 76, February 1986.
9. Collins, M. R., Amy, G. L., and Steelink, C., ''Molecular Weight Distribution, Carboxylic Acidity, and Humic Substances Content of Aquatic Organic Matter: Implications for Removal During Water Treatment.'' *Envir. Sci. & Technol.,* p. 1028, October 1986.
10. Thompson, J. D., White, M. C., Harrington, G. W., and Singer, P. C., ''Enhanced Softening: Factors Influencing DBP Precursor Removal,'' *JAWWA,* p. 94, June 1997.
11. Cornwell, D. A., et al., *Water Treatment Plant Waste Management,* AWWA Research Foundation, 1987.
12. Mayhugh, J. R., Smith, J. A., Elder, D. B., and Logsdon, G. S., ''Filter Media Rehabilitation at a Lime Softening Plant,'' *JAWWA,* p. 64, August 1996.

CHAPTER 14

Iron and Manganese Removal

INTRODUCTION

Water containing iron and manganese is visually objectionable to consumers because precipitation of these metals causes the water to turn a rusty-looking yellow-brown or black. It can stain plumbing fixtures and laundry, and when used for irrigation, can stain buildings and concrete surfaces. It can also produce objectionable tastes and other aesthetic problems.

The presence of iron and manganese may also lead to the growth of microorganisms in a water distribution system. Slime layers several centimeters thick have been observed in distribution pipelines. These accumulations—which consist of hydrous iron, manganese oxides, and bacteria—reduce pipeline capacity, require higher chlorine dosages, and deplete dissolved oxygen (DO) levels. Sloughing or resuspension of this material by high pipeline velocities during hydrant flushing or peak demand periods causes high turbidities and leads to complaints of rusty or black water. Taste-and-odor complaints can be expected if the biological slime decomposes prior to arriving at the customer tap. Discussions of bacteria associated with the presence of iron and manganese can be found in *Water Quality and Treatment* and *Standard Methods.*[1,2]

The presence of iron and manganese in a water system has not been linked to health problems and is not regulated on these grounds. Thus, removal of iron and manganese has not received the attention and funding given to other treatment processes. In the absence of both a strong consumer demand for low iron and manganese levels at the tap and a willingness to pay for construction of treatment facilities, water systems have operated with elevated levels of these metals for extended periods of time. Some consumers deal with the iron and manganese problem at their homes by installing ion exchange water softeners or by changing the way they use the water for washing, cooking, and irrigation.

Water containing less than 0.1 mg/L of iron and 0.05 mg/L of manganese is not objectionable to the average customer; however, some industries may require lower levels of these metals. An American Water Works Association (AWWA) task group suggested limits of 0.05 mg/L for iron and 0.01 mg/L for manganese for an ‘‘ideal’’ quality water for public use.[3] The U.S. Environmental Protection Agency (USEPA) has established secondary maximum contaminant levels (SMCLs) for iron and manganese at 0.3 mg/L and 0.05 mg/L, respectively. Although regulations and suggested limits in Europe allow small concentrations of iron and manganese in drinking water, the goal there is to try to achieve nondetectable iron and manganese concentrations in the drinking water delivered to the customer.

Two types of iron are found in water supplies, and methods for their removal are quite different. Inorganic iron is generally associated with groundwaters low in dissolved oxygen. It will oxidize readily to its insoluble form and form a turbid water that can be filtered to remove the iron. The second type of iron is organically complexed iron, which may be found in both groundwaters and surface waters. The formation of organic complexes and chelates may increase the solubility of iron in some waters. Organically complexed iron can be significantly more difficult to oxidize than inorganic iron and may require special treatment considerations. At pH values encountered in natural waters, it is possible that organically bound iron will be insoluble but highly dispersed. This condition may require a coagulation step to adequately remove the complexed iron. Natural color found in water is frequently due to such highly stabilized colloidal dispersions of Fe(II).

Mn(IV) does not readily form complexes with organic or inorganic ligands in water and, therefore, does not have the same treatment problems associated with organically complexed iron. Inorganic manganese is readily oxidized and removed by filtration.

The sources and occurrence of iron and manganese are discussed in Chapter 2, "Inorganic and Radionuclide Contaminants."

FACTORS INFLUENCING THE TREATMENT OF IRON AND MANGANESE

It has been reported that, according to surveys, only 50 to 60 percent of the iron and manganese removal plants in the United States and 67 percent of those in Europe consistently meet the minimum standards for finished water iron and manganese concentrations.[5] Several factors were identified as to the cause of this poor performance, including:

- Iron complexation by silica and humic substances
- Too low oxidation pH
- Negative effect of chlorination
- Flocculation problems with the iron and manganese floc
- Too large effective media size
- Lack of analytical data at the time of design
- Deterioration in raw water quality
- Interference by nitrification
- Insufficient detention time for metal oxidation
- Unoptimized chemical dosing locations

All of these factors should be considered during pilot testing, design, and operation of an iron and manganese removal plant to minimize operational problems. A thorough, well-designed pilot study prior to design and construction is essential to the success of a treatment system. A pilot column in an operating plant can also be used to evaluate and optimize the process and provide the utility with valuable process information.

Of the reasons listed for poor plant performance, the presence of total organic carbon (TOC) in the water can provide some of the most significant treatment challenges. TOC will exert an oxidant demand in the system, making the required chemical dosages higher than might be expected. This condition, if unknown during design, could result in undersized pumps and other chemical handling or storage facilities. Under the right conditions, iron can become complexed with organic material and silica. Complexed iron is much more difficult to remove than simple dissolved iron in that it is resistant to oxidation and is not readily removed by filtration. Iron will complex with TOC material in the water at a rate of 0.1 mg/L of Fe for every 1.0 mg/L of TOC in the water; *this* will make iron removal very difficult without a coagulation step prior to filtration. Oxidant alone will usually not remove complexed iron.[6]

No significant organic complexation of manganese has been reported in the research literature. As far as the oxidation of manganese in the presence of dissolved organic carbon (DOC) is concerned, the main issue is to provide sufficient oxidant chemical to complete the oxidation reaction. The TOC in the water will impart an oxidant demand on the water that must be satisfied in addition to the oxidant demand of the manganese.[7] Raw water quality must be adequately characterized during the predesign and pilot study phases for manganese treatment facilities to be properly designed and constructed.

TOC has been shown to foul oxide-coated media. Therefore, the presence of TOC in a raw-water source will negatively impact the ability of oxide-coated or catalytic media to work successfully. The surfaces of the oxide-coated or catalytic media grains must be clean to work properly. They must have direct contact with the water in order to adsorb the metals and remove them from the water. The TOC in the water will coat the medium, fouling it and preventing it from working properly. Simple routine backwashing will not generally remove this coating, and special cleaning techniques may be required to restore the medium's effectiveness. In extreme cases, the medium will be fouled beyond repair and will have to be removed and replaced. When the use of an oxide-coated or catalytic medium is being considered, the presence or absence of TOC in the raw water must be known.

In systems using ozone as the oxidant for iron and manganese removal, the presence of TOC in the water can result in significant bacterial regrowth in the system. It has been widely reported in the research literature that ozonating organic material in water will increase the biologically assimilable organic carbon (AOC). This will result in increased biological activity in the system and potentially the buildup of biological slimes in the filters and the distribution system. Disinfectant demand will increase in these systems because of this biological activity. The control of biological slime growth in the distribution system is discussed in Chapter 22, ''Water Quality Control in Distribution Systems.'' Periodically disinfecting the filter media and backwashing will help control growths in the treatment system. Careful consideration is required when proposing the use of ozone for iron and manganese oxidation in the presence of TOC.

Water quality analyses, as well as evaluation and pilot studies, should be conducted to develop design criteria for each of the proposed systems. The following sections outline and describe many of the systems that are typically used to treat iron and manganese in drinking water.

STABILIZATION OF SEQUESTERING METHODS

A low-cost alternative to removing iron and manganese is to hold the metals in solution by stabilization or sequestering. Sequestration has several advantages over conventional treatment:

- Inexpensive to install
- Suitable for wellhead treatment
- Operator friendly
- Produces no sludge that would require handling and disposal[8]

Iron and manganese must be in the ionic state for this process to be effective. Generally, sequestering is appropriate for groundwaters containing sufficient carbon dioxide to ensure that iron and manganese will be present as bicarbonates. The sequestering agent is added directly into the water as the water is pumped from the well. Commonly used sequestering agents include sodium hexametaphosphate, trisodiumphosphate, and sodium silicates. The addition of sequestering agents, especially phosphates, can provide a source of nutrients and, therefore, promote bacterial regrowth in the system. To minimize this possibility, a disinfectant residual must be maintained in the system. If the disinfectant has oxidation capabilities (i.e., chlorine), it must be added after the sequesterant to avoid oxidation of the metals.

Sequestering should be considered only for waters in which the sum of the iron and manganese is less than 1.0 mg/L. Sodium silicates, while effective in sequestering iron, have not been shown to be effective for manganese sequestering.[8] Required chemical dosages can vary significantly from product to product and from water to water. Polyphosphate dosages should be limited to less than 10 mg/L, because phosphorus can stimulate biological regrowth in the distribution system. Calcium hardness has been found to interfere with treatment by both polyphosphates and orthophosphates.[8] Given the variability of the chemicals and their effectiveness, it is advisable to pilot-test several chemicals prior to investing in a sequestering system. Features of the sequestering process (as well as other iron and manganese removal processes) are described in Table 14–1; advantages and disadvantages are listed in Table 14–2.

Following the addition of a polyphosphate, an adequate disinfectant residual must be maintained to control bacterial slime regrowth in the distribution system. The phosphates in the system can serve as a nutrient source for bacteria. Failure to control the biological slimes could result in Total Coliform Rule (TCR) testing problems for the utility. For best results, the sequestering agent is added first and the disinfectant second. The reaction of the sequestering agent with the metals is very rapid, and it is recommended that the disinfectant be added within a few seconds of adding the sequesterant. Sequestering agents are generally effective in cold water systems; when the water is heated or boiled, polyphosphates lose their dispersing properties and the metal may oxidize and come out of solution. Sequestering agents are typically effective for only 48 to 72 hours. Systems that have large storage reservoirs with long detention times or distribution systems with long dead-end pipelines where water could sit for significant periods of time may have problems using sequestering agents. When the sequestering agents cease to be effective, the iron and manganese could oxidize in the distribution system, leading to complaints of rusty or black water and staining from consumers. Sequestering agents also have a limited effective shelf life. As the agents age, they lose their effectiveness and will require higher dosages. The utility must monitor the system to ensure continued long-term success of the sequestering feed system.

Sequestering is an attractive treatment process because of its low capital costs and simple operation. All it requires is a chemical feed pump and a dosing location. For small community systems, seasonal systems, or wellhead treatment, sequestering may

TABLE 14–1. Iron and Manganese Removal Processes

Treatment Option	Raw Water Metal Concentration Applicabale for Treatment	Filtration Type	Typical Hydraulic Filter Loading Rate	Detention Time Prior to Filtration	Factors Affecting Removal of Iron and Manganese	Notes and Comments	Operational Considerations	Backwashing Requirements	Permitting
Sequestering	Recommended that sum of Fe and Mn concentration be <1.0 mg/L	No filters required for the process	No filters required for the process	Sequestering chemicals should be injected a few seconds before the chlorine at the wellhead. Reaction is instantaneous.	Role of pH uncertain. Process may be impacted by the presence of TOC. Calcium hardness may interfere with sequestering chemicals.	Generally effective only for 48 to 72 hours. Problems can develop in long dead-end lines or storage reservoirs.	Only requirements are chemical feed pumps and chemical storage. Phosphates may result in regrowth in the distribution system. Process does not remove the Fe and Mn from the water.	No backwashing facility required	Generally accepted by the regulatory agencies for low concentrations of Fe and Mn. Agencies may view seqestering as a temporary measure only.
Aeration and filtration	Fe: 1.0–10.0 mg/L Mn: Not effective at pH <9.5	Conventional pressure or gravity filters with dual media of anthracite and silica sand	2 to 5 gpm/ft^2 (4.9 to 12.2 m/h)	Fe: Generally from 5 to 20 min Mn: >1 hr at pH of 9.5; >12 hr at pH of 7	Treatment performance is improved with: • Increasing temperature • Higher pH • Decreasing TOC concentration • Longer detention time Aeration is not effective on organically complexed iron. Not usually applicable for Mn removal because of long required detention time.	Commonly used where Fe is found alone. Recommended for high Fe concentration to save on chemical costs.	Repumping usually required. Significant pH adjustment up and then down would be required for Mn removal.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	Widely accepted treatment technology for Fe removal. Process would readily be permitted by the regulatory agencies.

TABLE 14-1. (*Continued*)

Treatment Option	Raw Water Metal Concentration Applicabale for Treatment	Filtration Type	Typical Hydraulic Filter Loading Rate	Detention Time Prior to Filtration	Factors Affecting Removal of Iron and Manganese	Notes and Comments	Operational Considerations	Backwashing Requirements	Permitting
Chlorination and filtration	Fe: <2 mg/L Mn: Oxidation with chlorine not recommended	Conventional pressure or gravity filters with dual media of anthracite and silica sand	2 to 5 gpm/ft^2 (4.9 to 12.2 m/h)	Fe: Generally from 5 to 20 min Mn: 2 to 3 hr at pH > 8.0	Treatment performance is improved with: • Increasing temperature and pH • Decreasing TOC concentration • Longer detention time Not effective on organically complexed iron. Chlorine oxidation of Mn is not practical at neutral pH and typical detention times.	Commonly used in systems where Fe is found alone. Can also provide primary disinfection.	Simple and reliable to dose and control.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	Readily accepted by the regulatory agencies
$KMnO_4$ and filtration	Fe: Unlimited Mn: Unlimited	Pressure or gravity filters with either: 1. Dual media of anthracite and sand. 2. Dual media of anthracite and greensand or oxide-coated medium	With silica sand: 1. Gravity, 2–5 gpm/ft^2 (4.9–12.2 m/h) 2. Pressure, 4–5 gpm/ft^2 (9.8–12.2 m/h) With greensand: ~4 gpm/ft^2 (9.8 m/h) max.	No additional time required—sufficient time available in piping and the filter headspace.	Rate of removal and treatment performance are improved with: • Increasing temperature • Higher pH • Decreasing TOC content Overdosing with $KMnO_4$ can lead to pink water complaints.	$KMnO_4$ is a strong oxidant that requires care in handling. $KMnO_4$ alone is typically used where Mn is found alone and Fe is not a problem.	The advantage of having only one chemical feed is attractive for small systems. $KMnO_4$ is corrosive and requires special handling. Simple and reliable to dose and control in typical groundwater.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	Common method for the removal of Fe and Mn. Readily accepted by the state regulatory agencies.

Ozonation and filtration	Fe: Unlimited Mn: Unlimited	Usually use pressure filters with dual media of anthracite and silica sand	4–5 gpm/ft^2 (9.8–12.2 m/h) using pressure filters	Generally provide an ozone contact tank with 3 to 5 min of contact time prior to the filters	Rate of removal and treatment performance are improved with increasing temperature and decreasing TOC content. Overdosing with ozone can drive the manganese to permanganate and result in pink water complaints and manganese breakthrough.	Requires on-site generation and significant power consumption. Ozone is a primary disinfectant.	Ozone generation equipment is complex and expensive to buy, requires significant electrical power, has no chemicals to handle, and is easy to control for a stable groundwater chemistry.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	Relatively new technology for Fe and Mn removal but should readily be permitted by state regulatory agencies
Chlorine, $KMnO_4$, and filtration	Fe: Unlimited Mn: Unlimited	Pressure or gravity filters with either: 1. Dual media of anthracite and sand 2. Dual media of anthracite and greensand or oxide-coated medium	With silica sand: 1. Gravity, 2–5 gpm/ft^2 (4.9–12.2 m/h) 2. Pressure, 4–5 gpm/ft^2 (9.8–12.2 m/h) With greensand: ~4 gpm/ft^2 (9.8 m/h) max.	No additional time required—sufficient time available in piping and the filter headspace	Rate of removal and treatment performance are improved with: • Increasing temperature • Higher pH • Decreasing TOC content Overdosing with $KmnO_4$ can lead to pink water complaints.	$KMnO_4$ is a strong oxidant that requires care in handling. $KMnO_4$ is an expensive chemical. The use of Cl_2 reduces the $KMnO_4$ demand.	Requires the addition and handling of two chemicals. $KMnO_4$ is corrosive and requires special handling. Simple and reliable to dose and control in typical groundwater.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	‘‘Classic’’ North American method for the removal of Fe and Mn. Readily accepted by the state regulatory agencies.

TABLE 14-1. (*Continued*)

Treatment Option	Raw Water Metal Concentration Applicabale for Treatment	Filtration Type	Typical Hydraulic Filter Loading Rate	Detention Time Prior to Filtration	Factors Affecting Removal of Iron and Manganese	Notes and Comments	Operational Considerations	Backwashing Requirements	Permitting
Chlorine dioxide and filtration	Fe: Unlimited Mn: Unlimited if sufficient time is provided to complete the reaction	Pressure or gravity filters with either: 1. Dual media of anthracite and sand 2. Dual media of anthracite and greensand or oxide-coated medium	With silica sand: 1. Gravity, 2–5 gpm/ft^2 (4.9–12.2 m/h) 2. Pressure 4–5 gpm/ft^2 (9.8–12.2 m/h) With greensand: ~4 gpm/ft^2 (9.8 m/h) max.	Fe reaction is very fast. Mn reaction at neutral pH requires >6 hr.	Rate of removal and treatment performance are improved with: • Increasing temperature • Higher pH • Decreasing TOC content Oxidation and removal of Mn are difficult at neutral pH without a long detention time.	Requires on-site generation and two chemical feeds. ClO_2 is more commonly used for disinfection.	Requires on-site generation and the handling of two chemicals. Operationally complex.	Anticipated filter run times of 24 to 48 hr and backwash rates of 12–16 gpm/ft^2 (29–39 m/h) are typical with backwash volumes of 2 to 5% of the water produced	Readily accepted by the regulatory agencies
Oxide-coated or catalytic media	Fe: Unlimited; TOC <2 mg/L Mn:Unlimited; TOC <2 mg/L	Pressure or gravity filters with one of the following: 1. MnO_2 ore 2. Greensand media 3. Manganese dioxide coated silicon dioxide 4. Conventional silica sand media conditioned with $KMnO_4$	2–10 gpm/ft^2 (4.9–12.2 m/h) depending on the metal concentration. For Mn ore, 5–15 gpm/ft^2 (12.2–36.7 m/h).	Metals adsorbed onto the media and held there until backwashing. No detention time required for removal. Media require regular or continuous regeneration to maintain effectiveness.	Generally not recommended for water with TOC >1 mg/L because fouling of the media may result. Media require regular or continuous regeneration with Cl_2 or $KMnO_4$ to work effectively.	Often used with a continuous Cl_2 or $KMnO_4$ feed for regeneration of the media. The Cl_2 and $KMnO_4$ will also oxidize the iron and reduce the loading on the media.	Simple to use and control. Because of the sensitivity of the media to TOC concentration, raw water quality assessment is essential for success of the process. Chemical requirements are reduced because of media adsorption.	Filter run times: • Standard media: 24–48 hr • Greensand: 24 hr • MnO_2: 8–12 hr Backwash rates: • Standard media: 15 gpm/ft^2 (37 m/h) • Greensand: 12 gpm/ft^2 (29 m/h) • MnO_2 ore: 25–30 gpm/ft^2 (61–73 m/h)	Readily accepted by the regulatory agencies

Biological filtration	Fe: Unlimited; sensitive to the presence of zinc and H_2S Mn: Unlimited; sensitive to NH_4	Pressure or gravity filters 1. Silica sand to support bacteria 2. Sand that is slightly larger than conventional filtration media	5–15 gpm/ft^2 (12.2–36.7 m/h) depending on the metal concentration	None required because no chemicals are used	Optimum conditions for Fe removal: • Zinc <0.45 mg/L • H_2S <0.1 mg/L • Temperature >50°F (10°C) • Redox potential: 0–500 mV For Mn removal: • Zinc <0.45 mg/L • H_2S <0.1 mg/L • pH >7.4 • NH_4 <0.1 mg/L • Fe <0.1 mg/L • DO >4.0 mg/L • Temperature >50°F (10°C) • Redox potential: >200 mV	Widely used in Europe. New technology to North America. Only one known permitted biological Fe removal plant in North America. No permitted biological Mn removal plants in North America.	Need to operate the plant continuously for best operational results. May require variable-speed drives on the well pumps. Seasonal start-up periods would be required for the biological filters.	Unchlorinated backwash water required. Anticipated filter run times of 48 hr and backwash rates of 10 gpm/ft^2 (24 m/h) are typical with volumes of 0.5 to 2% of the water produced. Air scour typical.	Widely used in Europe but new to the United States. Permitting would require extensive pilot testing to demonstrate that this is a viable and safe treatment technology.

TABLE 14-2. Advantages and Disadvantages of Iron/Manganese Removal Processes

Treatment Option	Advantages	Disadvantages
Sequestering	• Operationally simple—only chemical feed and storage required • Inexpensive • Suitable for wellhead treatment • Does not produce residuals that have to be handled	• Does not remove iron and manganese—only holds them in solution • Effectiveness exhausted after 48 to 72 hr, which could lead to staining problems for large storage volumes and long dead-end pipelines • Addition of phosphorus could enhance biological growth in distribution system • A free chlorine residual must be maintained in the system to minimize regrowth
Aeration and filtration	• Can reduce high concentrations of iron • Simple to operate and maintain • No chemicals to handle	• Generally considered to be unsuccessful for manganese removal at neutral pH • Repumping usually required • Not effective for removal of organically complexed iron
Chlorination and filtration	• Effective for Fe removal • Operationally simple and stable • Able to provide primary disinfection benefits • Readily accepted by regulatory agencies	• Not effective on Mn without high pH and long detention time • Not effective on organically complexed iron
$KMnO_4$ and filtration	• Widely used and effective method for both Fe and Mn removal • Rapid reaction to oxidize both metals • Operationally simple • Readily accepted by regulatory agencies	• Potential to overdose the $KMnO_4$, which could cause pink water complaints • High chemical costs • Not effective on organically complexed iron without a clarification step • $KMnO_4$ is corrosive, so special handling is required
Ozonation and filtration	• Effective for both Fe and Mn removal • Provides benefits in terms of taste and odor • Able to provide primary disinfection benefits • No chemicals to buy or store	• Potential to overoxidize Mn to permanganate, causing pink water • High capital and energy costs • On-site generation required • Ozone equipment is complex and requires specialized servicing • TOC in the water could cause biological slime growth problems in the system

Chlorine, $KMnO_4$, and filtration	• Widely used and effective method for both Fe and Mn removal • Use of Cl_2 reduces $KMnO_4$ requirement and therefore cost • Rapid reaction to oxidize both metals • Operationally simple • Readily accepted by regulatory agencies	• Potential to overdose the $KMnO_4$, which could cause pink water complaints • Two chemicals required • Not effective on organically complexed iron without a clarification step • $KMnO_4$ is corrosive, so special handling is required
Chlorine dioxide and filtration	• Effective for Fe removal • Can provide some benefits in terms of taste and odor • Provides primary disinfection	• Mn removal requires significant reaction times • Not effective on organically complexed iron without a clarification step • On-site generation required—operationally complex • High equipment costs
Oxide-coated or catalytic media	• Can be effective for both Fe and Mn removal	• Media require regular or continuous regeneration to maintain effectiveness
Greensand	• Operationally simple	• Media are sensitive to the TOC level in the raw water and may not be effective • Use of greensand results in shorter filter runs and lower filter loading rates because of the small media size
Manganese dioxide ore	• Lowers chemical costs relative to using ozone or $KMnO_4$ • Readily accepted by regulatory agencies • May allow oxidation of the Mn through the use of only Cl_2	• MnO_2 ore media require very high backwash rates
Biological filtration	• Capable of consistently producing lowest concentrations of Fe and Mn and therefore the best water quality • Simple operation—chemical handling minimized • Reduced backwash requirements, flows, and time • Smaller space requirement because of high filter loading rates • Longer filter runs and less sludge production and handling • Capital cost savings of 20 to 30% over conventional treatment • Operations and maintenance costs reduced by reduced chemicals and fewer operations	• Would require extensive pilot testing for acceptance by regulators • Will not apply to all water qualities • Two-stage filtration required for high-rate Fe and Mn removal • Limited experience in North America • Long initial start-up for manganese removal • A short start-up period required after each shutdown of more than 7 days • Impacted by the presence of ammonium, zinc, and H_2S

be the treatment method of choice. For larger community systems—greater than 100,000 gpd (378,500 L/d)—sequestering is typically used only for temporary treatment and is rarely used for long periods. State regulatory agencies vary on their acceptance of the long-term use of sequestering for iron and manganese systems. It is recommended that a utility consult the regulatory agencies when considering the installation of a sequestering system.

PHYSICAL/CHEMICAL REMOVAL OF IRON AND MANGANESE

The most commonly used processes for physical/chemical removal of iron and manganese involve oxidation and precipitation followed by filtration. Oxidation methods include:

- Aeration
- Chlorination or chlorine dioxide oxidation
- Potassium permanganate oxidation
- Prechlorination and potassium permanganate oxidation
- Ozonation

Other physical/chemical methods for the removal of iron and manganese are also sometimes used either alone or in conjunction with oxidation and filtration processes:

- *Use of oxide-coated or catalytic media.* Such media include manganese dioxide ore, manganese greensand, manganese dioxide–coated silicon dioxide, and other specially treated media that generally use adsorption as the mechanism for metal removal or act as a catalyst to speed the oxidation reaction. An oxide-coated or catalytic medium is often used as the filtration medium in conjunction with either chlorine or $KMnO_4$ oxidation to reduce chemical costs.
- *Lime softening.* While generally too expensive to use specifically for iron and manganese, lime-softening processes will remove the metals during treatment as a secondary benefit.
- *Ion exchange.* Ion-exchange resins are commonly used for household water softening and will remove iron and manganese from the water as a secondary benefit during the softening process. In light of economic and operational constraints, ion exchange is generally used only for very small community systems (100 gpm or less), and then only when hardness of the water is also an issue.
- *Manufacturers' proprietary processes.* Companies have developed packaged or specialized processes that will remove iron and manganese from the water while providing a utility with cost or operational benefits.

The features and characteristics of several physical/chemical iron and manganese removal systems are listed in Table 14–1. The advantages and disadvantages of each of the listed processes are itemized in Table 14–2.

General Principles

The chemistry of iron and manganese oxidation is not fully understood. There are no simple relationships among the iron, manganese, alkalinity, pH, TOC, and redox potential of the water.[8] Each water source must be evaluated to determine the proper oxidant and the optimal conditions for iron and manganese oxidation; no single treatment process is appropriate for all water qualities. An important factor in the removal of iron and manganese is that sufficient oxidation be achieved prior to filtration, with adequate chemical dosage and detention time to allow the reaction to go to completion. For iron and manganese to be removed by physical/chemical methods, the metals must be oxidized from their soluble states to their insoluble states. The rate of the iron oxidation reaction and the final oxidation product are dependent on the pH, carbonate alkalinity, strength of the oxidant being applied, reaction time provided, and presence or absence of organic material. The chemistry of manganese oxidation is much more complex and dependent on the pH, oxidation potential of the oxidant being used, and time available for the reaction.[9]

The following sections discuss specific chemistry for the commonly used oxidants for iron and manganese removal. The associated chemical reactions are shown in Table 14–3.

Aeration, Precipitation, and Filtration

A commonly used oxidation technique and simple method of removing iron (and, to a lesser extent, manganese) from water involves aeration, precipitation, and filtration. Simple aeration converts ferrous bicarbonate to ferrous hydroxide, as described in Table 14–3.

The rate of oxidation of Fe(II) by oxygen is slow under conditions of low pH. Aeration of water low in dissolved oxygen and high in carbon dioxide will tend to raise the pH slightly because carbon dioxide is easily removed by aeration. Reaction rates are fairly slow at a pH less than 7, and a pH of 7.5 to 8.0 may be required to complete the reaction in less than 15 minutes.[10] As much as 60 minutes or more of contact time may be needed to complete the reaction under certain conditions. Detention tanks are usually provided for systems using simple aeration for oxidation of the iron. Aeration will not remove organically complexed iron. Because TOC will complex with iron, a utility should evaluate the TOC level prior to considering the use of aeration for iron removal. Filtration is always required following aeration to remove the oxidized metals.

The rate of oxygenation of Mn(II) by simple aeration is very slow at pH values of less than 9.5 Even at a pH of 9.5, the oxidation of manganese using only aeration will require more than 60 minutes to complete.[10] The chemical reaction is listed in Table 14–3. Theoretically, 1 g of oxygen is required to oxidize 7 g of Fe(II), and 1 g of oxygen is required to oxidize 3.5 g of Mn(II). Reaction times of anywhere from 3 to 12 hr are required to oxidize manganese by simple aeration of the water for a pH range of 7 to 8. For this reason, when manganese is present in a water source, simple aeration is usually not used alone.

A typical aeration process layout is illustrated in Figure 14–1. The process shown uses a tray aeration system and a large detention tank to complete the oxidation reaction. Aerating the water for oxidation can be accomplished in other ways, including pressure aeration, spray aeration, and the use of aeration basins.

Self-precipitation of ferric and manganic hydroxides can be slow; therefore, flocculation rates are frequently accelerated by contact and by catalysts. Contact aerators and contact filters are used to accelerate oxidation. Water is trickled over coke, crushed stone, or other contact materials. Deposits of hydrated oxides of iron and manganese accumulate on the surfaces and catalyze further oxidation to ferric and manganic

TABLE 14-3. Reactions and Dosages for Physical/Chemical Methods of Removing Iron and Manganese

Oxidant or Medium	Chemical Reaction	Dosage	Comments
		Iron Oxidation	
Oxygen	$4Fe(HCO_3)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 + 8CO_2$	0.14 mg/mg Fe	Very effective for high concentrations of Fe. Not effective on organically complexed iron.
Chlorine	$2Fe(HCO_3)_2 + Ca(HCO_3)_2 + Cl_2 \rightarrow 2Fe(OH)_3 + CaCl_2 + 6CO_2$	0.64 mg/mg Fe	Reaction is pH and temperature dependent. Not effective on organically complexed iron.
Chlorine dioxide	$Fe(HCO_3)_2 + NaHCO_3 + ClO_2 \rightarrow Fe(OH)_3 + NaClO_2 + 3CO_2$	1.21 mg/mg Fe	Reaction is pH and temperature dependent. Not effective on organically complexed iron.
Potassium permanganate	$3Fe(HCO_3)_2 + KMnO_4 + 2H_2O \rightarrow 3Fe(OH)_3 + MnO_2 + KHCO_3 + 5CO_2$	0.94 mg/mg Fe	Essentially instantaneous. Not always effective on organically complexed iron.
Ozone	$2Fe(HCO_3)_2 + O_3 + 2H_2O \rightarrow 2Fe(OH)_3 + O_2 + 4CO_2 + H_2O$	0.43 mg/mg Fe	Essentially instantaneous
Manganese dioxide (oxide-coated media and greensand)	$2Fe^{2+} + 2MnO_2 + 5H_2O \rightarrow 2Fe(OH)_3 + Mn_2O_3 + 4H^+$	—	MnO_2 not generally used for Fe removal. The MnO_2 holds Fe on the medium rather than reacting with it; Fe is then removed during backwash. TOC concentrations greater than 1 mg/L will foul the medium and reduce its effectiveness. MnO_2 is not recommended for waters containing >1 mg/L TOC.

		Managanese Oxidation	
Oxygen	$2MnSO_4 + 2Ca(HCO_3)_2 + O_2 \rightarrow 2MnO_2 + 2CaSO_4 + 2H_2O + 4CO_2$	0.29 mg/mg Mn	Very slow reaction at neutral pH. Reaction speed increases with increasing pH and temperature. Not usually implemented for Mn removal.
Chlorine	$Mn(HCO_3)_2 + Ca(HCO_3)_2 + Cl_2 \rightarrow MnO_2 + CaCl_2 + 2H_2O + 4CO_2$	1.29 mg/mg Mn	Reaction is slow at neutral pH. Reaction speed increases with increasing pH and temperature.
Chlorine dioxide	$Mn(HCO_3)_2 + 2NaHCO_3 + 2ClO_2 \rightarrow MnO_2 + 2NaClO_2 + 2H_2O + 4CO_2$	2.46 mg/mg Mn	Reaction is slow at neutral pH. Reaction speed increases with increasing pH and temperature. Faster than free chlorine alone.
Potassium permanganate	$3Mn(HCO_3)_2 + 2KMnO_4 \rightarrow 5MnO_2 + 2KHCO_3 + 2H_2O + 4CO_2$	1.92 mg/mg Mn	Reaction is very rapid at all pH values.
Ozone	$Mn(HCO_3)_2 + O_3 + 2H_2O \rightarrow MnO_2 + O_2 + 2CO_2 + 3H_2O$	0.88 mg/mg Mn	Reaction is very fast and can drive the Mn to permanganate, causing pink water if there is an ozone overdose.
Manganese dioxide (oxide-coated media and greensand)	$Mn^{2+} + MnO_2 + H_2O \rightarrow Mn_2O_3 + 2H^+$	—	MnO_2 does not react with manganese but holds it on the medium and gives the oxidant an opportunity to oxidize it. Frequent backwashing is necessary to keep the adsorption sites on the medium clean and allow them to function. TOC concentrations greater than 1 mg/L will foul the medium and reduce its effectiveness. MnO_2 is not recommended for waters containing >1 mg/L TOC.

Source: See Reference 3. (Reprinted from *Water Treatment Plant Design*, 3d ed., by permission. Copyright © 1998, American Water Works Association.)

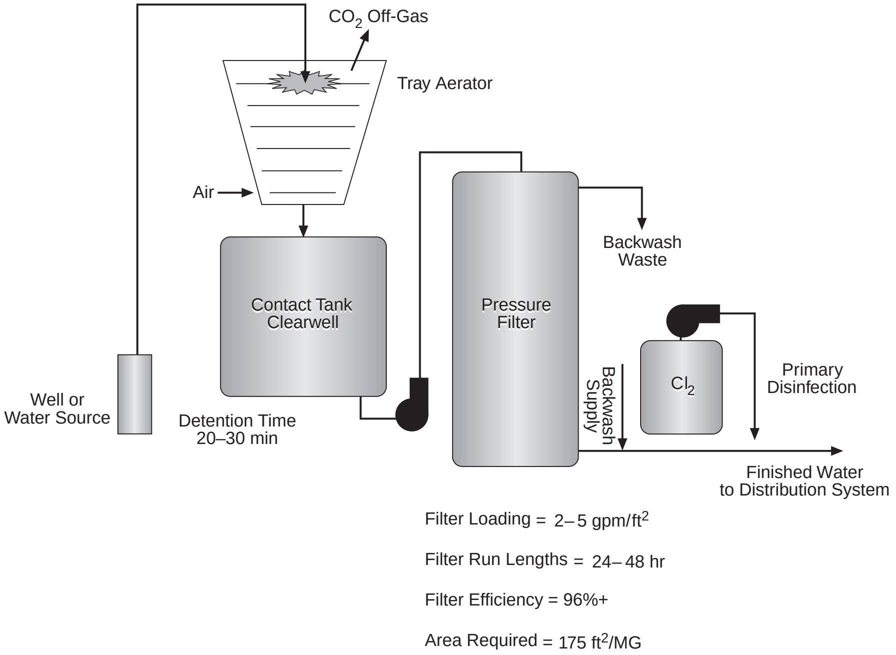


Fig. 14–1. Aeration system for iron and manganese removal

oxides. Limestone and pyrolusite (MnO_2) have also been used for this purpose. Contact beds are usually arranged in a series of trays 12 to 18 in. (0.3 to 0.45 m) deep with perforated bottoms to enhance aeration. Loading rates range from 15 to 20 gpm/ft^2 (37 to 49 m/h).[10]

Chlorination Oxidation, Precipitation, and Filtration

Chlorine is a powerful oxidizing agent that is used in the removal of iron and manganese. Chlorine is the preferred oxidant because of its economical cost and ease of use. Free or combined chlorine reacts to oxidize ferrous iron as illustrated in Table 14–3.

The oxidation of iron by chlorine takes place over a wide pH range of 4 to 10, with an optimum pH of 7.0. At a pH of 7.0, the reaction of the chlorine with the iron is rapid. Organically complexed iron is resistant to oxidation by chlorine and is generally not recommended.

Manganese is oxidized by chlorine according to the reaction noted in Table 14–3. The reaction time for manganese oxidation is much longer than that for iron oxidation. At a pH of 8.0, the reaction may take 2 to 3 hours, whereas at pH 6.0 the time extends to as much as 12 hours. For pH values of 9.0 to 10.0, the oxidation is completed within minutes. O'Connor recommended a minimum pH of 8.0 for oxidation of Mn with free chlorine.[7]

When chlorine is used alone as the oxidant, oxide-coated or catalytic media are commonly used to hold the metals on the filter to give the chlorine sufficient time to complete the reaction. The metal oxides are then removed from the filter by backwashing. The free chlorine residual in the filter feedwater provides a continuous regeneration of the media, allowing the process to work successfully.

Chlorine is an inexpensive chemical that is easily applied and provides the benefit of primary disinfection; its use is, therefore, attractive for iron and manganese removal systems. Care should be used in developing the proper dosing facilities for waters containing ammonia or other nitrogenous compounds that interfere with the formation of a free chlorine residual. The presence of TOC in the water, in addition to causing complexing of the iron, can lead to objectionable levels of trihalomethane (THM) formation with the use of chlorine and must be evaluated during the pilot study phase of the project.

Chlorination or Chlorine Dioxide Oxidation, Precipitation, and Filtration

Chlorine dioxide is a powerful oxidizing agent that is used in the removal of iron and manganese. It may be chosen over chlorine if trihalomethane formation is a problem. Chlorine dioxide reacts to oxidize ferrous iron as illustrated in Table 14–3.

The oxidation of iron by chlorine dioxide takes place over the wide pH range of 4 to 10. At the optimum pH of 7.0, the reaction with iron is rapid. Organically complexed iron is difficult to oxidize with chlorine dioxide.

Chlorine dioxide reacts more quickly with manganous compounds than does free chlorine; therefore, it is more successful for manganese removal. The reaction of chlorine dioxide and manganese is listed in Table 14–3. However, the high cost of chlorine dioxide treatment limits its use to applications where manganese concentrations are less than 1.0 mg/L.

Potassium Permanganate Oxidation, Precipitation, and Filtration

The use of potassium permanganate in iron and manganese removal systems is very common throughout North America. $KMnO_4$ is simple to apply and regulate. Its use for removal of both iron and manganese is attractive because reactions are rapid and complete. The reactions and required dosages are illustrated in Table 14–3. Generally, because of the catalytic effect of precipitated manganese dioxide on the filter medium, less $KMnO_4$ is needed to complete the reactions than is theoretically required.

Iron oxidation by $KMnO_4$ is essentially instantaneous and requires no special provisions for detention time. Sufficient time is provided in the piping and the filter above the medium to allow the reaction to go to completion. The reaction of $KMnO_4$ in oxidizing ferrous iron is further enhanced by the formation of MnO_2 on the medium, which acts as a catalyst. However, because of its high cost and the availability of more economical oxidants, $KMnO_4$ is not generally used solely for iron removal.

$KMnO_4$ is a popular means of oxidizing manganese because it is easy to use and offers a fast reaction time. It will oxidize manganous ions to manganese dioxide rapidly and over a wide pH range. The reaction time can be as short as 5 minutes,[7] and the amount of $KMnO_4$ required for oxidation decreases with increasing pH. Generally, no special provisions are provided for detention time for the reaction of $KMnO_4$ and manganese. Sufficient time is provided in the piping and in the filter above the medium. As the manganese is oxidized in the process and removed on the filter, manganese oxides coat the medium and aid in manganese removal by chemical adsorption. This phenomenon reduces the required $KMnO_4$ dosage.

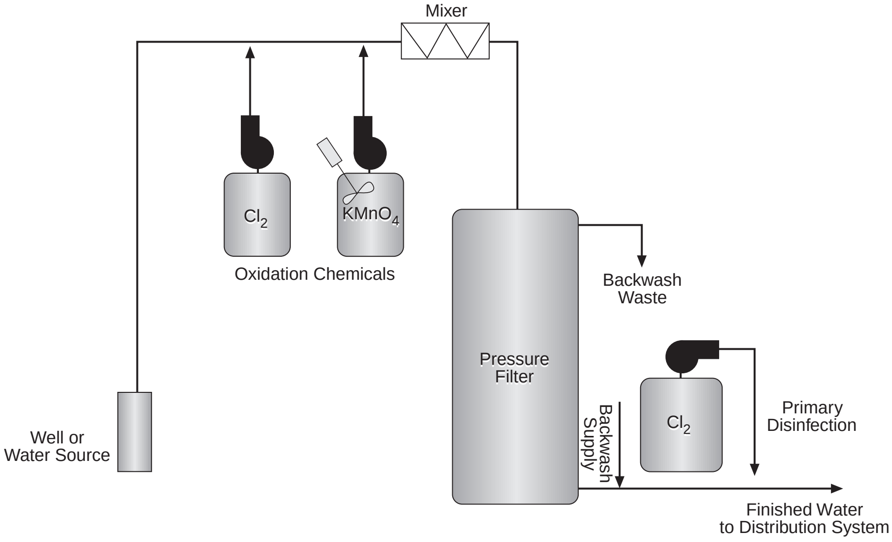
Utilities must be careful not to overdose $KMnO_4$ during treatment. Excessive quantities will result in permanganate breakthrough and a characteristic pink hue in the water. Pink water complaints are common in systems where high chemical dosing is a problem. For groundwaters with a stable water chemistry, the required dosage remains steady and pink water is generally not a problem. For waters with a variable water chemistry and an erratic chemical demand, pink water due to overdosing will be more of a problem. In these instances, instrumentation can be added to measure the color in the water and shut the system down when an overdosing condition exists.

Prechlorination, Potassium Permanganate Oxidation, Precipitation, and Filtration

Prechlorination is frequently used in practice to reduce the required $KMnO_4$ dosage for iron and manganese removal. This is advantageous because it reduces the overall cost of $KMnO_4$, which is significantly more expensive than chlorine. Figure 14–2 illustrates a typical prechlorination and $KMnO_4$ process layout.

Ozone Oxidation, Precipitation, and Filtration

Ozone is a powerful oxidizing agent that is becoming increasingly common in the treatment of iron and manganese. Ozone equipment that is less complex, more affordable, and easier to operate is making the process attractive. Ozone provides primary disinfection and some taste-and-odor advantages to a utility.



Filter Loading = 4–5 gpm/ft ^{2}

Filter Run Lengths = 24– 48 hr

Filter Efficiency = 96%+

Area Required = 140 ft^2/MG

Fig. 14-2. Prechlorination system for removal of iron and manganese

Ozone reacts to oxidize ferrous iron as shown in Table 14–3. The reaction between ozone and iron occurs essentially instantaneously and requires no detention time. It takes place over the wide pH range of 4 to 10, with an optimum of 7.0. Organically complexed iron may be oxidized by ozone; however, oxidation of this form of iron should be pilot tested to verify the oxidation process.

Table 14–3 also shows the reaction via which manganese is oxidized by ozone. Manganese oxidation rates using ozone are acceptable above pH 5.[7] The reaction with manganese is complete in a few minutes, and a detention tank with a 3–5-minutes residence time is needed in the system. A typical ozone system is illustrated in Figure 14–3.

During ozonation, overdosing is possible as the manganese is driven to permanganate, a soluble form of manganese that passes through the filter. When this occurs, the utility may receive consumer complaints of pink water. For groundwaters that have a reasonably stable water quality and for which the dose remains constant, overdosing should not be a problem. In water having a variable water quality, the operator may have to adjust the dosage to meet demand, or instrumentation may be required to control the process.

Ozone will react with TOC in the water and can break down complex organic molecules into more AOC. This can lead to problems with slime regrowth in the filters and the distribution system, as well as higher THM formation. Utilities contemplating the use of ozone should consider these more complex issues in the design and operation of the system.

The depth of filtration systems to remove AOC is discussed in Chapter 12, ‘‘Filtration.’’

Adsorptive or Catalytic Media Filtration

Special filter media are regularly used successfully in the removal of iron and manganese because of their adsorptive or catalytic capacity. Adsorptive media have the

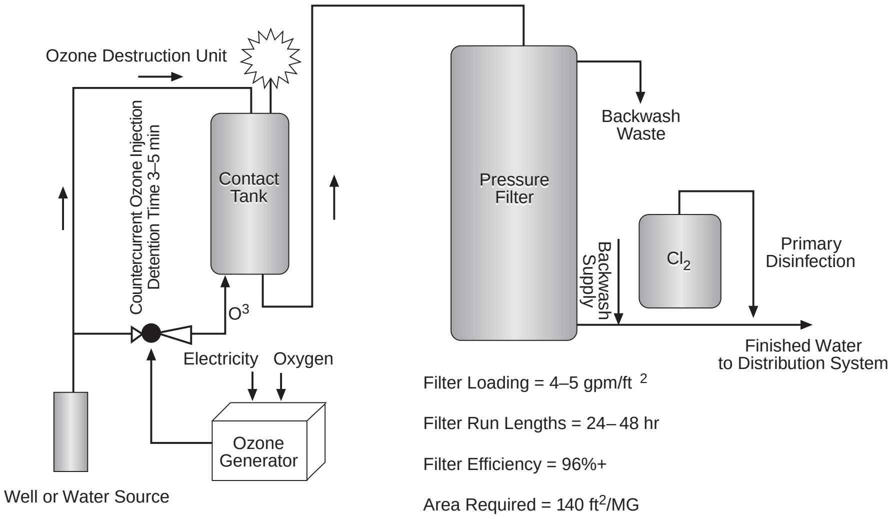


Fig. 14–3. Ozonation system for iron and manganese removal

ability to hold the metals on them, thus removing the metals from the water with no chemical addition. Catalytic media speed the oxidation reaction to completion. Several different types of adsorptive and catalytic media have been developed and are commonly used in iron and manganese removal plants. Generally, all of these media use some form of manganese dioxide ore or coating, and the chemistry of removal is similar. Many of the process mechanisms are the same for each medium described. If a mechanism applies to one medium, it more than likely applies to them all and should be considered during process evaluation. These special materials will be discussed in detail in the following paragraphs.

A drawback to using these special media is that they are generally susceptible to fouling when the TOC concentration is greater than 1.0 mg/L. The presence of TOC in water may also be an indication that a portion of the iron is organically complexed. Adsorptive or catalytic media are ineffective in helping to remove organically complexed iron. Even in the absence of organically complexed metals, as the TOC levels increase in the feedwater, there is a tendency for the organic material to coat the medium, seal off the adsorptive sites, and significantly reduce or even eliminate the medium's ability to function. In the presence of significant TOC concentrations, the medium must continually be cleaned with a strong oxidizing agent or even a detergent. For some oxide-coated media, this organic fouling can be permanent and would ultimately require that the medium be replaced to restore its adsorptive capacities. Use of adsorptive or catalytic media in the presence of TOC greater than 1.0 mg/L is not recommended.

Manganese Dioxide Ore The ability of manganese dioxide ore to aid in the removal of iron and manganese from water has been known since the 1920s. In the 1930s, naturally occurring manganese dioxide ore (known as pyrolusite) was substituted for the filter media in several water treatment plants in an attempt to take advantage of this ability. These attempts were generally not successful because of problems in expanding the media during backwashing and effectively cleaning the bed. Manganese dioxide ore is very heavy, weighing in excess of 120 lb/ft^3 (1,900 kg/m^3), and requires significantly higher backwash rates than more conventional filter media for effective cleaning of the filter bed. The filter beds where pyrolusite was tried did not have the hydraulic capacity to allow proper backwashing of the filters and media cleaning. As a consequence, the media soon fouled and became ineffective.[11]

Small package treatment systems of proper hydraulic design have successfully used manganese dioxide ore media for the removal of iron and manganese. The backwash systems in these units are designed for the media and are capable of very high backwash flow rates of 25 to 30 gpm/ft^2 (61 to 73 m/h). This high backwash flow rate will expand and clean the media and remove the oxidized metals. However, the inherent problems associated with these high backwash rates, as well as the resulting volumes of backwash water, have prevented the use of MnO_2 ore media for large-scale systems.

Manganese dioxide ore will act as a catalyst in the oxidation of iron. The iron is adsorbed onto the surface of the media and permits the oxidant to operate more effectively and efficiently. Since iron oxidation is relatively fast in the presence of common oxidants at neutral pH, manganese dioxide ore is not considered when iron is found alone. If manganese dioxide ore is installed to aid in the oxidation of dissolved manganese, it will provide a mutual benefit in the oxidation of the soluble iron in the water.

In the absence of an oxidant, manganese is removed only via adsorption onto the manganese dioxide medium. The chemical reaction for oxide-coated media is listed in Table 14–3. Knocke and coworkers[12] reported no evidence of a chemical reaction between manganese and MnO_2 media. Thus, if the Mn(II) is not eventually oxidized, the adsorption sites on a medium will ultimately be used up and the Mn(II) will pass through the filter. Alternatively, if there is a change in water chemistry, the Mn(II) could be released from the filter into the finished water. It is for this reason that either ClO_2, Cl_2, $KMnO_4$, or O_3 is added to water being applied to manganese dioxide media filters. Use of one of these oxidants will convert the Mn(II) that is attached to the media grains to its insoluble form, Mn(IV). This freeing up of the adsorption sites on the media is referred to as *regeneration*. The insoluble Mn(IV) is then removed during backwashing of the filter. To maximize the benefits derived from using these oxidants, care must be used in the selection of the oxidant and in establishing the proper dosage.

The pH of the water and the number of adsorption sites available on the media surface have a significant impact on the manganese adsorption capacity of the manganese dioxide ore (or, for that matter, manganese dioxide coated media). Knocke and coworkers[12] reported that for pH values of less than 6.0 minimal Mn(II) was noted. Above a pH of 7.0, the number of adsorptive sites available increased significantly.

Manganese Greensand Alternative media have been developed to take advantage of the adsorptive capacity of manganese dioxide while eliminating the hydraulic problems associated with the medium's heavy weight. Manganese greensand is a product manufactured by treating New Jersey glauconite with manganous sulfate and potassium permanganate to provide an active supply of iron and manganese oxides on the sand grains. When the oxidizing power of the bed is exhausted, the bed is regenerated with permanganate or chlorine and returned to service. These beds are quite effective in high-carbonate, iron-bearing water, but they are exhausted or fouled quickly if other reducing substances, such as organic matter, nitrogenous matter, or hydrogen sulfide, are present. Greensand has the ability to adsorb and then catalyze the oxidation of iron and manganese and then provide filtration. Because of greensand's small size, hydraulic loading rates must be kept low; the filter bed can bind off quickly, resulting in short filter runs. When the amount of precipitate is large, a layer of crushed anthracite coal is usually placed over the exchange medium to reduce the loading on the greensand and thus prolong filter runs.

Because greensand media regeneration is time-consuming, continuous regeneration is commonly employed at plants in North America. During continuous regeneration, a solution of $KMnO_4$ is fed continuously into the raw-water line ahead of the filter to reduce the amount of soluble iron and manganese applied to the filter. The dose of $KMnO_4$ is critical. Underdosing results in manganese breakthrough, and overdosing produces pink water.

Manganese Dioxide Coated Silicon Dioxide Another oxide-coated medium that has been used for the removal of iron and manganese is manganese dioxide coated silicon dioxide. Known by the trade name ''Birm,'' this medium was developed in the 1930s and first used in 1939. The primary use of manganese dioxide coated silicon dioxide is for iron removal. The unit weight of the medium is only 50 lb/ft^3 (800 kg/m^3), which reduces the required backwash flow to 10 to 12 gpm/ft^2 (24 to 29 m/h). The adsorptive and catalytic capacity of the medium is not used up during operation and requires only frequent backflushing to remove the accumulated oxidized metals.

This backflushing process is a short-duration water flush of the medium that serves to free up adsorption sites, thereby regenerating the medium. The medium is sensitive to the presence of H_2S and TOC in the feedwater. When the medium operates in the presence of H_2S, the sulfur accumulates on the medium and can permanently foul the bed. TOC can also foul the medium and eliminate its effectiveness as a catalyst. Organically bound iron, which may be present with the TOC, will not be affected or removed by manganese dioxide coated silicon dioxide.[11]

Formation of Oxide Coating on Conventional Media Over time, during the operation of a conventional manganese removal system using standard media, a manganese dioxide coating will develop on the medium. Just as with manganese dioxide ore, this manganese dioxide coating aids in the removal of soluble manganese from the feedwater via adsorption onto the medium. Knocke attributed the rate of manganese removal to the number of adsorption sites available on the medium.[12] As the manganese is adsorbed onto the medium, the adsorption sites are used up and the medium loses its effectiveness for manganese removal. The medium must then be regenerated to regain its ability to remove soluble manganese. This regeneration can be accomplished either in a batch mode by using $KMnO_4$ or continuously by adding $KMnO_4$ to the water prior to the filters.

A utility can build up the oxide coating on the media by just operating the manganese removal system and waiting for the coating to develop. Alternatively, it can actively promote the development of a manganese dioxide coating by batch mode as described by Bailey.[11] The batch mode process involves applying a concentration of 100 mg/L of $KMnO_4$ to the filter medium for a period of 24 hr to quickly establish the manganese dioxide coating. This procedure will not have to be repeated if the filter is kept in an oxidized state with continuous regeneration.

Ion Exchange (Zeolite Softening)

Most modern ion exchange resins are polymers chemically bonded to an acidic or basic functional group (AWWA 1990). The major use for ion exchange in water treatment is water softening. The calcium and magnesium are exchanged for sodium via SAC resin.[1] In addition to water softening, sodium cation exchange and hydrogen cation exchange units can remove ferrous iron (Fe(II)) and manganous manganese (Mn(II)). Ion exchange units, because of their cost, are typically used only for household or small community systems (less than 100 gpm). For this reason, ion exchange is not typically used specifically for iron and manganese removal. Instead, such removal is a secondary benefit of softening systems.

Iron and manganese removal is effective only on the soluble form of the metals in ion exchange. If the metals have been oxidized prior to reaching the resin, the resin will quickly be fouled and become ineffective. Therefore, the water should not be exposed to oxygen, chlorine, or other oxidants prior to the resin bed.

The ion exchange resin can also be fouled by TOC in the water. If TOC is present in the water source, testing of the ion exchange system is recommended prior to installation.

Lime Softening

Chemical precipitation of iron and manganese is brought about at pH values approximating the metals' isoelectric point of 9.4. Because of the pH shift during lime softening, manganese and iron are effectively removed. In the absence of oxygen, iron precipitates as ferrous hydroxide and manganese as manganous hydroxide. The solubility of $Mn^{+2}(OH)_2$—as calculated from its solubility product, i.e., $[Mn^{+2}][OH]^2 = 7.1 \times 10^{-15}$—is 3.9 mg/L as Mn at a pH of 9.0. The solubility of both $Mn^{+2}(OH)_2$ and $Fe(OH)_2$ decreases 100-fold for each unit rise in pH. Significant precipitation of ferrous hydroxide and manganous hydroxide generally proceeds only for pH values greater than 9.5 and 10.0, respectively.[13] Precipitation of iron and manganese with lime is not usually cost-effective unless lime treatment is also required for hardness reduction.

Manufacturers' Proprietary or Package Processes

Given the prevalence of iron and manganese in groundwater supplies, several manufacturers have developed package-type proprietary processes that efficiently and effectively remove the metals. These units are usually skid mounted at the factory to reduce the engineering and construction costs. Once installed, the processes usually offer operational advantages that will reduce the operation and maintenance costs of water production. Individual manufacturers should be contacted for details concerning the operation of the different processes.

BIOLOGICAL REMOVAL OF IRON, AMMONIUM, AND MANGANESE

Biological processes for the removal of iron, ammonium, and manganese from drinking water were developed in Europe during the 1980s. Their discovery was quite by accident. When iron bacteria were present in a treatment system, it was noted in some treatment facilities that removal efficiencies improved. Based on this observation, processes were developed that optimized the bacterial population within the treatment train to remove iron; later, biological processes were developed to remove manganese and ammonium.[14]

Advantages of the biological process include:

- Significantly higher filter loading rates
- Longer filter runs
- Lower backwash flow rates for shorter time periods
- Flexibility of using either gravity or pressure filtration
- Reduced sludge production and handling
- Elimination of the need for chemical oxidant addition
- The fact that, for some systems, the only chemicals required are for pH adjustment
- The fact that, for many biological systems, supplemental aeration is the only process required
- Consistently higher-quality finished water
- A stabler process
- A process that is easier to control
- Improved economics
  - Construction costs that range from 60 to 90 percent of those for conventional treatment processes

- Reduced O&M costs due to elimination of chemical feed equipment, reduced sludge handling, longer filter runs, and lower backwash water requirements
- Reduced labor requirements due to less equipment to maintain and a stabler process

There are also disadvantages to biological treatment:

- It is sensitive to the presence of H_2S and zinc.
- Iron and manganese usually cannot be removed effectively in a single reactor because of the significantly different conditions required for iron and manganese removal. A two-stage process is generally required when both metals are present.
- Some researchers have successfully removed both iron and manganese in a single reactor; however, the filtration rates are drastically reduced and some of the advantages of the process are lost.
- An initial start-up period is required for the process to establish the biomass in the reactor. This time is significant for manganese removal. Shorter start-up periods are also required after shutdowns of the system.
- Once started, the systems operate better if run continuously, which could impact utility operations.
- Ammonium must be removed prior to biological manganese removal.

General Principles and Operating Conditions of the Biological Process

Biological Iron Removal Iron bacteria are commonly found in raw waters and can multiply rapidly on sand filters under the proper growth conditions. Iron-oxidizing bacteria include a variety of different species of bacteria, with the most common being *Gallionella ferruginea.* Other species of iron-oxidizing bacteria include *Leptothrix, Crenothrix,* and *Siderocapsa,* to name a few examples. Each of these species has different growth and iron oxidation characteristics. The environment will select the predominant species, and selection is dependent on pH, temperature, dissolved oxygen, salinity, and organics content in the feedwater.[5]

Iron-oxidizing bacteria are able to oxidize ferrous ions (Fe^{2+}) to ferric ions (Fe^{3+}) and accumulate a metal precipitate. For *Gallionella,* one of the most common iron-oxidizing bacteria found in water treatment filters and an obligate autotroph, this oxidizing process is described as an exothermic reaction for energy. The energy released by this reaction allows the bacteria to assimilate the carbon from CO_2. *Leptothrix* and *Crenothrix* are thought to be facultative autotrophs using enzymatic oxidation of iron. *Siderocapsa* species are thought to be obligate heterotrophs. In practice, iron oxidation in water treatment plants is usually thought to occur via two mechanisms: (1) intracellular oxidation by enzymatic action, and (2) extracellular oxidation by the catalytic action of excreted polymers.[5]

The growth of the bacteria on the filter medium consists of two main types. The first type of growth is filamentous and fixed growth that forms dense, cohesive flocs that are easy to manage, operate, and control during backwashing. *Gallionella, Leptothrix,* and *Crenothrix* species fall into this category. The second type of growth involves independent and small-sized bacteria forming small and fragile floc that are difficult to fix on the medium, making it easily affected by sudden hydraulic changes across the filter. *Siderocapsa* species grow in this manner.[14] Once established in a filter, the biomass is not always stable and the predominant species can change over a period of time. The operator has little control over the species that populates the filter. Slight changes in either pH, dissolved oxygen levels, or redox potential in the feedwater may result in one species becoming predominate over another. The common iron-oxidizing bacteria species all remove iron with similar efficiency, and the operator is usually not concerned with the specific species in the filter. However, some species may require slight adjustments in the operation of the plant, especially backwashing, and the operator needs to be on the lookout for these changes.

The biological iron removal process is dependent on the pH and the redox potential of the source water. If the redox potential is allowed to go too high, the iron will be chemically oxidized and the process efficiency will drop. Because the filters are not designed to handle a chemical precipitate, this condition can lead to filter breakthrough of nonsoluble iron. The pH and redox potential conditions conducive to biological oxidation were charted by Mouchet.[5] The conditions required for biological iron removal are described in Table 14–4. Among the optimum conditions are a pH of 6 to 7.6 and a redox potential of 0 to 400 mV (NHE). A schematic for a typical two-step high-rate biological process is shown in Figure 14–4.

Typical operating conditions for biological iron removal include:

- Filtration rates of 5 to 15 gpm/ft^2 (12 to 37 m/h), depending on the iron concentration
- Supporting media with an effective size of 1.0 to 1.5 mm
- Media depth of 36 to 60 in. (0.9 to 1.5 m)
- A start-up or seeding period generally of 2 to 7 days to establish the biomass in the reactor
- A relatively constant flow rate or stable water quality. The process does not react quickly to rapid changes in the hydraulic loading on the filter or changes in source water quality.
- Filter run times of 24 to 72 hours
- Backwash conditions:
  - Unchlorinated backwash supply water
  - Air scour (usually implemented)
  - Low-rate backwash at 4–6 gpm/ft^2 (10–15 m/h)
  - High-rate backwash at 10–12 gpm/ft^2 (24–29 m/h)
  - Total backwash duration of 5 to 10 minutes

Recovery of the biomass after normal operational shutdowns of 1 to 12 hr or after backwashing is rapid, generally requiring less than 10 minutes. It is recommended that a filter-to-waste cycle be employed on each start-up of the filter system following short-term shutdowns and backwashing. For process shutdowns of several days, weeks, or even months, the restart time is significantly longer; however, it is usually faster than the initial reactor seeding. To ensure a high-quality finished water following a system shutdown of more than 48 hr, it is necessary to provide for a start-up cycle that could last from 10 minutes to several hours.

If the medium in a biological iron filter is kept wet during a filter shutdown, it will recover rapidly from shutdowns of a few days to weeks or even months. If the filters are drained and dry out during process shutdowns, the biomass will have to reestablish

TABLE 14-4. Reactions and Required Conditions for Biological Removal of Iron, Manganese, and Ammonium

Substance	Reaction	Conditions	Comments
Iron	Fe^{2+} + bacteria → bacteria + FeOOH (with proper conditions for redox potential, pH, O_2)	Temperature > 50°F (10°C) pH 6 to 7.6 Redox potential 0–400 mV (NHE) H_2S <0.1 mg/L Zinc <0.45 mg/L	Not all waters are suitable for biological iron removal. If the pH, DO, and redox potential are too high, the iron will be chemically oxidized and could pass through the filter. Seeding the reactor takes 2 to 7 days. Reactor does not respond quickly to changes in water quality.
Manganese	Mn^{2+} + bacteria → bacteria + MnO_2 (with proper conditions for redox potential, pH, O_2)	Temperature >50°F (10°C) pH 6.5 to 8.0 Redox potential > 200 mV (NHE) DO >4.0 mg/L NH_4 <0.1 mg/L Fe <0.1 mg/L	Conditions are significantly different than with biological Fe removal. Manganese is not usually removed in the same vessel as iron at high filtration rates. Both Fe and Mn have been removed in a single filter at loading rates of 2–3 gpm/ft² (4.9–7.3 m/h). If ammonium is present, it must be nitrified prior to biological Mn removal. Seeding the reactor at start-up usually takes 6 to 12 weeks. Reactor does not respond quickly to changes in water quality.
Ammonium	Step 1: $2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 2H_2O + 4H^+$ (in the presence of DO and Nitrosomonas bacteria) Step 2: $2NO_2^- + O_2 \rightarrow 2NO_3^-$ (in the presence of DO and Nitrobacter bacteria)	Temperature > 46°F (8°C)—reaction stops below 41°F (5°C) pH 6.0 to 8.0 DO >1.0 mg/L	The two reactions require 3.55 mg of O_2 per 1 mg of NH_4^+ They consume 11.2 mg HCO_3^- per 1 mg of NH_4^+. Nitrifying bacteria will ‘‘outcompete’’ the Mn bacteria, so the NH_4^+ must be nitrified before biological Mn removal can proceed. Nitrifying bacteria are very slow-growing organisms, requiring 6 to 8 weeks to seed. They do not respond quickly to changes in water quality. For NH_4^+ concentration <1.0 mg/L, NH_4^+ can be removed in same filter as Mn. For NH_4^+ concentration between 1.0 and 2.0 mg/L, filter must be aerated. For NH_4^+ concentration >2.0 mg/L, design system specifically for NH_4^+.

Source: See reference 5.

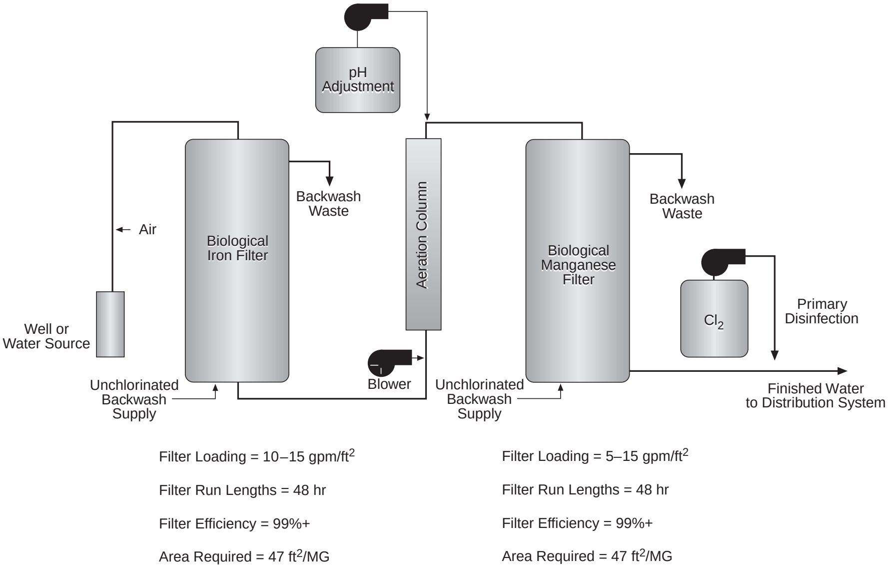


Fig. 14-4. Biological filtration for iron and manganese removal

itself for the process to regain its efficiency. The actual length of time required for this reseeding will depend on the conditions at the facility.

Mouchet reported solids retention on biological iron filters to be 3 to 5 times greater than for conventional oxidation filtration processes.[5] This additional loading capacity results in longer filter runs and produces a denser backwash floc with superior settling and handling characteristics. The backwash water supply must be unchlorinated to protect the biomass on the filter. The backwashing process could be described as more of a rinsing or flushing that removes the old and dead bacteria and allows new bacteria to take over. The low backwash rates used for biological iron filters reduce the required size of the backwash supply system and backwash storage basin, and they increase the water production efficiency of the treatment plant. The backwash solids can readily be handled in the sanitary sewer system or can be dried and disposed of in a landfill.

Biological Manganese Removal Biological manganese removal requires significantly different conditions than biological iron removal. Manganese bacteria are strictly aerobic organisms and prefer a higher pH than iron bacteria. Saturated dissolved oxygen conditions are required for manganese bacteria to grow and thrive. As mentioned earlier, to allow the high filter loading rates normally used with biological filtration, manganese and iron are not generally removed in the same biological reactor. Figure 14–4 illustrates a two-stage high-rate biological process for removing iron and manganese.

Manganese oxidation by bacteria takes place by any one of three processes: (1) primary intracellular oxidation by enzymatic action; (2) adsorption of the dissolved manganese at the surface of the cell membrane, followed by oxidation by enzymatic action; or (3) simple catalysis in the vicinity of the cell, under the influence of the biopolymers secreted by the bacteria.[5] The actual process used to oxidize the manganese is dependent on the predominant bacteria species and the environmental conditions in the filter. In all cases, the oxidized manganese is deposited as MnO_2 as described in Table 14–4.

Typical operating conditions for biological manganese removal include the following:

- Filtration rates of 5 to 15 gpm/ft^2 (12 to 37 m/h), depending on the manganese concentration
- Effective size of supporting media of 0.8 to 1.0 mm
- Media depth of 36 to 60 in. (0.9 to 1.5 m)
- Saturated dissolved oxygen conditions
- A start-up period generally of 6 to 12 weeks to establish the biomass
- A relatively constant flow rate and stable water quality. The process does not react quickly to rapid changes in the hydraulic loading on the filter or to changes in water quality.
- Filter run times of 24 to 72 hours
- Backwash conditions:
  - Unchlorinated backwash supply water
  - Air scour (usually implemented)
  - Low-rate backwash at 4–6 gpm/ft^2 (10–15 m/h)
  - High-rate backwash at 10–12 gpm/ft^2 (24–29 m/h)
  - Total backwash duration of 5 to 10 minutes

Biological manganese removal is more difficult to establish than biological iron removal. It generally takes 6 weeks to 2 months for the bacteria to seed the reactor, but once established the biomass is reasonably stable and will recover from backwashing and short shutdown periods of 1 to 12 hours within 10 minutes at typical filtration loading rates. As with biological iron removal, it is recommended that the treatment facility be able to filter to waste.

If the biomass in the manganese removal reactor is kept wet during process stoppages, it will recover readily from shutdowns of a few days to weeks or even months. This recovery time is significantly shorter than the initial seeding time for the reactor, generally taking 60 minutes to a few days depending on the flow rate and the length of system shutdown. If the biological filters are allowed to drain and dry out during process shutdowns, the recovery time will be slower but will still not be as long as the original seeding and media-ripening period. The operator should plan for this recovery time and adjust plant operations accordingly to protect consumers when the plant is restarted following maintenance shutdowns.

In pilot tests of a biological manganese reactor, researchers noted a significant spike in the manganese levels of the filter effluent following extended shutdowns of several days to several weeks.[15] On start-up, these levels were two or three times higher than those of the raw water. The researchers speculated that this was the result of bacteria sloughing off the filter into the finished water on filter restart. The spike was short, and the finished water manganese levels dropped rapidly after two or three filter volumes of water had passed through the bed. Utility personnel need to be careful when restarting the filters after extended shutdowns to prevent this spike from entering the distribution system. It is recommended that several filter volumes be wasted following extended filter shutdowns.

Typical filter loading rates for biological manganese filters are dependent on the manganese concentration and generally range from 4 gpm/ft^2 (10 m/h) for 2.0 mg/L Mn to 16 gpm/ft^2 (40 m/h) for 0.5 mg/L or less Mn.[4] Filter runs are typically 24 to 72 hr, depending on the raw water quality.

As with biological iron removal, solids retention on a biological manganese filter is 3 to 5 times higher than with conventional filtration. With these filters, the objective of backwashing is not to clean the filters but to provide additional space for bacteria to grow. The backwash water is unchlorinated to protect the biomass and is applied at a lower rate than for conventional filter backwashing. The dead or old bacteria are rinsed or flushed from the filter during the backwashing cycle, which is of short duration. This reduced backwash requirement results in smaller facilities in terms of both backwash supply and backwash storage and handling. The backwash residuals can either be wasted to the sanitary sewer system or dried and disposed of in a landfill.

Biological Ammonium Nitrification If ammonium is present in the water and biological manganese treatment is going to be used, the ammonium will be nitrified first before the manganese can be oxidized. The designer must be aware of these conditions and construct the system to handle the ammonium present. The biological conversion of ammonium to nitrates is a common wastewater process that is well documented in textbooks and the literature. The removal or nitrification of ammonium in drinking water is not a common process. This is due to the fact that ammonium usually occurs in very low concentrations in a water source and these low concentrations are oxidized during chlorine addition. Because of its interference with biological manganese removal, the process is described here. Nitrification of the ammonium is accomplished by obligate autotrophic aerobic bacteria that oxidize the ammonium in the presence of oxygen.[14] The conversion of ammonium to nitrates is described by the reaction given in Table 14–4.

Typical operating conditions for ammonium nitrification include the following:

- Filtration rates of 2 to 5 gpm/ft^2 (5 to 12 m/h), depending on the NH_4^+ concentration. Higher filtration rates may be possible for low NH_4^+ concentrations.
- For NH_4^+ concentrations less than 2.0 mg/L, supporting media of 1.0 to 1.5 mm effective size, with a media depth of 36 to 60 in. (0.9 to 1.5 m)
- For NH_4^+ concentrations greater than 1.0 mg/L, a filter bed that is aerated to provide sufficient dissolved oxygen for nitrification
- For NH_4^+ concentrations greater than 2.0 mg/L, provide special nitrification/ammonia removal process to reduce ammonia concentration below 2 mg/L
- Saturated dissolved oxygen conditions
- A start-up period generally of 6 to 12 weeks to establish the biomass
- A relatively constant flow rate and stable water quality. The process does not react quickly to rapid changes in the hydraulic loading on the filter or to changes in water quality.
- Filter run lengths of 48 to 96 hours
- Backwash conditions:
  - Unchlorinated backwash supply water

- Air scour (usually implemented)
- Backwash rates of 10–12 gpm/ft^2 (24–29 m/h)
- Backwash duration of 4 to 8 minutes

The speed of the nitrifying reaction stabilizes close to the optimum value at DO levels greater than 2.0 mg/L. The reaction requires 3.55 mg of O_2 for every 1.0 mg of NH_4^+(4.56 mg O_2/mg NH_4^+ as N). When the NH_4^+ concentration is greater than 1.0 mg/L, the filter medium must be aerated to sustain the nitrification of the ammonium. Temperature also affects the speed of the reaction. Below 41°F (5°C), the reaction slows down significantly and essentially stops. The reaction consumes alkalinity (1.0 mg of NH_4^+ consumes 11.2 mg HCO_3^- or 7.14 mg alkalinity as $CaCO_3$ per mg NH_4^+ as N), and the pH will be lowered during nitrification. In poorly buffered waters, alkalinity may have to be added to the water to keep the pH from dropping too low. The reaction will not take place at a pH below 6.0.[14] The concentration of ammonium will determine the type of nitrifying process selected. For NH_4^+ concentrations less than 2.0 mg/L, a sand medium is adequate for bacteria support. For higher concentrations, a nitrifying packed tower or multiple stages may be required. Pilot testing is highly recommended when a utility is considering the use of biological methods for ammonium nitrification.

The food source for the nitrifying bacteria in drinking water treatment is very limited, and the life span of the biomass must be significantly longer than that typically found in wastewater treatment. Controlling the biomass formation and concentration is essential to process success and becomes more difficult with increasing concentrations of ammonium. For ammonium concentrations greater than 2.0 mg/L, large quantities of biomass are required. As the biomass develops, it can produce significant hydraulic problems with the system, sealing off the flow and reducing hydraulic capacities. Means of preventing this problem from occurring while still maintaining treatment efficiency must be studied during the pilot test.

Conditions for biological ammonium removal are very similar to those for biological manganese removal. The nitrifying organisms will ‘‘outcompete’’ the manganese bacteria; therefore, the ammonium must be nitrified prior to biological manganese removal. Generally, for waters containing less than 1.0 mg/L of ammonium, the ammonium and manganese will be removed in the same filter vessel and an additional step is not required. During pilot testing for biological manganese removal, the efficiency of ammonium nitrification must be investigated.

Backwashing a nitrification filter is a sensitive operation, and care must be taken to avoid overwashing the medium. The bacteria grow slowly and do not recover quickly from upsets or changes in flow. Further study is needed to develop systems that work simply and consistently for ammonium concentrations greater than 2.0 mg/L.

DESIGN AND OPERATION OF IRON AND MANGANESE REMOVAL SYSTEMS

Water Quality Evaluation and Process Selection

The first step in either developing a new iron and manganese removal facility or upgrading an existing facility is to fully evaluate the raw-water quality and develop treatment and finished water quality objectives. Such raw-water quality parameters as

Fe and Mn concentrations, color, TOC levels, pH, H_2S levels, NH_4^+ concentration, hardness, and dissolved oxygen concentrations can all have an influence on the treatability of the water and the treatment process selected. No one treatment process will apply to all water sources. In fact, a process that will work in one instance will not always work in the next. Treatment goals must be established that account for finished water Fe and Mn levels, color, pH, and trihalomethane formation potential (THMFP), but these are not the only concerns. Obviously the finished water must meet all current and projected regulations. Other criteria to consider include capital costs, ease of operation, backwash handling characteristics, cost of operation, regulatory acceptance of the process, expandability, flexibility, and the potential for phasing of the facilities.

Bench-Scale Studies

An important first step in designing or upgrading an iron and manganese removal facility is a bench-scale study. Such studies can be very useful in the preliminary determination of the suitability or feasibility of a treatment process train at a fraction of the cost of a pilot study. The engineer can evaluate iron and manganese oxidation chemicals, establish chemical dosages, determine sequence of chemical addition, estimate oxidation detention times, determine filtration efficiency, and quickly eliminate several treatment options. Working out some of the details of the iron and manganese treatment train in the lab is more economical and easier to adjust than during the pilot study phase of the investigation. It should not be assumed, however, that a bench-scale study will be a cheap replacement for a pilot study. The bench-scale study is only a way to eliminate some possible iron and manganese treatment alternatives and make the pilot study more efficient.

Pilot Studies

The second step in developing an iron and manganese removal system is to conduct a pilot study. This step is necessary to establish the process design criteria for a particular water quality, thereby allowing the engineer to size system components. Pilot studies can provide valuable information on the operating characteristics of the proposed treatment plant. They can determine if a process will work, and they can identify problems before construction of the full-scale facility, often allowing the utility to avoid costly repairs or modifications either during or following construction. Depending on the type and number of processes to be investigated, who operates the pilot plant, who constructs the pilot unit, and the overall pilot study objectives, pilot studies can require anywhere from 4 weeks to 6 months to complete and can cost anywhere from $15,000 to $75,000.

An iron and manganese pilot study should be designed to test the limits of the proposed treatment process, the goal of which is to see how heavy a process can be loaded before it breaks or fails to meet the stated treatment objectives. Knowing the limits of a process will allow the engineer to design the system for maximum effectiveness and efficiency. In conjunction with this guideline, the pilot test should be designed to develop the following design and operational criteria:

- Full design criteria for all aspects of the proposed iron and manganese removal process to include but not necessarily be limited to:

  - Any expected water quality changes over the life of the facility
  - Seasonal operational changes, if they exist, such as temperature, pH, or other water quality parameters
  - Any potential water quality changes due to blending of multiple wells or water sources
- Aeration requirements (if used)
- Chemical dosage requirements, including:
  - Oxidation chemical dosing points
  - Coagulant chemical dosing points (if used)
  - Required detention or reaction times
  - Selected chemicals
  - Dosing quantity
- Filter characteristics
  - Pressure or gravity filters
    - Effective media size
    - Media material
    - Media uniformity coefficient
  - Media depth
- Hydraulic characteristics
  - Detention tank volume (if used)
  - Clarifier/flocculator requirements (if used)
  - Filter loading rates
- Filter run characteristics
  - Empty bed head loss
  - Head loss/pressure loss buildup during filtration over time
  - Length of filter run
- Backwash requirements
  - Backwash rates (low and high)
  - Air scour requirements (if used)
  - Length of backwash
  - Filter-to-waste requirements
  - Filter recovery rates
- Washwater residual characteristics
  - Quantity of suspended solids (amount per volume of water produced)
  - BOD_5
  - Settleability

Pilot-testing units can be set up to examine several different iron and manganese removal processes. This will allow the utility to select the process that will best serve their needs and will be the most cost-effective to build and operate. Often, especially with difficult water qualities, pilot testing becomes an exercise in trial and error as the engineer looks for the right combination of process, aeration, chemicals, chemical dosing locations, detention times, filter loading rates, media, and filter run lengths to meet the established process objectives. A process that appears to work on paper may experience operational difficulties during pilot testing as a result of some unforeseen water chemistry complication. It is better to identify such problems during pilot testing than to discover a problem after expensive treatment facilities have been constructed.

Iron and Manganese Treatment Plant Layout

The typical iron and manganese removal plant is a stand-alone facility providing treatment at a well head or for a wellfield. Iron and manganese oxidation and filtration may occur as a secondary benefit of other treatment processes. Those types of plants will not be considered here. Iron and manganese treatment plants are simple in design and layout. A typical plant will consist of a chemical feed system, pressure filters, and backwash water processing facilities. Gravity filters may be used for iron and manganese removal, depending on the hydraulic conditions at the facility. Pumps for process or backwash supply may be required, as well as standby power generation. A small lab or control room provides the operator a convenient place to monitor plant performance and prepare regular reports. Iron and manganese removal plants often operate unstaffed, being controlled instead by a supervisory control and data acquisition (SCADA) system.

The layout of an iron and manganese plant should account for seasonal variations in water demand and for projected system growth. It is recommended that a minimum of two filters be provided in the treatment plant. To minimize the size of the backwash facilities and to provide flow flexibility, it is recommend that the maximum filter size be between 400 and 700 gpm (25 and 44 l/s). During the low-demand months of the winter, filters can be taken off line to reduce operations and maintenance costs and provide an opportunity for maintenance and repair.

The typical iron and manganese removal treatment plant uses pressure filters. These types of filters offer the advantage of a flow-through configuration and eliminate the need to repump. The finished water is pumped through the filters either directly into the distribution system or into the system storage reservoir. A gravity filtration system, on the other hand, usually requires a clearwell and pumps to lift the filtered water into the system, resulting in higher capital costs.

Washwater Recycle

A large volume of water is used during a backwash cycle to clean and restore a filter's capacity. This volume of water can account for as much as 5 percent of the total water produced during the filter run. In many instances, this water is wasted to the sanitary sewer system and lost. The large slug flow can overload the sewer system and cause problems at the wastewater treatment plant. To alleviate this condition, the treatment facility can install holding tanks or basins to meter the washwater into the sewer system; this will reduce the hydraulic loading on the sewer system. The installation of holding tanks or basins, however, does not reduce the amount of water lost to the sewer system during every backwash cycle.

Water conservation concerns are driving many utilities to consider washwater recycle for their iron and manganese removal plants. Washwater recycle is the process of returning the washwater supernatant to the head end of the treatment plant for reprocessing rather than just dumping it down the sewer. By recycling the water after settling in a covered tank or basin, the majority of the water is returned to the system and only a small fraction is released to the sewer system. Little research has been conducted on the effects of washwater recycle on treatment plant efficiency and finished water quality in iron and manganese removal plants. To minimize any potential adverse impacts of recycle, it is recommended that recycle flow be continuous and that recycle flow be kept to below 10 percent of the total flow.

If a utility is considering adding washwater recycle to their facility, the backwash tank or basin should be covered and protected.

Process Control

On-line finished water turbidity or color instruments provide process monitoring and control in an iron and manganese treatment plant. To determine the levels of these metals, the operator takes regular grab samples that are either analyzed in the field or submitted to a commercial lab. To evaluate and control oxidant dosage, some plants use on-line chlorine analyzers or redox potential probes. Once the chemical dosage is set by manually adjusting the stroke length on the chemical feed pump, the feed pump speed is varied to match the flow through the treatment facility as measured by the raw water meter. This control system is commonly referred to as flow pacing the feed pumps. The head loss or solids accumulation in the filter medium is monitored by pressure sensors on the inlet and outlet of the filter. When the pressure loss across the filter reaches a preset level, it will automatically trigger the backwash cycle using motor-controlled valves. A series of alarms monitor critical process set points and have the ability to shut down the plant if conditions warrant. The modern iron and manganese removal plant is usually unattended for significant periods of time. Programmable logic controller (PLC) monitoring tools and the associated alarm systems can alert the operator of plant problems from remote locations through the SCADA system via computer modem linkup.

An iron and manganese system treating a groundwater source has only minor variations in raw-water quality. Therefore, once fine-tuned and trimmed, the system will operate satisfactorily with only minor process adjustments by the operator. If the treatment plant processes water from several wells, minor adjustments in the process may be required as wells come on or off line. Seasonal variations in water quality may also result in required process changes. The operator must be careful not to get complacent in the operation of the system because problems can develop that would be difficult and expensive to correct.

Conversion to Biological Processes

In Europe, many conventional physical chemical iron and manganese removal plants are converting to the biological process. These plants are making this change to take advantage of the features of the biological processes, to reduce operations and maintenance costs, to expand plant capacity within the same plant footprint, and in many cases to improve finished water quality. To date, North American plants have not shown a similar conversion trend because of an unfamiliarity with the biological process, an acceptance of lower-quality water, a lack of desire to commit the monetary resources required to make the conversion, and a lack of desire to abandon the current system of treatment. However, the situation is likely to change as utilities and engineers become more familiar with the biological process, as the public demands higher-quality water and consents to pay for it, and as additional capacity requirements conflict with the lack of available space for expansion.

REFERENCES

1. AWWA (American Water Works Association), *Water Quality and Treatment,* 5th ed., McGraw-Hill, New York, 1999.
2. APHA, AWWA, and WEF (American Public Health Association, American Water Works Association, and Water Environment Federation), *Standard Methods for the Examination of Water and Wastewater,* 20th ed., APHA, Washington, D.C.,1998.
3. AWWA and ASCE (American Water Works Association and American Society of Civil Engineers), *Water Treatment Plant Design,* 3rd ed., American Water Works Association, Denver, Colo., 1998.
4. Bean, E. L., ''Progress Report on Water Quality Criteria,'' *JAWWA* 54:1313, 1962.
5. Mouchet, Pierre, ''From Conventional to Biological Removal of Iron and Manganese in France,'' *JAWWA* 84(4):158, 1992.
6. Knocke, W., Shorney, H., and Bellamy, J., *Impacts of Dissolved Organic Carbon on Iron Removal,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1993.
7. Knocke, W., Van Benschoten, J., Kearney, M., Soborski, A., and Reckhow, D., *Alternative Oxidants for the Removal of Soluble Iron and Manganese,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1990.
8. Robinson, R. B., Reed, G. D., Christodos, D., Frazer, B., and Chidambariah, V., *Sequestering Methods of Iron and Manganese Treatment,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1990.
9. James M. Montgomery, Consulting Engineers, Inc., *Water Treatment Principles and Design,* John Wiley & Sons, New York, 1985.
10. Kawamura, S., *Integrated Design on Water Treatment Facilities,* New York: John Wiley & Sons, 1991.
11. Clark Corp., ''Iron Removal with Birm,'' Technical report 89-WT-8, August 4, 1989.
12. Knocke, W., Occiano, S., and Hungate, R., ''Removal of Soluble Manganese from Water by Oxide-Coated Filter Media,'' American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1990.
13. Stumm, W., and Lee, G. F., ''Oxygenation of Ferrous Iron,'' *Ind. Eng. Chemistry* (American Chemical Society) 53:143, 1961.
14. Bourgine, F. P., Gennery, M., Chapman, J. I., Kerai, H., Green, J. G., Rap, R. J., Ellis, S., and Gaumard, C., ''Biological Processes in Drinking Water Treatment: The Case of Saints Hill,'' Paper presented at a joint meeting of the Institution's South Eastern Branch and Scientific Section, June 23, at the Europa Gatwich Hotel in Crawley, United Kingdom, 1993.
15. Larson, A., ''Bacteria Becomes the Solution Rather Than the Problem for Water Treatment,'' in *Proceedings of the 1996 Annual Conference and Exposition,* American Water Works Association, Denver, Colo., 1996.

CHAPTER 15

Membrane Treatment

INTRODUCTION

Membrane filtration processes for the treatment of potable water covered in this chapter are reverse osmosis, nanofiltration, ultrafiltration, microfiltration, and electrodialysis reversal. The reverse osmosis, nanofiltration, and in some cases ultrafiltration processes use the ability of semipermeable membranes to separate aqueous salts, organic molecules, and metal ions from solutions. Ultrafiltration and microfiltration utilize physical straining to remove colloidal and particulate contaminants, including microbial pathogens such as bacteria, and *Giardia* and *Cryptosporidium* cysts. Electrodialysis utilizes electrically charged membranes to remove dissolved ions by dialysis.

In addition to a description of membrane processes, this chapter covers the principles of membrane operation, including configurations and fouling mechanisms. It also describes methods for disposal of concentrates from membrane processes.

MEMBRANE PROCESSES FOR DRINKING WATER

Membrane separation technology has been used for many years in the small-volume treatment of pure and ultrapure water for many industries, such as beverages, pharmaceuticals, and electronics. Membranes are increasingly being considered and utilized for large-volume potable water treatment for the following reasons:

- Membranes remove particulates by physical straining above a target size, so they can reliably meet stringent drinking-water regulations for removal of microbial contaminants such as *Giardia, Cryptosporidium,* and viruses.
- Membranes have the ability to efficiently remove pathogens and may reduce the disinfection chemicals required and the potential for formation of disinfection byproducts.
- Increased water demands and limited high-quality raw-water supplies have forced the use of lower-quality raw-water supplies with elevated levels of mineral or biological contaminants. Membranes are able to treat these more difficult waters.
- Membranes have the ability to remove organic precursors, leading to more biologically stable water.

MEMBRANE APPLICATIONS

Membranes can be categorized into five types, with four types based on membrane pore size range, and electrodialysis reversal (EDR) being a fifth type. Reverse osmosis (RO) is the membrane process with the smallest pores that is capable of removing all organic compounds with a molecular weight above 100, most ions, all bacteria, viruses, microorganisms, and even radionuclides. Nanofiltration (NF) is a lower-pressure RO process that removes substances with molecular weights above 100–500 and is capable of removing hardness, pathogens, and organically derived color. Ultrafiltration (UF) is a low-pressure process intended primarily for removal of microorganisms, colloids, and high-molecular-weight compounds. UF can also remove some trihalomethane formation potential (THMFP) and disinfection products, and some ionic material, depending on the design characteristics of the membrane. Microfiltration (MF) is similar to UF except that the pore size is slightly larger and operating pressures are lower. Microfiltration can remove *Giardia* and *Cryptosporidium* cysts, but is less effective at removing viruses. Electrodialysis is a low-pressure electrochemical separation process whereby ionic contaminants are removed from solution by being induced to pass through the membrane into the concentrated reject water. Electrodialysis cannot remove organics or pathogens and is used almost exclusively for dimineralization of brackish water.

Figure 15–1 shows the range of particle sizes and corresponding membrane size. Use of membrane treatment for municipal water supply began in the 1970s when improvements to RO and EDR technologies resulted in a substantial increase in their use. Today there are over 2,500 operating membrane process plants in the United States (including electrodialysis reversal), with a total capacity of more than 600 million

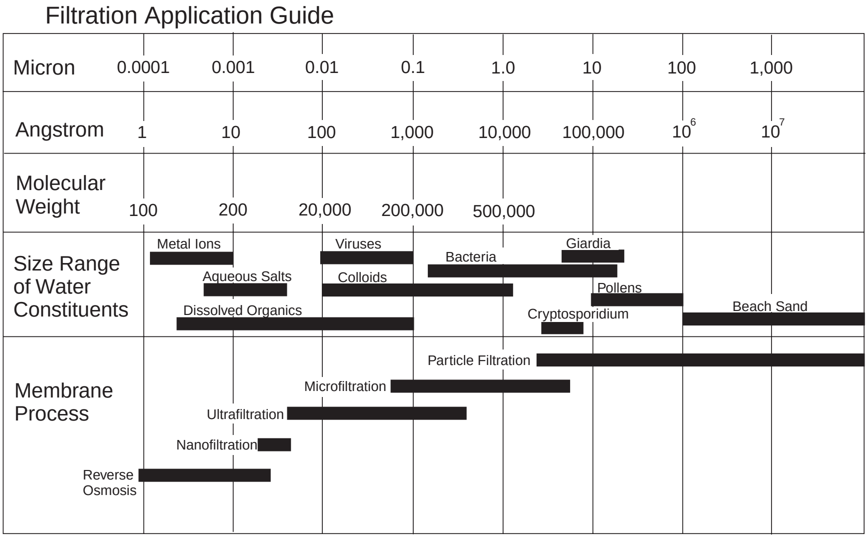


Fig. 15–1. The filtration spectrum (Courtesy of the Water Quality Improvement Center, Yuma, AZ)

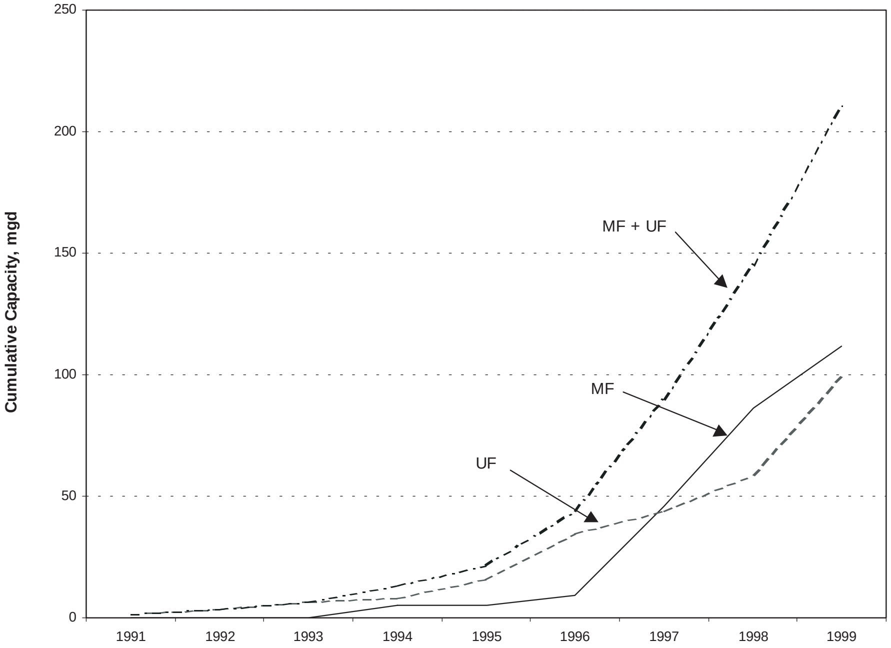


Fig. 15–2. Cumulative installed capacity of membranes in the United States (Reprinted from *Proceedings of 1999 AWWA Membrane Technology Conference,* by permission. Copyright © 1999, American Water Works Association.)

gallons per day (mgd). Figure 15–2[1] shows the increasing trend in MF and UF membrane usage.

PROCESS SELECTION / PERFORMANCE

Drinking water applications for membranes include the following:

- Pathogen removal (disinfection)
- Particle removal (turbidity reduction)
- Organics removal (TOC reduction)
- Softening
- Desalination
- Specific contaminant removal (e.g. arsenic, nitrate, etc.)

Giardia cysts, *Cryptosporidium* oocysts, bacteria, algae, and some viruses can be removed from water using microfiltration membranes, which operate with driving pressures between 5 and 45 psig (35 and 210 kPa). Without pretreatment, dissolved organic carbon (DOC) and dissolved inorganic materials will not be removed by microfiltration membranes. Ultrafiltration membranes are capable of removing the same colloidal materials as microfiltration, but are also able to remove all viruses as well as larger macromolecular material at driving pressures between 7 and 60 psig (50 and 420 kPa). Substantial improvements in performance can be realized when these membranes are used in conjunction with various chemical pre-treatments.

A summary of operating conditions of the major types of membranes is presented in Table 15–1.

PRINCIPLES OF MEMBRANE SEPARATION PROCESSES

Removal Mechanisms

Removal mechanisms used by membranes to purify drinking water include:

- Sieving via physical straining utilizing pressure as a driving force
- Solution diffusion and exclusion utilizing pressure as a driving force
- Ion exchange with electrical potential as the driving force.

Table 15–2 summarizes membrane removal mechanism, and membrane structure and driving force.

Solution/Diffusion and Exclusion When solutions of different concentrations are separated by a semipermeable membrane, the solutions attempt to reach equilibrium. The membrane allows only limited passage of solutes; thus, the equilibrium mechanism must be flow of solvent from the dilute solution to the concentrated solution. This flow produces a measurable pressure, called the *osmotic pressure,* toward the concentrated solution. If pressure equal to the osmotic pressure is maintained on the concentrated solution, the flow of solvent ceases. If the pressure on the concentrated solution is further increased, solvent flows across the membrane to the less concentrated solution; this process is known as *reverse osmosis*.

The equation that describes the flow of water across a semipermeable membrane is:

$$Fw = Kw(DP - Dp),$$

where:

Fw = water flux, m/d (cu meter/sq m/day) or gfd (gal/sq ft/day)
Kw = rate coefficient, m/d/atm or gfd/atm
DP = differential pressure applied across the membrane, atmospheres
Dp = differential osmotic pressure across the membrane, atmospheres

The osmotic pressure Dp is a function of temperature and the concentration of ions in solution; therefore, the applied pressure required to remove water from the concentrated solution increases as the total dissolved solids (TDS) content increases.

Salt or solute flow through the membrane is proportional to the difference in concentration across the membrane:

$$Fs = Ks(Cf - Cp),$$

TABLE 15–1. Various Membrane Performance Criteria and Applications

Membrane Process	Operating Pressures	Recovery	Flux	Primary Application
Microfiltration	5–40 psi (5–7 psi vacuum for immersed systems)	95–98%	1–20 m/d	Pretreatment for NF and RO; particle, cyst, and bacteria removal
Ultrafiltration	15–60 psi	80–95%	0.5–10 m/d	Pretreatment for NF and RO; macromolecule, cyst, bacteria, and virus removal
Nanofiltration	80–200 psi	70–90%	0.3–1.0 m/d	Softening, NOM Removal
Electrodialysis	60–100 psi (0.2–0.4 kWh per 1,000 gal for dialysis)	40–95%	N/A—ion exchange removal process	Brackish water desalination
Reverse osmosis		50–85%		Demineralization, TDS reduction
Brackish water	150–400 psi	50–85%	0.4–0.8 m/d	
Seawater—Standard	800–1,200 psi	30–50%	0.3–0.5 m/d	

TABLE 15–2. Summary of Membrane Operations in Water Treatment

Membrane Operation	Driving Force	Mechanism of Separation	Membrane Structure
Microfiltration	Pressure	Sieve	Macropores
Ultrafiltration	Pressure	Sieve	Mesopores
Nanofiltration	Pressure	Sieve + (solution/diffusion, + exclusion)	Micropores
Reverse osmosis	Pressure	Solution/diffusion + exclusion	Dense, or thin film composite
Electrodialysis	Electrical potential and pressure	Ion exchange, dialysis	Ion exchange

Source: Adapted from reference 2. (Reprinted from *Water Treatment Membrane Processes,* by permission. Copyright © 1996, American Water Works Association, AWWA Research Foundation, Lyonnaise des Eaux, and Water Research Foundation of South Africa.)

where:

Fs is the solute flux, kg/sq m per day or lb/sq ft/day
Ks is the rate coefficient, m/d or ft/d
Cf is the feedwater solute concentration, kg/cu m or lb/cu ft
p is the product solute concentration, kg/cu m or lb/cu ft

The *recovery rate* is equal to the percentage of feedwater that is produced as permeate; the solute *rejection rate* is the percentage of solute in the feedwater that is retained on the membrane. Theoretically, for a given feedwater and membrane, recovery will depend on pressure, temperature (higher temperatures increase the rate coefficient), and flow rate, and salt rejection will be a constant, depending only on differential concentration. In practice, recovery and rejection rates are affected by operating conditions of the membrane and by specific characteristics of the feedwater in addition to solute concentration.

Ion Exchange/Dialysis and Electrodialysis Reversal In the electrodialysis process, the membrane consists of a stack of ion-exchange resin material molded into semipermeable layers. When an electrical potential is applied across the membrane, charged ions are induced to travel through the membrane to the opposite attractive charge. Membranes are stacked with alternating cationic and anionic charged membranes. Water to be treated passes through the membrane channels; only the ions and a small stream of water pass through the membrane—the treated water exits through the manifolds. To extend time between cleaning and to improve operating efficiency, the polarity of the system is reversed periodically.

Membrane Characteristics

Cellulose acetate (CA) was one of the first membrane materials and is still widely used. The characteristics of a membrane can be altered by changing the polymer content; for example, increasing the acetate content of a CA membrane generally reduces both water and salt passage. Synthetic polymer membranes are cast with swelling agents to increase the water content of the membrane; the water flux and salt rejection are controlled by this water content. In asymmetric membranes (membranes made of one material, but with two distinct, heterogeneous layers), the porous layer is actually about two-thirds water. The ''active layer,'' or effective membrane, is a dense, nonporous film in which water and polymer are in equilibrium.

The mechanism by which solutes (salts) are rejected is still debated, but it is believed that organic molecules are screened, ions of a weight greater than the molecular weight cutoff (MWC) are adsorbed on the surface, and ions of a weight less than the MWC are adsorbed within the membrane, depending on the membrane material and the characteristics of the ion, such as dissociation and diffusivity. Cellulose acetate membranes reject nitrate nitrogen at about 50 percent, but most polyamide membranes are able to remove it completely, which is one reason why polyamide membranes are a good choice for wastewater treatment. Most low-molecular-weight and volatile organics pass through both cellulosic and polyamide membranes.

Systems are normally designed on the basis of solute rejection rather than water flux. The total membrane area required for a plant is based on design flux rate coefficients, but this calculated area must always be increased to compensate for the gradual decline in water flux that occurs in service.

Membrane Manufacture

Most commercial membranes are produced as flat sheets, fine hollow fibers, or in tubular form. The flat sheets are of two types: asymmetric and composite. Asymmetric membranes are cast in one process and consist of a very thin (less than 1 micron) layer and a thicker (up to100 micron) porous layer that adds support and is capable of high water flux.

Composite membranes are made by bonding a thin cellulose acetate or polyamide active layer (0.15–0.25 micron) to a thicker porous substrate, which provides stability. Most of these thin-film composites (TFCs) are manufactured by techniques in which a thin barrier layer of polymeric material is deposited on the substrate with a polymer solution or a ''sol-gel'' coating process. In the sol-gel method, a concentrated solution of polymer in solvent is cast on the support sheet, then immersed in water to precipitate the polymer. The membrane film can be transformed after deposition to modify selectivity and water flux.[3] Polysulfone (PS) is usually used as the porous substrate because it is more resistant to chemical and microbial attack than cellulosic blends or polyamide.

Fine hollow fibers were developed by DuPont using polyamide resins; the fibers are approximately the thickness of a human hair, with an overall diameter of about 106 micron.[4] Dow also developed fibers about twice as thick, and more dense, using cellulose triacetate (CTA).[5] Hollow fibers are made by melt spinning, a process in which the melted polymer is extruded through an orifice and nitrogen is injected into the core. The fiber is annealed to set the rejection characteristics. Hollow fibers have a high density and low permeability, but their flux area-to-volume ratio is the highest of all membranes.

Tubular membranes are usually asymmetric cellulose acetate cast on a porous support tube. They are expensive and are usually reserved for applications where fouling is severe.

Membrane Configurations

The process application and feedwater quality determine the type of membrane and configuration to be used. Ultrapure water and potable water supplies are usually produced in spiral-wound or fine hollow-fiber permeators. These units can be used when the raw-water supply is of high quality or where pretreatment is used to remove high-molecular-weight organics and suspended solids. Tubular permeators are typically used in UF and MF applications for waters having high suspended solids or organics, because they are easiest to clean. Flow can be either from outside in or inside out. Hollow-fiber units have also been used for raw water, but they are much more susceptible to plugging.

Spiral-Wound Permeators These units are named for the rolled-up arrangement of membranes and support sheets that are used in a tubular pressure vessel. The most common arrangement is a ‘‘sandwich’’ of two thin-film composite membranes and a single porous backing sheet (called a product carrier). The composite membranes, which extend slightly beyond the substrate on three sides, are pinched together and sealed with epoxy. The fourth edge is attached to a perforated tube that receives the permeate flow. A fabric or polyester mesh spacer is placed on each side of the completed ‘‘leaf’’ to provide a flow channel, and the entire sheet and spacers are rolled up around the flow tube, such that the unit resembles a roll of paper towels (see Fig. 15–3). A variation of this arrangement is to use a leaf of two asymmetric membranes bonded to a single product carrier sheet.

Usually, multiple leaves are attached radially to the flow tube. The individual leaf and flow tube units are usually connected in series as they are installed in the tubular

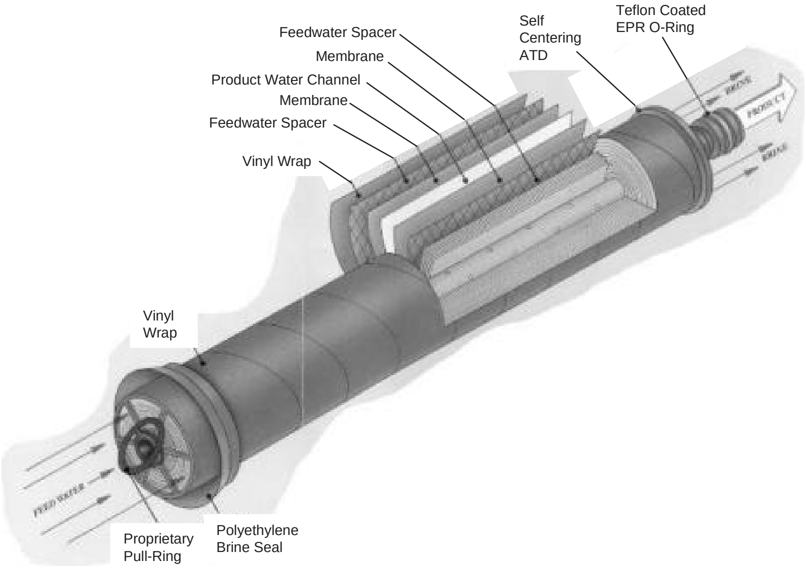


Fig. 15–3. Spiral-wound RO membrane (Courtesy of Osmonics Corporation)

pressure vessel. A common arrangement is a 21-foot-long vessel with six linked sets of leaves. Feedwater enters the shell of the vessel and passes between the leaves in an axial direction. The permeate passes through the membrane and into the backing sheet, and moves in a spiral-radial direction toward the product water tube. The concentrate is removed from the shell at the end opposite the feed inlet. Spiral-wrap units can be operated at high pressure and have high flux rates. Plugging is a problem inherent in the design, and the units must be cleaned chemically on a regular schedule.

Fine Hollow-Fiber Permeators Hollow-fiber tube sheets are constructed by wrapping and gluing hollow fibers around a long, flat, porous sheet of backing material (the web) in a continuous helix. The sheet is cut to the proper length and rolled tightly around a perforated feed tube, with the fibers running axially. The fibers are then cut at one end and glued with epoxy into a doughnut-shaped manifold, which slips over the feed tube. The entire assembly is inserted into a pressure tube, such as a small-diameter pipe. In operation, feedwater leaves radially from the central tube and travels through the fiber bundle to the wall of the shell. Water permeates through the hollow fibers and travels through the inside of each fiber to the manifold, where it is collected. The concentrate is collected from the shell. UF and MF hollow-fiber systems are designed to either have the feedwater travel inside the fibers and the permeate is collected outside the fiber in the shell (cross flow) or have the feedwater flow from the outside to the inside of the fiber (transverse flow). A typical hollow fine fiber permeator (RO application) is shown in Figure 15–4.

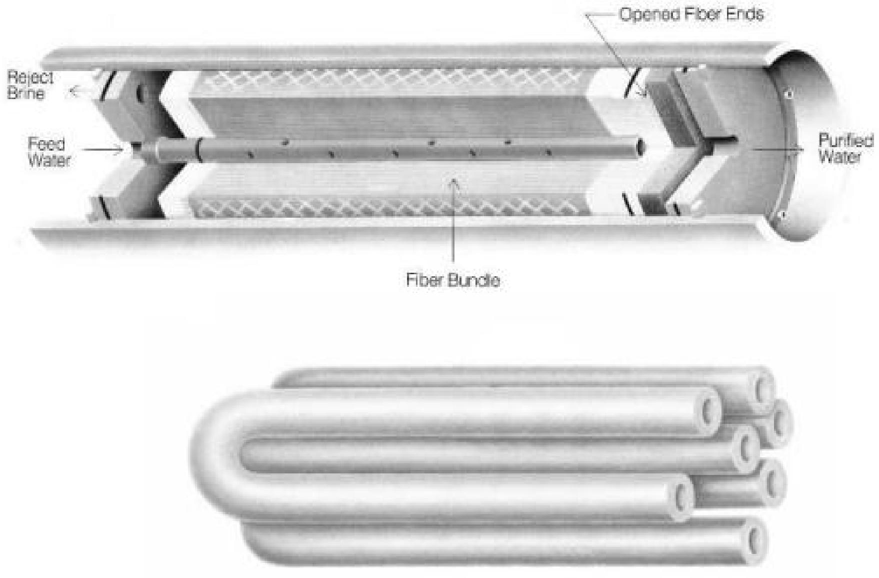


Fig. 15–4. Hollow-fiber permeator (Courtesy of Dupont Corporation)

Tubular Permeators Tubular permeators use a membrane cast on the inside of a porous support tube, which is then inserted (either singly or in a bundle) into a pressure vessel. Figure 15–5 is a photograph of tubular membranes. The feedwater is pumped through the feed tube, the product water is collected from the shell, and the concentrate continues through the feed tube. These units are generally used for water with high suspended solids concentrations or plugging potential. Tubular units are the easiest to clean, which is accomplished by circulating chemicals and pumping a ''foamball'' or ''spongeball'' through to mechanically wipe the membrane. Tubular units produce at a low product rate relative to their volume and are not used in municipal drinking water systems.

Membrane Arrays

For membrane applications with an unlimited feed source, such as seawater desalination, the recovery rate may not be as important as product quality. By operating at a lower recovery, brine concentrate TDS can be kept lower and less salt appears in the product water. In applications other than seawater desalination, the recovery rate is important and membranes are grouped in stages whereby the concentrate from one stage becomes the feed for the next stage. The overall arrangement is known as an *array*. The required membrane array is a function of the feedwater quality, desired recovery, and membrane characteristics. The recovery of each stage in a three-stage system is typically 50 percent–50 percent–40 percent, for an overall feedwater recovery of 85 percent. Multiple trains are usually required so that they can be isolated for cleaning, membrane replacement, and repairs.

Pretreatment and Operating Requirements

Pretreatment may be required to:

- Remove solids in the raw water.
- Disinfect to prevent growths in the membrane system.
- Chemically condition the feedwater to prevent damage to the membranes.
- Chemically condition the feedwater to reduce membrane fouling.

Hydrolysis Hydrolysis is accelerated at high pH and is controlled by adding sulfuric acid to the feedwater prior to cartridge filtration. The optimum pH range for several membrane materials is shown in Table 15–3.

Temperature The useful life of cellulose acetate membranes decreases above 40°C, which is a serious limitation for some industrial waters. Polyamide membranes can operate effectively up to 45°C.

Membrane Performance Decline

The predominant cause of membrane performance decline is fouling and plugging. Plugged membranes decrease water flux and usually increase the passage of less mobile ions. The membrane rejection process is not purely a surface adsorption phenom-

Fig. 15–5. Tubular membranes (Courtesy of Koch Membrane Systems)

TABLE 15–3. Operating pH for Various Membrane Materials

Membrane	Best pH Range
Cellulose acetate (CA)	3–8
Cellulose diacetate (CDA)	2–9
Cellulose triacetate (CTA)	2–11
Polyamide	2–11

enon; some ions, molecular compounds, colloidal particles, precipitates, and bacteria are adsorbed within the membrane itself.

Another cause of long-term performance decline is membrane hydrolysis. Cellulose acetate is particularly vulnerable; acetyl groups are replaced by hydroxyl groups, leaving a less dense, etched structure that allows greater salt passage. Hydrolysis is accelerated at very low or very high pH; it is minimized for cellulose acetate at about pH 5. The newer cellulose acetate blend membrane's allowable operating pH range is 4 to 6.

Membranes are also subject to bacterial attack, especially those made with cellulosic polymers. Certain species of bacteria metabolize the acetyl groups, leaving the structure open to cellulutic microorganisms that are able to degrade the remaining material.[6] The vulnerability of cellulose acetate to bacteria was a major impetus for the development of more resistant membranes such as polyamide and polysulfone.

Polyamide membranes are becoming popular because they are resistant to microbial attack and performance is well documented. In addition to compaction, hydrolysis, and bacterial attack, both cellulose acetate and polyamide membranes are degraded by high temperatures, strong oxidants, chlorine, and certain organic solvents, oils, and grease.

Plugging and Fouling

Plugging and fouling result from colloids, high-molecular-weight organics, humics, silica, silt, metal oxides and precipitates, bacterial slimes, and oils and greases.[7] Hollow-fiber units are more susceptible to plugging than spiral-wound units,[8] but almost all feedwaters require some type of pretreatment to prevent premature fouling. One measure of plugging potential is the silt density index (SDI), which is determined from a special 0.45-micron filter test. For most water supply plants, 5-micron cartridge filtration is adequate. UF or MF are sometimes used as a pretreatment for RO or NF to reduce plugging, or in combination with ion exchange to provide a water with lower TDS, fewer organic materials, less trihalomethane potential, and lower chlorine demand.

Scale and Precipitation Many natural waters used as RO feedwater are close to saturation in carbonates and sulfates. As the TDS in the concentrate increases, precipitation occurs on the membrane surface. Precipitation of calcium, barium, and strontium sulfates and magnesium hydroxide is a common problem in Florida. Sulfate precipitation is usually controlled with sodium hexametaphosphate (SHMP). Carbonate precipitation is usually prevented by lowering the pH with sulfuric acid, which is also desirable with respect to membrane hydrolysis. If the pH is depressed sufficiently, carbon dioxide is formed and must be removed from the permeate by degasification. Plants that use acid to control feedwater pH often have to add sodium hydroxide to the product water; degasification serves to raise the pH and reduce the amount of sodium hydroxide required.

Bacterial Fouling and Related Problems

Bacterial slimes are a major membrane foulant, especially for those units that are taken off line for an extended period. Bacteria oxidize ferrous ion to ferric ion, which forms precipitates that cause plugging. Sulfate reducing bacteria can cause the production of hydrogen sulfide gas, which must be removed from the permeate. Bacteria can also metabolize the acetyl groups in cellulose acetate, allowing other microorganisms access to the membrane. Almost all cellulosic membrane plants require combined chlorine pretreatment. Bacterial fouling usually requires shutdown and cleaning; common cleaning compounds/biocides include citric acid, enzyme-active laundry detergent, and formaldehyde. Free chlorine is an effective biocide but the maximum allowable concentration for CA membranes is 1.0 mg/L and for polyamide the allowable concentration is zero (0.0).

Fouling by Organics

Organics decrease membrane performance due to adsorption and blocking of pore spaces, and the effects can be irreversible. Organics are usually the main cause of plugging or fouling for feedwaters that are high in chemical oxygen demand (COD) or total organic carbon (TOC), and the first units in a permeator train generally show the greatest adsorption. Organics can also promote biofouling by providing conditions that enhance growth of microorganisms.

Membrane Life and Cleaning

The economic life of a membrane depends on the type of service and operating pressure. Currently, wastewater membranes have a life of about 2 to 5 years, whereas membranes used with good quality feedwater for potable water supply have a life of 3 to 10 years or more. Typical contracts for RO membranes to desalinate seawater guarantee an economic life of five years. Generally, membranes are replaced when the water flux declines below some economic value, or the ratio of TDS in the product to TDS in the feedwater exceeds some acceptable value, and cleaning no longer restores water flux or salt rejection.

MICROFILTRATION AND ULTRAFILTRATION

General

Both MF and UF processes are pressure-driven membrane processes that operate at low (5 to 60 psig) feed pressures and flux rates of 15 to 75 gfd (0.6 to 3.1 m/d). These membranes remove contaminants purely by physical straining (sieving). Chemical conditioning of the raw-water feed is not required except in the case where enhanced organics removal, or iron and manganese removal, is desired. The UF membranes pore size (0.01 μ) is about one order of magnitude less than the MF pore size (0.1–0.2 μ). The smaller UF pore is able to remove macromolecules such as humic acids, colorants with a molecular weight cutoff greater than 10,000 to 100,000 daltons, and viruses. UF and MF membrane systems are normally completely automated to control feed rate and pressure, and backwashing.

Applications

Microfiltration and ultrafiltration primarily are used to treat slight to moderately turbid surface waters to produce potable water. These membrane processes can replace conventional water treatment technology consisting of coagulation, flocculation, sedimentation, and filtration. The ability of these membranes to provide a greater than a 7-log removal of *Giardia* cysts, *Cryptosporidium* oocysts, without chlorine addition, has led to increased interest in their use. A significant benefit is their ability to produce a consistently high-quality effluent without the use of chemicals. Common applications for UF and MF include:

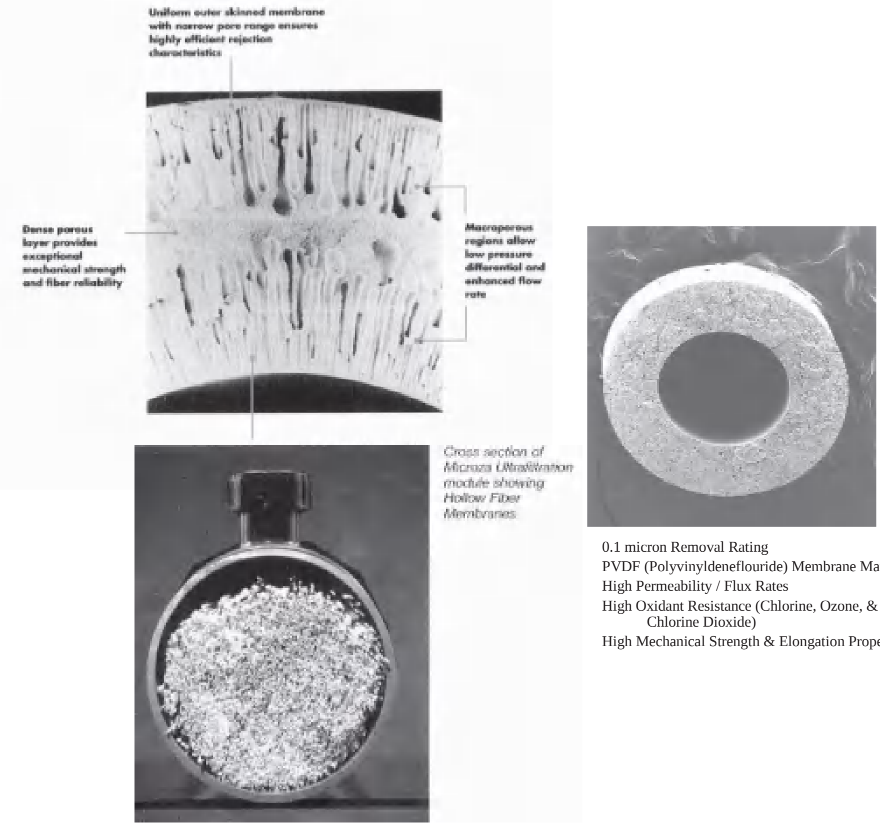
- Conventional treatment replacement (turbidity, suspended solids, bacteria and cyst removal)
- Pretreatment for RO and NF processes
- Waste backwash water recovery
- Iron and manganese removal (with prior oxidation)
- Color removal (MF with coagulant addition)
- DBP precursor removal (with or without PAC addition)

Membrane Characteristics The majority of MF and UF potable water applications are either hollow-fiber or tubular configurations. Hollow-fiber membranes are typically have an outside diameter in the range of 0.5 to 2.0 mm with a wall thickness of 0.2 to 0.4 mm. These are also known as capillary tubes. Thousands of fibers are bundled into a pressure vessel with one end potted with an epoxy resin. Hollow-fiber systems are generally lower in cost because of the greater surface-area-to-volume ratio compared to tubular systems. Hollow fibers are constructed of polymers and plastics, whereas tubular membranes are typically ceramic. A photomicrograph of a hollow-fiber membrane designed for bidirectional operation is shown in Figure 15–6. Typical commercially available MF and UF hollow-fiber membranes are shown in Table 15–4.

Membrane Process Configurations MF and UF hollow-fiber membrane systems can operate in either inside-out, outside-in (transverse flow), or bidirectional configuration. For either configuration, feedwater can be fed in either a cross-flow configuration with recirculation, or in a dead-end (direct feed) configuration as shown in Figure 15–7.

Cross-flow operation generally must maintain at least a 2.5 m/s flow past the membrane surface, and hence requires additional piping and power for recirculation. Cross flow is easier to control with an inside-out design, because the fibers provide well-defined flow paths. With transverse flow, cross-flow recirculation is normally not practiced, because maintaining consistent velocity around the fibers is impeded by the high number of fibers bunched into the pressure vessel. Some manufacturers have dealt with this problem by using air in the backwash sequence. With dead-end transverse-flow filtration using air backwash, 100 percent recovery is possible with only a portion of the raw water lost for bleeding and backwashing. Transverse-flow or cross-flow systems using permeate for backwashing can still achieve a recovery as high 95 percent. Although transverse flow design offers greater cleaning challenges, it offers the significant advantage of lower pressure drop for a given flow and number of fibers. The lower pressure drop is due to the greater surface area on the outside of the fiber than on the inside.

Conventional Treatment Replacement UF and MF both may be considered in lieu of conventional treatment. Because of its smaller pore size, UF will remove high-molecular-weight molecules such as humic acids and colorants, colloidal material, and



0.1 micron Removal Rating
PVDF (Polyvinyldeneflouride) Membrane Material
High Permeability / Flux Rates
High Oxidant Resistance (Chlorine, Ozone, & Chlorine Dioxide)
High Mechanical Strength & Elongation Properties

Fig. 15–6. Pictograph of hollow-fiber UF membrane (Courtesy of Pall Advanced Separation Systems)

viruses. The advantage of virus removal is diminished by the small CT requirement for virus reduction using chlorine. Other factors such as costs, ability to remove DBP precursors, ability to remove color, operation, maintenance, and capital costs are normally more important in making a decision on which membrane system to use.

Full-scale operation of UF membrane plants has demonstrated that the permeate quality is virtually independent of the raw-water quality. The permeate turbidity is less than 0.1 NTU, with a typical value of less than 0.03 NTU. Permeate particle counts in the size range of 0.5 to 5 μ are normally less than 5 per mL. Complete removal of bacteria and cysts is achieved with UF membranes. Virus removal is dependent on the molecular weight cutoff of the membrane. A photograph of a UF treatment system is shown in Figure 15–8.

For MF systems with a pore size of 0.2 μ or less, permeate turbidity is typically below 0.1 NTU. A photograph of an MF treatment system is shown in Figure 15–9.

UF has not been widely used for municipal water treatment in the United States to date; however, there are many existing and planned applications of MF for conventional water treatment. Microfiltration systems to replace conventional treatment of groundwater and surface water are available from several manufacturers, including:

TABLE 15–4. Dimensions of Hollow Fiber Ultrafiltration and Microfiltration Membranes

	Aquasource LED 35	KOCH PM-100	Memcor M10	PALL Microzoa	ZENON ZW-150
Type	UF	UF	MF	MF	MF
Material	CA	PS	PP	PVDF	—
Dimensions					
Pore size*	100,000 D	100,000 D	0.2 microns	0.1 microns	0.2 microns
I.D. (mm)	0.94	0.76	0.31	0.7	0.9
O.D. (mm)	1.3	1.32	0.65	1.3	1.91
Length (m)†	1.3	1.02	0.97	2.0	1.5
Fibers/module	14420	3228	11000	—	1533
Active surface‡	Lumen	Lumen	Shell	Shell	Shell
Area/module					
(m^2)	55.4	7.8	21.7	50	13.8
(ft^2)	596	84.3	233	538	148.5

(Courtesy of Erie Public Works Department)

CA, cellulose acetate; PS, polysulfone; PP, polypropylene; PA, polyacrylonitrile.

*UF rejection properties as a molecular weight cutoff (MWCO) expressed in daltons.

†Length of fiber exposed to the feed.

‡Lumen denotes inside of fiber. Shell denotes outside of fiber.

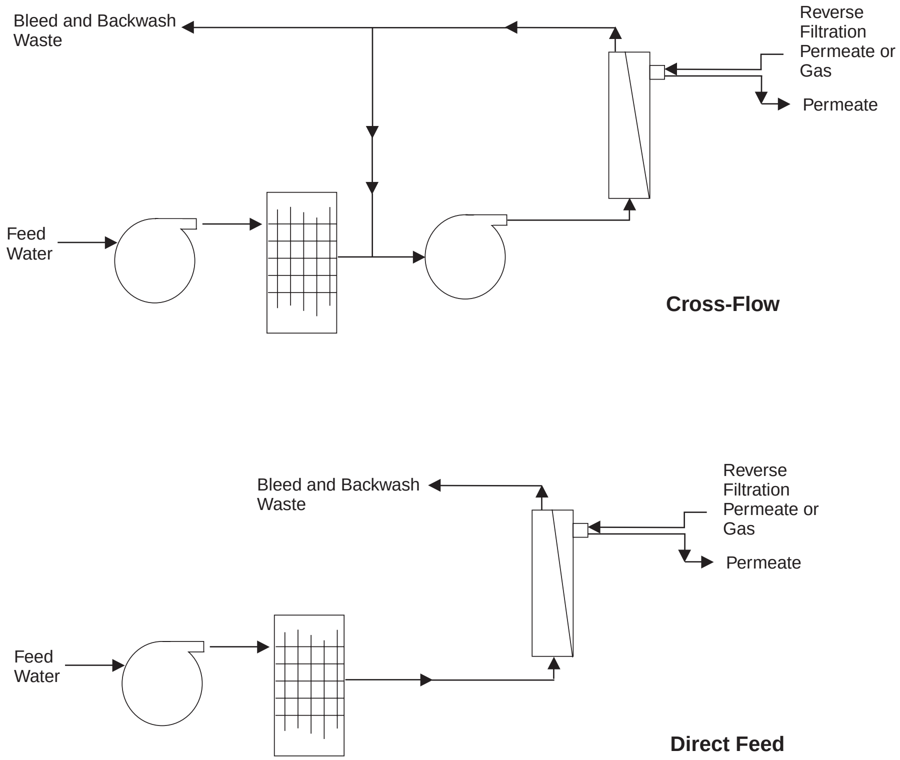


Fig. 15–7. Typical MF and UF process flow schematics

- Memcor—USFilter
- Pall
- Zenon
- Koch

The systems are capable of producing low-turbidity water with little or no pretreatment. The state of California has approved MF as an alternative filtration technology that satisfies the requirements of the Surface Water Treatment Rule.

Memcor—USFilter. Memcor plants are in service producing drinking water at several locations in the United States. Memcor is a continuous microfiltration system that is capable of removing virtually all particles greater than 0.2 μ in size. A low-pressure feed pump (30–35 psi) supplies feedwater to the membrane modules. Contaminants in the feedwater build up on the outside of the hollow-fiber membrane surface and clean filtrate flows down the inside. A driving force called the transmembrane pressure difference (TMP) sets up across the membrane. The TMP increases as the contaminant load increases, and at a set point a gas backwash is automatically initiated. Backwashing is with compressed air from inside the hollow-fiber membranes. This pulse of air dislodges contaminants from the outside membrane surface and the contaminants

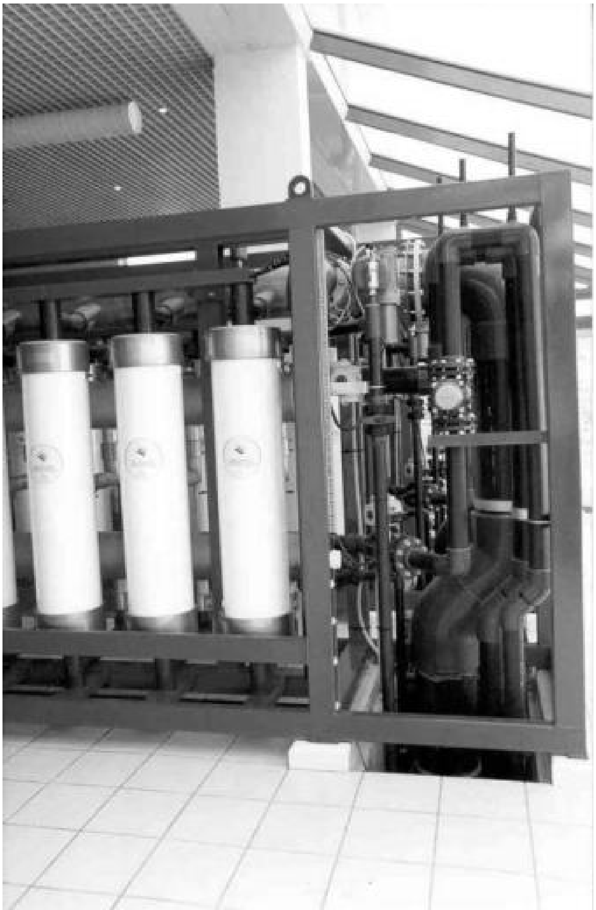


Fig. 15–8. UF treatment system (Courtesy of AquaSource, North America)

are carried out of the system with feedwater to a waste backwash water receiving basin. The frequency of backwash varies with the feedwater quality from about 20 to 30 minutes. The duration of each backwash is about 2.5 minutes.

The standard size module for full scale plants is the 90M10C with a total membrane surface area of 14,526 sq ft (1,340 sq m). The modules are skid mounted and the system is equipped with a compressed air system, backwash water receiving tank, and clean in place (CIP) system. A typical layout for a three-module system is shown in Figure 15–10. Typically, the membranes require chemical cleaning at three-week to six-week intervals. Cleaning chemicals vary somewhat with the feedwater characteristics and include: citric acid, caustic soda, and detergent formulations.

Pall. There are several existing and planned municipal water treatment plants utilizing the Pall MF system in the United States. Pall UF treatment systems are more widely used in Japan for both industrial and municipal water treatment.

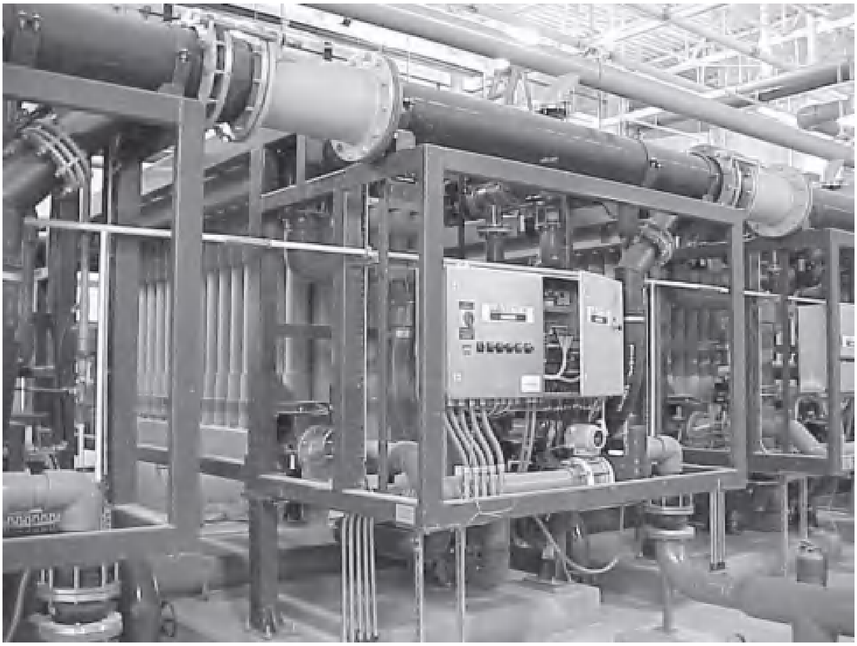


Fig. 15–9. 4 MGD microfiltration plant, Erie, CO (Courtesy of Erie Public Works Department)

The Pall MF system utilizes a hollow-fiber membrane with outside-to-inside flow during the service cycle. Backwashing is from inside-out with water and air. Figure 15–11 is a flow diagram of a Pall system, and Figure 15–9 is a photograph of Pall membrane modules. Membranes in a Pall system for small plants are generally arranged in two parallel trains. The number of membranes in each train is adjusted to provide the desired plant capacity. The systems are supplied for automatic operation, including the service, backwashing, and cleaning cycles.

Immersed Membranes (*Zenon and Memcor CMF-S*). Membranes operate in the outside-to-inside flow mode. The membranes operate under a vacuum created within the hollow membrane fibers by a permeate pump. Raw water is drawn through the membranes and is pumped out to treated water storage or distribution. Air scour is continuously introduced at the bottom of the membrane modules to create turbulence, which helps keep the membranes clean and maintain flux.

Zenon supplies a proprietary MF system called ZeeWeed. Memcor has also developed a similar system, called CMF-S, in which the membranes are suspended vertically in a basin containing the raw water. The immersed membrane systems are shown in Figure 15–12.

The proprietary membrane material is resistant to chlorine up to concentrations of 1,000 mg/L. Membrane cleaning by backpulsing is achieved by reversing the permeate flow and backwashing the hollow fibers with permeate at low pressure.

Pretreatment Requirements The objective of pretreatment for UF and MF is to remove large solids to prevent plugging of the hollow fibers. Typically, a self-cleaning

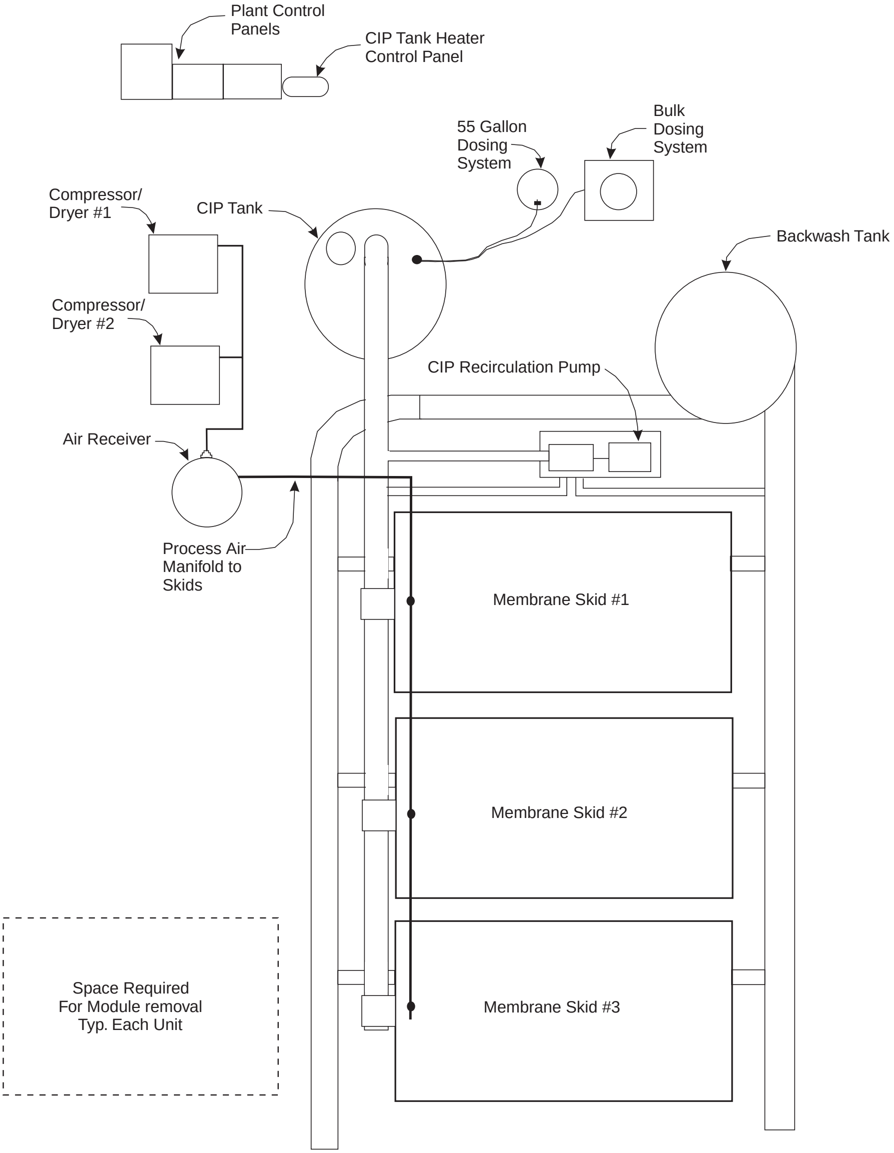
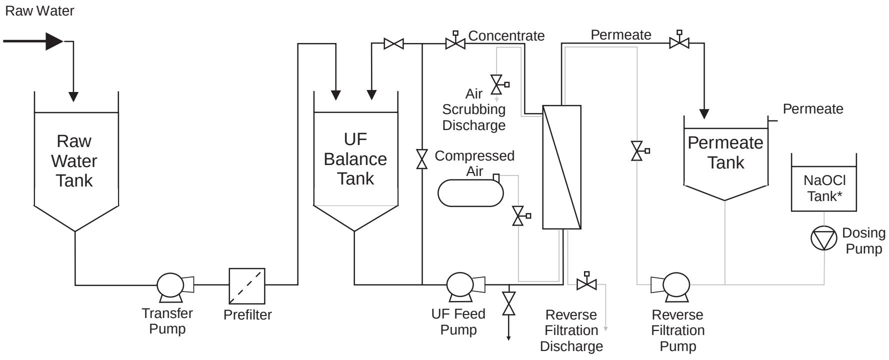


Fig. 15–10. Typical layout of microfiltration plant



* Optional

Fig. 15–11. Backwash system using compressed air for scrubbing (Reprinted from *Water Treatment Membrane Processes,* by permission. Copyright © 1998, American Water Works Association, AWWA Research Foundation, Lyonnaise des Eaux, and Water Research Commission of South Africa.)

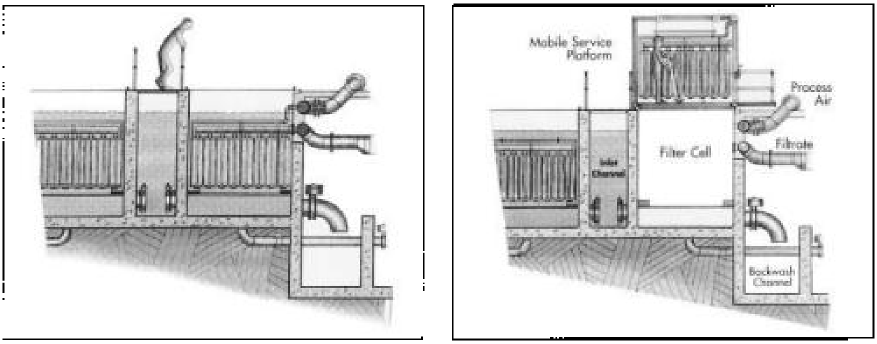
screen capable of removing particles above 100 to 200 μ is all that is required for MF pretreatment. More extensive solids removal pretreatment should be considered if the raw-water suspended solids concentration is above 200 mg/L.

Adjustment of pH is normally not required, because virtually every membrane now manufactured has a wide range of tolerance. If scaling is a concern, steps to balance the alkalinity of the water should be taken. If iron and manganese are present, oxidation should be performed to force precipitation to occur prior to entering the membrane.

Care must be taken to determine the compatibility of the membrane material to oxidants and cleaning agents. The Memcor polypropylene hollow-fiber membranes have a low tolerance for combined chlorine and cannot tolerate any free chlorine. Membranes constructed of polysulphone, PVDF, and PAN tend to be more tolerant to chlorine.

Temperature Effects Feedwater temperature affects the flow through MF and UF membranes. The filtrate flow rate at any given pressure changes with temperature and is related to the change in water viscosity with temperature. The following correction factors can be used as a guide to approximate flow rate changes. Membrane manufacturers should be consulted to determine expected performance at a specific location.

Temperature °C	Correction Factor
1	0.58
5	0.66
10	0.77
15	0.88
20	1.00
25	1.13
30	1.26



The CMF-S is designed for easy service inspection.

The modular filter rack is easily removed from the filter cell for convenient maintenance.

(a)

(b)

Fig. 15–12. Schematic of an immersed membrane system (Courtesy of (a) USFilter/Memcor and (b) ZENON Environmental, Inc.)

These correction factors indicate that for an MF system rated at 1.50 mgd at 20°C:

- The rating at 10°C would be $1.50 \times 0.77 = 1.16$ mgd
- The rating at 25°C would be $1.50 \times 1.13 = 1.70$ mgd

It should be noted that these correction factors do not account for other water characteristics that may also change with temperature. For example, the nature and concentration of solids in the feedwater may change with temperature. Also, biological activity changes with temperature, which can affect membrane fouling. These and other temperature-related changes can affect flow rates that are not related to viscosity.

Membrane Monitoring Unlike conventional treatment plants that provide multiple barriers, membrane systems are a single-barrier treatment technology. Thus, monitoring of the membrane systems is critical to ensure that the membrane system is producing permeate free of biological contaminants. The most common locations of loss of integrity include broken hollow fibers and leaking seals. Indirect techniques for monitoring membrane performance in increasing order of sensitivity include:[9]

- On-line turbidity monitoring (continuous)
- On-line particle monitoring (continuous)
- On-line particle counting (continuous)
- Silt density index monitoring (grab samples)
- Biological monitoring using virus seeding (off-line test)

Direct membrane monitoring techniques include:

- Air pressure holding test
- Bubble point testing
- Sonic testing

A combination of indirect and direct measurement techniques should be employed to ensure membrane system integrity.

Case Histories

San Jose Water Company, California In February 1994, the San Jose Water Company began operating a 5 mgd MF plant.[10] The MF system replaced an obsolete diatomaceous earth filtration system that was determined to be unable to meet the requirements of California's Surface Water Treatment Rule. The MF plant provides an alternative seasonal source of drinking water for about 30,000 people in the Santa Clara Valley of Northern California. System characteristics are as follows:

- System operates between January and June when Saratoga Creek has enough water to supply the plant with at least 1.0 mgd of feedwater.
- Typical creek source water:

Turbidity	<1 to 250 NTU
TOC	1 to 2 mg/L
Temperature	5 to 15°C

- Pretreatment with 380 μ strainers
- Six Memcor 90M10C membrane modules
- All components in 1,600 sq ft building
- Feed pressure, 24 psi
- Operating transmembrane pressure, 2 to 20 psi
- Credit for 3-log *Giardia* removal
- Postdisinfection to achieve 3.5-log virus inactivation
- No coagulation chemicals used
- Chemical cleaning interval is 2 to 6 weeks, depending on raw water quality

Erie, Colorado The Town of Erie is north of Denver in the foothills of the Rocky Mountains. To meet the domestic water service demand, the town investigated replacement options of the existing conventional treatment plant. In the spring of 1998, the town board decided to pilot-test both microfiltration (MF) and ultrafiltration (UF) low-pressure membranes on the town's raw-water sources. Pilot scale units, shown in Figure 15–13, from two manufacturers were delivered to the existing plant site and installed for a side-by-side comparison of the treatment capabilities. Pilot testing began in May and continued through August 1998.

Two raw-water sources are utilized by the town in treatment for domestic supply: Erie Lake and the Northern Colorado Pipeline (NCP). Table 15–5 highlights raw-water data and reflects the varying quality differences of the two sources. Erie Lake is a shallow stagnant reservoir that has problems with excessive algae growth in the hot summer months. The algae produce taste and odors that are difficult to remove with conventional treatment. Additionally, elevated total organic carbon (TOC) concentrations are created by the natural organic matter present. By contrast, the NCP transports

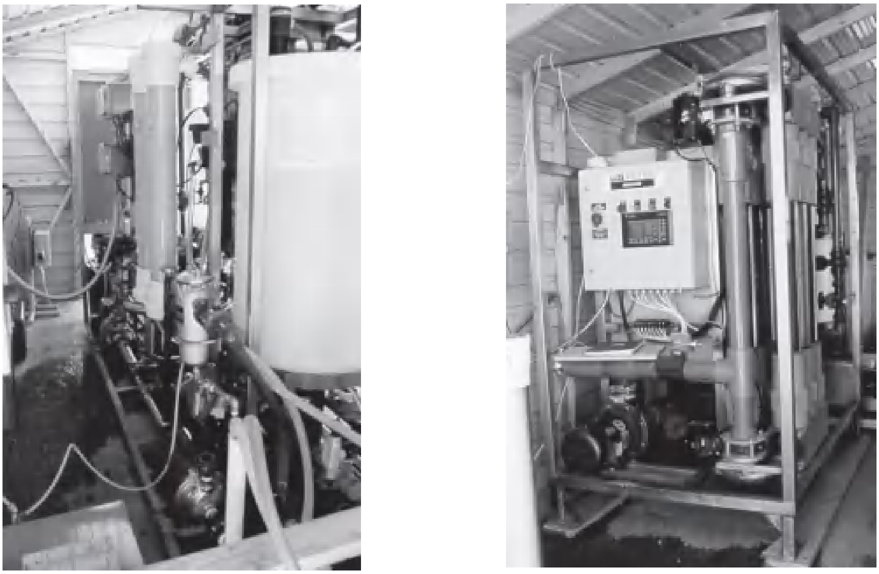


Fig. 15–13. Ultrafiltration and microfiltration units (Courtesy of Erie Public Works Department)

TABLE 15–5. Raw Water Data for the Erie Water Treatment Plant

Constituent	Units	Carter Lake	Erie Lake
Turbidity	Average/Maximum (NTU)	3/8	5/200
Total organic carbon	Average/Maximum (mg/L)	3/5	4/6.5
THMFP	μg/L		137
Total suspended solids	Average (mg/L)	<6	55
Temperature	Average/Minimum (°F)	50°/42°	60°/33°
pH	Range	6.8–9.2	7.0 –9.1
Alkalinity (mg/L as $CaCO_3$)	Range	30–40	95–110
Hardness (mg/L as $CaCO_3$)	Range	20–24	100–130

(Courtesy of Erie Public Works Department)

water from the deep, high mountain reservoir Carter Lake and supplies a consistently high-quality water.

The MF membrane tested was a polypropylene material nominally rated at a 0.2-micron pore size. The system was operated in a dead feed flow configuration, and utilized an air/raw water backwashing system. The UF membrane tested was a polysulfone material nominally rated at 0.05-micron pore size. The UF system was operated in a cross-flow configuration and utilized a sodium hypochlorite backwash using finished water. Both units required periodic chemical cleaning with a caustic cleaning solution and alternating citric acid cleanings.

The goals of the pilot study were: to identify the membrane life-cycle costs; to develop reliable operation and maintenance cost projections; to compare the operational and maintenance requirements of each manufacturer's system; and to evaluate the treatment options for the removal of TOC by use of chemical coagulation (aluminum chlorohydrate (ACH)), granular activated carbon (GAC) contactors, and the adsorbent heated iron oxide particles (HIOPs).

Pilot Test Results Results of the pilot testing for both the UF and MF membrane operation are summarized in Table 15–6. As expected, the MF membrane system has a much higher flux and production rate than a UF system.

TOC reduction through the membranes was evaluated in the first 60 days of the pilot study. Several data points were taken from each unit using the four raw-water scenarios. Figure 15–14 shows the quantity of TOC in the raw water as compared to that of MF- and UF-treated water. The Erie Lake source has a low ratio of UV-254 to DOC, identified as the source's SUVA value. A low SUVA value (between 2 and 3) indicates that the TOC is largely dissolved organics and therefore would pass through the membrane without being removed. The Carter Lake source has a slightly higher

TABLE 15–6. Operational Results from Erie Lake Pilot Study

	MF	UF
Operational flux rate	74 gfd at 50 gpm	16 gfd at 4 gpm
Feed water recovery	90% at 45 gpm	86% at 8 gpm
Backwash frequency	30 min	30 min

(Courtesy of Erie Public Works Department)

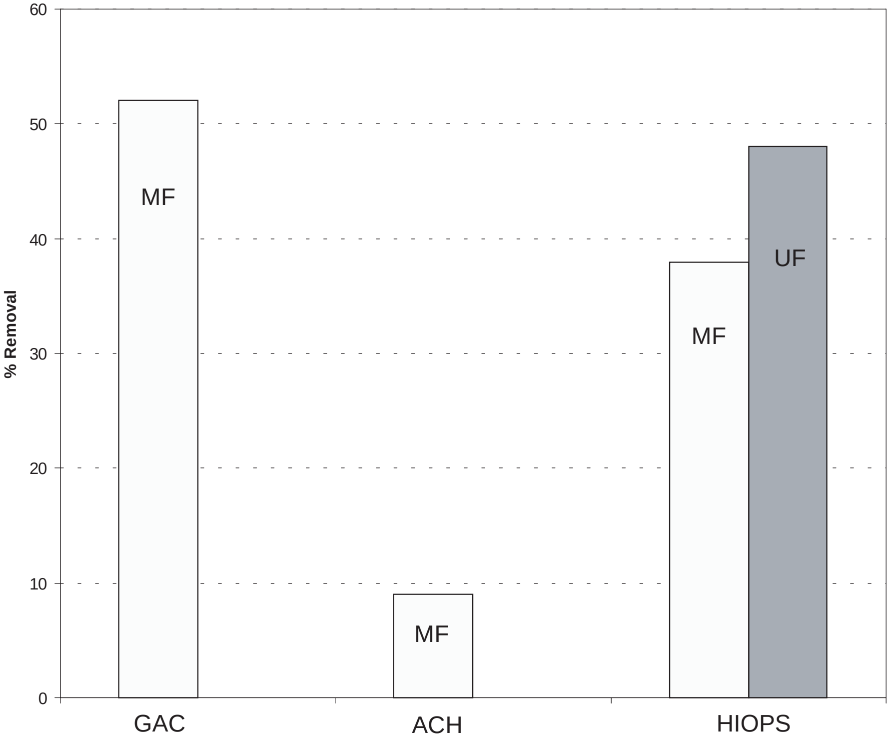


Fig. 15–14. TOC removal supplemented membrane

SUVA value (between 4 and 5), and TOC was removed at a higher percentage when this source was treated either alone or in a blend. The TOC removal in the Lake Erie water was only 7 to 10 percent for both the UF or the MF membrane. Because of the membranes' limited ability to remove TOC, the THMFP removal through either the MF or the UF was negligible.

TOC removal was greatly enhanced when absorbents were used in the treatment train. The first testing was to apply an absorbent known as heated iron oxide particles (HIOPs) directly to the feedwater. The pH of the water is adjusted using a mild acid, and HIOPs are added to the raw-water feed. The particles form a slight cake-layer on the membrane surface, where they absorb organics and protect the membrane from fouling. The results at Erie were increased backwash intervals and increased organic removal for both the MF and UF units. TOC reduction through the membrane averaged 50 and 60 percent for MF and UF testing with HIOPs. A comparison of TOC removal to GAC columns with a 15-minute empty bed contact time, ACH, and HIOPs are shown in Figure 15–14.

The results of the testing of the two types of membranes are summarized in Table 15–7. Based on both lower capital and operating costs, the MF system was selected.

The pilot study at Erie provided the operating parameters to design a 4.0 MGD membrane treatment plant. The town prepurchased an MF system using a competitive bid process. The plant design includes raw-water pumping and automatic prescreening,

TABLE 15–7. Summary of Pilot Plant Results—Erie, Colorado

Constituent	Microfiltration	Ultrafiltration
1. Turbidity goal: <0.1	<0.1 in 99%	<0.1 in 99%
2. Microorganisms goal: 4 Log	>8 Log	>7 Log
3. THMFP goal: <80 μg/L	>130 μg/L	>135 μg/L
4. TOC removal goal: 20%	Carter—21% 50/50—18% 70/30—11% Erie — 7%	Carter—23% 50/50—19% 70/30—19% Erie —10%
5. CIP interval goal: 14 day min.	Carter: 30 day Erie: 14 day	Carter: 30 day Erie: 3 day
6. Estimated annual O&M	\$0.35/1,000 gal.	\$0.56/1,000

(Courtesy of Erie Public Works Department)

membrane filtration, GAC treatment, and chemical addition in a pressurized system so that water flows to the new clearwell without additional pumping required. The waste system utilizes several options for the plant operators to recover or recycle settled backwash waste in the process or as irrigation water. Sludge waste is discharged to a sanitary sewer.

NANOFILTRATION AND REVERSE OSMOSIS

Introduction

Reverse osmosis (RO) membranes first became commercially available in the 1960s for treating seawater and brackish water. Today, RO is the most common type of membrane process for potable water production in the United States, though it is generally used for smaller plants. Nanofiltration is a much newer category of membrane filtration with relatively few installations but rapidly growing interest in the technology.

RO and NF membranes can separate a large spectrum of contaminants from water, including turbidity, pathogens, salts, hardness, heavy metals, and natural and synthetic organics. The only substances that are not typically separated by membranes are dissolved gasses.[11] The advantage of RO and NF is that they remove substances at the molecular and ionic levels, removals that UF and MF membranes are largely incapable of. However, RO and NF membranes require much higher operating pressures than UF and MF membranes.

Typical Applications

RO is most commonly used for desalination of brackish waters in the United States, where there are over 140 operating plants.[12] The lower salt concentrations of brackish water allow for more efficient treatment of brackish water as compared to seawater. Generally, RO is not cost effective for large-scale treatment of seawater in the United

States; however, it is used for this purpose in regions where water resources are more scarce, particularly the Middle East.

NF membranes are sometimes called loose RO membranes or low-pressure RO membranes, because they exhibit lower removal capabilities and operate at lower pressures than do RO membranes. NF membranes were first developed for softening and are commonly used on the hard groundwaters found in Florida. Nanofilters generally do not remove monovalent ions to the extent that RO membranes do, but are capable of high rejections of divalent ions, such as calcium and magnesium. The lower operating pressures used in NF make it a more efficient process than RO.

RO and NF are also used for controlling specific ions. Some of the more common applications are the removal of fluoride, nitrate and nitrite, ammonium, and phosphate. The anticipation of reduced limits on arsenic in drinking water has spurred interest in RO and NF for removing this contaminant.[13,14]

Both RO and NF are capable of rejecting organic matter from water, and for this reason are attractive for the removal of DBP precursors and color.[15–17] The extent of membrane applications for these purposes has been limited by costs, mainly resulting from pretreatment requirements and membrane fouling. However, more stringent regulations and improvements in the technology have made the process more feasible. The Information Collection Rule, promulgated in 1996, has required utilities to test new processes for DBP precursor control; as a result, 37 utilities are conducting tests with nanofiltration.[18]

Synthetic organic chemicals, and pesticides in particular, can also be removed by RO and NF processes. European countries and parts of the United States that rely on polluted surface waters have looked to membranes for pesticide removal.[19,20] The feasibility of the treatment varies greatly with the target pesticide, as well as other conditions specific to the application.

Membrane Characteristics

Several characteristics are important to the performance and longevity of RO and NF membranes:

- Mechanical strength
- Resistance to chemicals, hydrolysis, and biological attack
- Temperature stability
- Selectivity
- Productivity

RO and NF membranes require different characteristics than do UF and MF membranes owing to the different operating conditions and removal mechanisms. RO and NF membranes operate at much higher transmembrane pressures than do UF and MF membranes, and for this reason must have a greater mechanical strength. High transmembrane pressures can cause membrane compaction, thereby reducing productivity, or even membrane rupture.

RO and NF membranes can remove particles by physical straining processes, as do UF and MF membranes, but the key function of RO and NF is the removal of dissolved contaminants. The separation of solutes (e.g., salts and organic compounds) by RO and NF membranes is controlled by mechanisms not applicable to UF and MF, mainly diffusion, charge interactions, and steric effects (i.e., the arrangement of atoms in a molecule). The membrane material has a large impact on these mechanisms. Generally, RO and NF membranes are made of hydrophilic materials, which means the material has a high affinity for water. Compared to a hydrophobic membrane, a hydrophilic membrane passes water with less resistance and is less likely to bind with hydrophobic contaminants, which can result in fouling. However, the hydrophilic materials have some disadvantages compared to the hydrophobic ones, such as less mechanical strength and less resistance to chemical and biological degradation.[11]

The early RO membranes were made from cellulose acetate (CA) and were *asymmetrical*. Asymmetrical membranes are all one type of material, but have a thin membrane skin on a support backing. Cellulose acetate, as well as cellulose diacetate and cellulose triacetate, is still widely used today for RO and NF membranes. These materials provide a moderate tolerance to chlorine and are relatively hydrophilic, but are somewhat susceptible to hydrolysis, biological attack, and deformation above 30°C. Asymmetrical membranes made of polyamides are also common; their primary limitation is poor tolerance of chlorine, though they are relatively resistant to biological degradation.[21]

Thin film composite (TFC) membranes have more recently been used for RO and NF. TFC membranes also consist of a thin membrane skin on a support backing, but the membrane skin is a different material than the support material. Materials used in TFC membranes include polyamides, polyurea, polypiperazineamide, and polysulfone.[20]

Within the categories of RO and NF, membranes are characterized based on their removal capabilities. Typically, the manufacturer will rate its membranes for a specific rejection capability. Membranes used for desalination and softening are typically rated in terms of removal of specific salts, such as NaCl or $MgSO_4$. For example, a membrane for desalting seawater may reject 99 percent of NaCl under specific feedwater and operating conditions. An NF membrane may reject less than 50 percent of NaCl but 90 percent of $MgSO_4$. However, actual membrane performance can vary substantially with conditions specific to the application. Also, membrane performance for organics removal is typically harder to predict than the removal of salts, due to the greater variation and complexity of the organic compounds.

RO and NF membranes are also characterized by MWCO, as are ultrafilters. The MWCO, however, does not directly relate to the rejection of all molecules, because rejection is dependent upon the specific characteristics of the contaminant and membrane. For example, a nonpolar molecule of low molecular weight may be more highly rejected than a polar molecule of high molecular weight. The MWCO is just a rough measure by which membranes can be compared. RO and NF membranes that are rated by MWCO are typically in the range of 200 to 1,000 daltons.

Design Criteria and Performance

General operating and performance criteria for RO and NF are presented in Table 15–8. RO and NF membranes are available from several manufacturers, including:

- Fluid systems
- Hydranautics
- Triseps

TABLE 15–8. RO and NF Operating and Performance Criteria

Process	Operating Pressure	Product Water Flux	Recovery
Nanofiltration	80–150 psi	15–25 gfd	70–90%
Reverse osmosis	150–600 psi	3–20 gfd	30–85%

Note: gfd = gallons/sq ft/day.

- Osmonics
- Dow
- DuPont

These manufacturers can provide technical assistance on specific applications and many have software available to design engineers. These computer models can be used to help select membrane configurations and predict performance.

The operating pressures for RO and NF are significantly higher than for MF and UF. However, recent advances in membrane technology have reduced operating pressure requirements and several manufacturers have developed ultra-low-pressure membranes (100–150 psi). In order to maintain the flux of product water during operation, the operating pressure of a membrane system usually must be increased with time. The accumulation of contaminants upstream of the membrane increases the osmotic pressure with time. Also, solids accumulation on the membrane and membrane fouling, compaction, and degradation can reduce productivity if the operating pressure is not increased.

Product water flux varies from one membrane to another, primarily due to membrane material and manufacture. For a given membrane, flux is dependent upon a number of conditions, including feedwater quality and recovery, as well as operating pressure. The temperature of the feedwater affects the flux of product water through viscosity; a higher temperature feedwater results in a higher product water flux. The concentration of contaminants in the feed stream also influences flux.

The recovery of product water through a RO or NF system is generally lower than with MF or UF. High recoveries are more difficult to achieve with RO and NF membranes because of the resulting high concentration of dissolved contaminants in the concentrate.

The removal capabilities of RO and NF systems vary with several factors, including the specific membrane used, contaminant of concern, operating conditions, and feedwater quality. Generally, RO membranes are used if the removal of monovalent ions is required. NF membranes are used for the rejection of divalent ions (e.g., softening) and organics. Membrane manufacturers can give an estimate of rejection capabilities of their membrane for some contaminants based on anticipated feedwater characteristics and operating conditions.

Pretreatment Requirements

Most applications of membrane systems require pretreatment to ensure effective and reliable treatment. Pretreatment may be used to aid in contaminant removal or minimize plugging, scaling, and fouling. Pretreatment processes can be categorized as follows:

- Suspended solids control
- Scaling control
- Microbiological control
- Specific inorganics control
- Organics control
- pH control
- Temperature control

Suspended Solids Control Suspended solids must be removed from virtually all RO and NF systems to prevent plugging of the membrane system and particle accumulation on the membrane itself. For waters with low particulate concentrations, such as a groundwater, a cartridge filtration system or a fine mesh screen is typically used. For waters with moderate to high suspended solids concentrations, more extensive pretreatment including coagulation-flocculation-sedimentation, granular-media filtration, microfiltration, or ultrafiltration may be beneficial or necessary.[22]

Manufacturers typically specify limits on suspended solids in terms of either turbidity or silt density index (SDI). For example, one membrane manufacturer strongly recommends that pretreatment equipment be designed to routinely attain feedwater turbidity of less than 0.2 NTU and states that the maximum allowable feedwater turbidity is 1.0 NTU.

The SDI is determined by the rate at which a 45-μm filter becomes plugged and is covered by ASTM D4189. Generally, feedwater to an RO membrane should have an SDI less than 2, though this may not be necessary for all applications.[11]

Scale Control The concentration of contaminants in the membrane feedwater increases with source water concentration and the recovery at which the system is operated. In some RO and NF systems, the feedwater concentration becomes 10 or more times greater than the source water concentration. The concentration at the membrane surface can be even higher than that. These high solute concentrations can exceed the solubility product of certain compounds, leading to scale formation in the membrane system. Typical contaminants of concern include $CaCO_3$, $CaSO_4$, $BaSO_4$, $SrSO_4$, and SiO_2. Scale can reduce product water flux, decrease permeate quality, and damage the membranes.

Pretreatment to control scaling is very common in RO and NF systems and can include one or more of the following:

- Acid or base addition to adjust pH and alkalinity
- Softening to reduce calcium concentrations
- Antiscalant addition to influence solubility
- Reducing the system recovery to reduce solute concentrations

Microbiological Control Microbial growth can clog or foul membranes and cause degradation of the membrane surface. The most common methods of controlling microbial activity are the use of disinfectants and chemical cleaning agents.[23] Chlorination of the feedwater is often attractive because of its use as a primary and secondary disinfectant; however, some membrane materials cannot withstand chlorine. Also, prechlorination of the feed stream can create unwanted DBPs. Alternative disinfectants, such as hydrogen peroxide, chloramines, and ultraviolet radiation,[23] applied to the feedwater can be used. Chemical cleaners are also commonly used to control microbial growth, though reliance on frequent membrane cleaning is not desirable due to costs associated with the cleaning operation and plant downtime. Nearly all membrane cleaners include a detergent to remove biofilms from the membrane surface. Other generic components of membrane cleaning solutions for control of microbial growth include enzymes, biocides, chelating agents, and chaotropic agents.[23]

Prefiltration or other contaminant removal processes can reduce microbial growth by reducing the loading on the membranes. These pretreatments are further described under suspended solids control and organics control.

Organics Control While some RO and NF systems are designed to remove organics, in many cases organic compounds can be problematic because they cause fouling and other damage to membranes. Natural organic material can bind reversibly and irreversibly to the membrane, reducing the membrane permeability to water. The impact of the organics can vary greatly from one application to another. Typically, pilot testing is used to determine the effect of natural organic matter on membrane performance. Oils and other hydrocarbons, as well as organic solvents, can also bind to or otherwise damage membranes.

Pretreatment options for removing organics include conventional coagulation, flocculation, sedimentation, and granular-bed filtration; in-line coagulation with MF or UF; or treatment with activated carbon or iron oxide particles followed by MF or UF. Some research has shown positive results for these processes reducing fouling by organics;[24] however, experience with controlling organic fouling, particularly with surface waters, is relatively limited.

Fouling by organics can also be controlled to some extent with membrane cleaning. Cleaning agents used to remove organic compounds include detergents, oxidizing agents, and bases. Some organic materials, however, bind irreversibly to membranes and cannot practically be removed.

pH Control Depending on the raw water and other pretreatments, pH control upstream of the membranes may be necessary. The manufacturer should recommend the acceptable pH range for a given membrane. Cellulose acetate membranes are usually operated in a pH range of 4.0 to 6.5. Outside of this range, the material can begin to undergo hydrolysis. Polyamide membranes can typically tolerate pH up to 10.

pH can also affect the properties of solutes and membranes. Natural organic matter will take on a different configuration depending on pH, thereby influencing its removal.[25] The charge of ions is important to ion passage through membranes and can be influenced by pH.

RO and NF Process Configurations

Plant Process Configuration. Figure 15–15 shows a basic process schematic for RO and NF plants; typically, additional treatment steps are required to achieve treatment objectives. Prefiltration may be accomplished with a fine mesh screen, cartridge filter, microfilter, or ultrafilter. More extensive pretreatment may include in-line coagulation or conventional treatment. Most RO and NF membranes remove a large percentage of

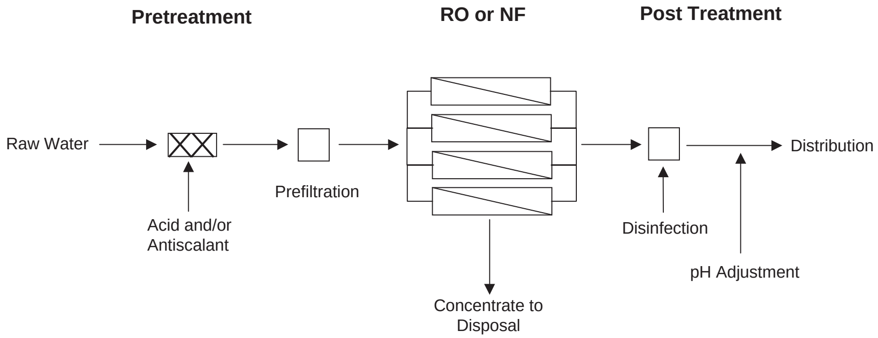


Fig. 15–15. RO or NF treatment plant schematic

the inorganic content from the raw water, resulting in a highly corrosive permeate stream. For corrosion control, pH is adjusted and sometimes a corrosion inhibitor is added. With many groundwater applications, treatment such as aeration is required downstream of the membranes to remove carbon dioxide and hydrogen sulfide.

Use of RO and NF may also be considered as a polishing step following a more conventional treatment. A 500-gpm nanofiltration prototype plant has been supplying water to Auvers-sur-Oise, France, for this purpose since 1993.[26,27] Feedwater to the prototype plant is first treated in the larger convention plant by coagulation, ballasted flocculation, sedimentation, ozonation, and granular bed filtration. The nanofiltration step is used to remove pesticides and DBP precursors and provide an additional barrier to microorganisms.

Membrane Process Configuration. The performance of a single RO or NF module is typically inadequate in terms of system recovery, product water quality, or both. For this reason, RO and NF modules are arranged in an array of many modules. The array is configured to either increase product water recovery or improve product water quality.

A concentrate-staged array is used to increase recovery. A typical, single spiral-wound RO or NF element achieves a recovery of 8 to 15 percent. In order to increase recovery, several elements are arranged in series into what is called a module or pressure vessel. In a hollow-fiber system, bundles of the membrane fibers are contained in each module. To further increase recovery from both spiral-wound and hollow-fiber systems, the modules are arranged in a concentrate-staged array. The concentrate stream from the first set of membrane modules is sent to a smaller number of second-stage modules. The second set of modules increases the overall recovery of product water. Often a third stage is added to further increase recovery.

When product water quality is insufficient through a single module, a permeate-staged array can be used. Permeate from the first set of modules is passed through a second set of modules for further treatment. The permeate-staged array is only common for seawater of very high salinity, and when product water quality requirements are more strict than usual drinking water standards.

In some applications, a RO or NF system will provide product water quality much greater than necessary. In these cases, is may be possible to blend treated water with untreated water and still meet finished water quality requirements. If blending is acceptable, the capacity of the membrane system can be minimized and the rate of concentrate generation is reduced. The blending ratio of treated to untreated water can be determined through mass balance.

Case Histories

Palm Beach County, Florida In 1996, the Palm Beach County Water Utilities Department (PBCWUD) began operating Water Treatment Plant No. 3, a new 9.4-mgd nanofiltration softening plant. Located adjacent to an existing lime softening plant, the new plant is used to meet increasing water demand. Groundwater is pumped to the plant from the Biscayne Aquifer through wells approximately 175 feet deep. Typical raw water and finished water qualities are presented in Table 15–9.

Pretreatment first consists of sulfuric acid and antiscalant addition to prevent precipitation of carbonate and sulfate, respectively, on the membranes. Flow then passes through 5-μm cartridge filters for the removal of silt, sand, and other suspended particles. Following pretreatment, water is fed to one of four membrane trains. Characteristics of the nanofiltration membrane and system are shown in Table 15–10.

Following membrane filtration, process flow is passed through aeration towers in order to strip dissolved hydrogen sulfide and carbon dioxide. Sodium hydroxide is then added to raise the pH. At this point, treated water from the NF plant is blended with treated water from the adjacent lime softening plant. The combined flow is dosed with free chlorine and ammonia to form chloramines for disinfectant residual.

The overall recovery from the membrane process is about 85 percent. The 15 percent of flow that remains as concentrate (maximum of 1.7 million gallons generated daily) is disposed of by deep-well injection. The injection wells are approximately 3,000 feet deep. Monitoring is conducted to ensure that the concentrate does not migrate to the aquifer above, which is used for source water.

TABLE 15–9. Typical Raw and Finished Water Qualities for PBCWUD Treatment Plant No. 3

Parameter	Unit	Raw Water	Finished Water
Turbidity	NTU	NA	0.01
pH		7.2	9.0
Alkalinity	mg/L as $CaCO_3$	219	35
Total hardness	mg/L as $CaCO_3$	266	45
Total dissolved solids	mg/L	384	95
Color	PCU	32	1
Total organic carbon	mg/L	11.8	<0.5
Dissolved oxygen	mg/L	<1–2*	NA
Sulfate	mg/L SO_4	20*	NA
Temperature	°C	25	NA

NA = not available.

*From reference 28.

TABLE 15–10. Nanofiltration Membrane and System Characteristics for PBCWUD Treatment Plant No. 3

Membrane molecular weight cutoff*	200 daltons
Active membrane area	400 sq ft per element
Average permeate flux	15 gfd
Maximum transmembrane pressure	120 psi
Membrane type	Thin-film composite
Membrane materials*	
Active layer	Cross-linked aromatic polyamide
Support	Polysulfone
Substrate	Polyester fabric
No. of membrane treatment trains	4
Stages of filtration per train	2
First stage vessels per train	35
Second stage vessels per train	21
Total vessels per train	56
Membrane elements per vessel	7
Membrane element diameter	8 inches
Membrane element length	40 inches
Membrane manufacturer*	Fluid Systems (TFCS8929ULP)

*From reference 28.

Riverside, California The Arlington Desalter in Riverside, California, is owned and operated by the Santa Ana Watershed Project Authority. The RO system treats groundwater that has been degraded through over 100 years of irrigated agriculture that resulted in increased salt and nitrate concentrations, leaving the water unfit for domestic use. The following summary outlines operating characteristics of the system:

- Start-up: 1990
- Capacity: 4.0 mgd product water
- Recovery: 76 percent
- Typical operating pressure: 200 psi
- Membrane type: thin film composite
- Brine disposal to nonreclaimable waste line that discharges to a regional wastewater treatment plant

The following water quality data summary is from the plant after 10 years of operation with the original membranes that are still in service:

	Concentration, mg/L	
Constituent	Feed	Permeate
TDS	1,130	76
Total alkalinity as $CaCO_3$	317	18
Total hardness as $CaCO_3$	541	15
Chloride	175	10
Nitrate nitrogen	18	3

ELECTRODIALYSIS

General

Electrodialysis (ED) is a well-developed process with over 30 years of operation on brackish well water supplies. Electrodialysis works by using electrical energy to force ions from the feed stream into the reject stream through ion exchange membranes. Electrodialysis reversal (EDR) is the same as ED except that the polarity of the charges on the membrane is periodically reversed, causing reverse flow. The cyclic reversal of flow tends to reduce fouling and extended membrane life.

ED/EDR Applications

The principal application of ED/EDR for potable water treatment is the desalination of brackish water. ED/EDR competes well with RO when the TDS concentration is less than 5,000 mg/L.

Typical removals of inorganic salts (listed in Table 15–11) from brackish water by ED range from 25 to 40 percent of dissolved solids per stage of treatment. Higher removals require treatment by multiple stages in series. Energy required for ED is about 0.2 to 0.4 kWh/1,000 gallons (52.8 to 105.7 kWh/ML) for each 100 mg/L dissolved solids removed, plus 2 to 3 kWh/1,000 gallons (528 to 798 kWh/ML) for pumping feedwater and brine. This corresponds to an energy requirement of 2.6 to 4.2 kWh/1,000 gallons (687 to 1,110 kWh/ML) of product water to remove 300 mg/L of dissolved solids.

In the ED process, brackish water flows between alternating cation-permeable and anion-permeable membranes, as illustrated in Figure 15–16. A direct electric current (DC) provides the motive force to cause ions to migrate through the membranes. Many alternating cation and anion membranes, each separated by a plastic spacer, are assembled into membrane stacks. Figure 15–17 shows the membrane arrangement. The spacers contain the water streams within the stack and direct the flow of water through a circuitous path across the exposed face of the membranes. Several hundred membranes and their separating spacers are usually assembled between a single set of electrodes to form a membrane stack. End plates and tie rods complete the assembly.

TABLE 15–11. Typical Contaminants Removed with ED/EDR

Contaminants			
Sodium	Nickel	Chloride	Fluoride
Potassium	Chromium	Sulfate	Chromate
Calcium	Copper	Nitrate	Acetate
Magnesium	Zinc	Phosphate	Hydroxyl
Ammonium	Strontium	Cyanide	Conductivity
Arsenic	Iron	Silver	TDS
	Aluminum		

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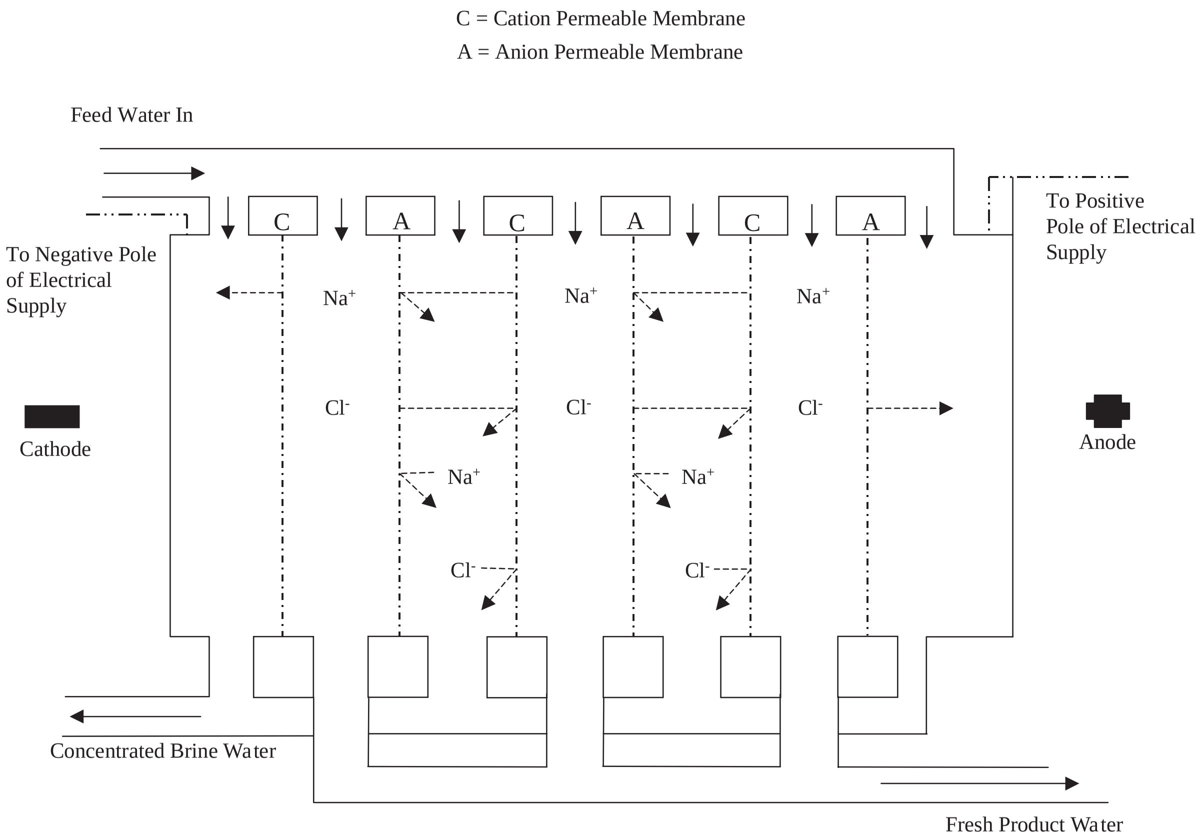


Fig. 15–16. Electrodialysis demineralization process

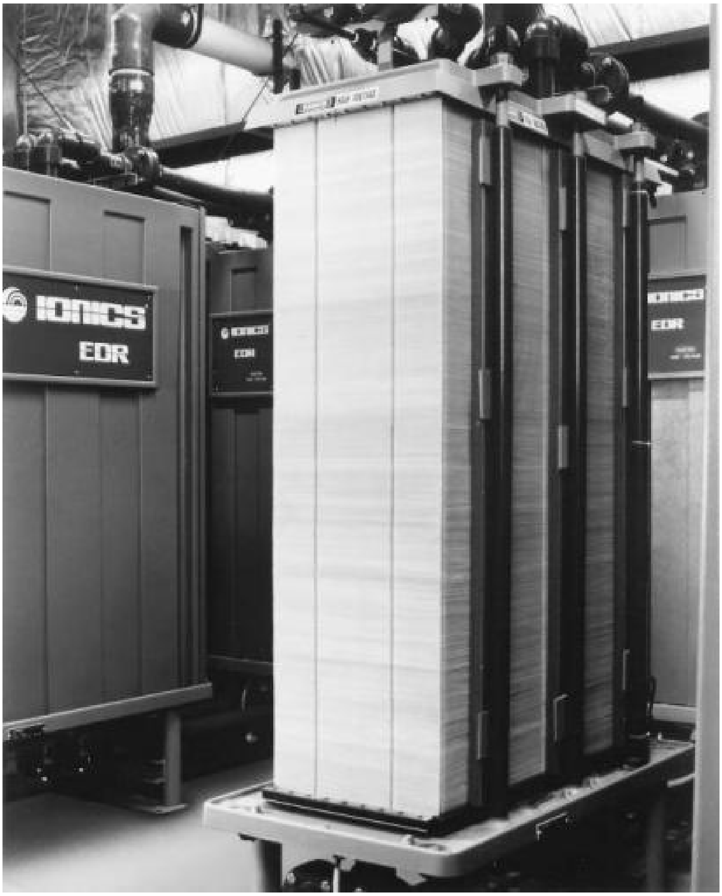


Fig. 15–17. Electrodialysis membrane stack arrangement (Courtesy of Ionics)

When a membrane is placed between two salt solutions and subjected to the passage of a direct electric current, most of the current will be carried through the membrane by ions; hence, the membrane is said to be ion-selective. Typical selectivities are greater than 90 percent. When the passage of current is continued for a sufficient length of time, the solution on the side of the membrane that is furnishing the ions becomes more concentrated. These desalting and concentrating phenomena occur in thin layers of solution immediately adjacent to the membrane, resulting in the desalting of the bulk of the solution. Figure 15–18 is an illustration of typical membrane reactions.

Passage of water between the membranes of a single stack, or stage, usually requires 10 to 20 seconds, during which time the entering minerals in the feedwater are removed. The actual percentage removal that is achieved varies with water temperature, type and amounts of ions present, flow rate of the water, and stack design. Typical removals per stage range from 25 to 40 percent; systems employ one to six stages. An ED system will operate at temperatures up to 100°F (43°C), with the removal

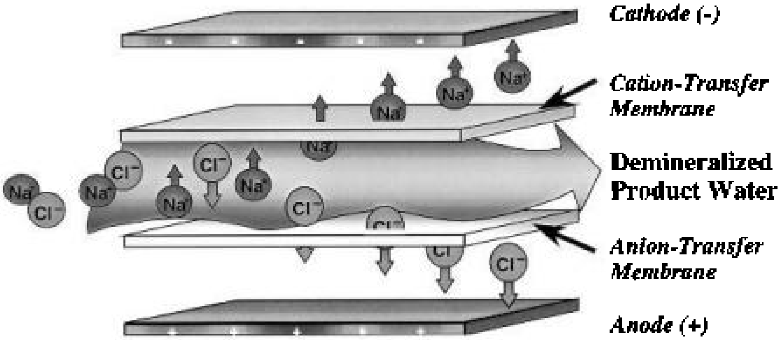


Fig. 15–18. A schematic of typical electrodialysis membrane (Courtesy of Ionics)

efficiency increasing with increasing temperature. Ion-selective membranes in commercial electrodialysis equipment are commonly guaranteed for as long as 5 years, and experience has demonstrated an effective life of over 10 years.

Since 1974, virtually all ED plants constructed have been the EDR type. Until the EDR process was developed, all ED plants (and RO as well) operated with the transfer of water or salts in one direction. Membranes had a brine side and a product side. Films of scale, slime, and other deposits generally formed on one side of the membrane only.

Partially successful attempts were made to control the precipitation of insoluble salts such as calcium carbonate and calcium sulfate on the brine side of unidirectional membrane processes by the addition of acids, polyphosphates, or similar agents. These chemical additions can control calcium carbonate scale effectively, and calcium sulfate scale less effectively, when precise dosing is maintained. However, when errors, failures, or upsets occur, scale starts to form at an accelerating rate.

The symmetrical nature of ED membranes allowed the development of the EDR process. The EDR system utilizes a standard electrodialysis array of alternating anion membranes, separated by alternating product and brine compartments, as shown in Figure 15–16. The array is operated in the standard ED manner for a fixed period of time—for example, 20 minutes—and then the process is reversed by an automatic timing circuit in the following sequence:

1. The direction of the DC field is reversed by reversing the polarity of the electrodes. This polarity reversal immediately begins converting the product compartments into brine compartments, and the brine compartments into product compartments, by reversing the direction of flow of the ions.

2. Automatic valves interchange the feed to and discharge from the product and brine compartments.

3. There is a one- to two-minute period immediately following the polarity reversal when the water from both sets of compartments is of lower quality, and both streams are automatically diverted to waste. This ‘‘purge’’ of the brine and product compartments every 20 minutes breaks up polarization films, carries off loose scale, and reduces the tendency of deposits to build up.

In operation, the EDR process requires the addition of a timing-control unit, automatic valves to interchange the product and brine streams, and relays to reverse the polarity of the DC power supply.

Fouling

The EDR process has greatly reduced membrane scaling and fouling. In many plants, the need for chemicals to control scaling and fouling has been eliminated through use of an EDR system.

Fouling of ED/EDR membranes is still a problem when dissolved organic and inorganic compounds in the raw water are above recommended level. Organic fouling is caused by the precipitation of large, negatively charged molecules on the anion-permeable membranes in the feed channels. Inorganic fouling is caused by the precipitation (scaling) of inorganic compounds (such as $CaSO_4$ and $CaCO_3$) in the concentrate compartments. Also the negative effects of ED membrane fouling include:

- Cations such as Fe and Mn can absorb on the cation-permeable membranes.
- Attached ions can neutralize or even reverse the fixed charge of the membranes, causing significant reduction in efficiency.
- Fouling increases the membrane stack electrical resistance, increasing electrical consumption and operating costs.

The following items can result in membrane fouling:

- Metals such as Fe, Mn, and Cu
- Dissolved gases such as O_2, CO_2, and H_2S
- Silica (colloidal and soluble)
- Fine particulates and colloidal material of a wide range of sizes and composition
- Divalent cations (Ca, B)

Many of these foulants present in raw water may be controlled by pretreatment steps which usually stabilize the ED/EDR process. However, according to Katz, the development of the EDR process has helped to solve the pretreatment problem more readily in that it provides self-cleaning of the vital membrane surfaces as an integral part of the desalting process.[29]

Pretreatment

The overall requirements for pretreatment in ED/EDR are less stringent than for RO due to the nature of the salt separation and the larger passages provided. In ED/EDR, the ions (impurities) move through the membranes, while the desalted water moves through the membranes' feed messages, flushing out solids. With the EDR process, precipitated salts in the brine compartments can be more readily dissolved and flushed out of the system, using polarity reversal without the need for chemical pretreatment.

However, high removals of suspended solids, iron, manganese, organic matter, and hydrogen sulphide are still critical to pretreatment of the feedwater (USAID Desalination Manual, 1980) if it contains the following ions: Fe >0.3 mg/L, Mn $>0,1$ mg

/L, H^2S >0.3 mg/L, free chlorine and turbidity >2 NTU.[30] In every case, of course, careful examination of the prospective water would be necessary to determine suitability and pretreatment. Normally at least a 5-micron cartridge filter should precede the ED/EDR stack.

The precipitation of slightly soluble salts in the standard ED process may be minimized by ion-exchange softening and/or reducing the pH of the brine through acid addition and/or the addition of a precipitation-inhibiting agent. However, a certain degree of fouling is unavoidable. Membranes should be regularly washed or cleaned in place with dilute acid and alkali solutions to restore performance when required.

Case History

Due to the characteristics of Lake Granbury, located in north central Texas, treatment beyond conventional surface water treatment is required to produce a palatable drinking water supply. Lake Granbury is a brackish surface water, which has a total dissolved solids and chlorides concentration approaching 2,200 mg/L and 900 mg/L, respectively. Based on these concentrations, the water tastes salty and does not meet the Minimum Drinking Water Standards for the state of Texas. Therefore, the water must be demineralized before it can be deemed an acceptable water supply.

The first phase of the ultimate 21-mgd facility consists of an intake structure in Lake Granbury, 4,000 ft of raw water line along with a 38-cm (15-in) concentrate return line, a conventional surface water treatment plant with EDR having a capacity of 13.2 ML/d (3.5 mgd), and a transmission system consisting of 38 km (24 miles) of 61-cm (24-in) and 46-cm (18-in) pipeline along with a 11.4-ML (3-mg) standpipe to deliver treated water to the Johnson County participants. Since that time, the EDR plant has effectively reduced the total dissolved solids of the brackish water found in Lake Granbury to produce a potable water in conformance with the requirements of the state of Texas and the Safe Drinking Water Act.

The modular components of the EDR facility makes it easy to expand. In 1998, the capacity was expanded by 1.5 mgd, bringing the total capacity to 5.0 mgd. The improvements included adding raw water and demineralizer feed pump capacity as well as increasing the capacity of the EDR system.

DISPOSAL OF MEMBRANE WASTE STREAMS

Introduction

Disposal of membrane waste streams is a primary consideration in determining the feasibility of a membrane process. The issues involved in disposal range from purely practical to regulatory to political. Many of these issues are common to residual disposal from any water treatment plant. A thorough review of this subject is presented in a handbook published by ASCE, AWWA, and the U.S. EPA[31] and is also discussed in Chapter 23, Residuals Management. Because the application of UF and MF systems for potable water treatment is relatively new, there is not extensive experience with disposal of large volumes of waste from these processes. Most engineering and regulatory work in membrane waste stream disposal has been with RO and NF processes.

Characteristics of Waste Streams

The largest volume of waste from a membrane plant is usually the concentrate. Yet membrane plants generate other waste streams, such as cleaning wastes and wastes from pretreatment processes.

The quantity of concentrate from a membrane process is determined by the recovery at which the facility is operated. Given that the various types of membrane plants can have recoveries anywhere between 30 and 98 percent, the quantity of concentrate can be very high.

The quality of the concentrate stream also depends on the recovery as well as quality of the raw water, level of contaminant rejection, and any pretreatment steps. An RO plant used for brackish water desalting will generate a concentrate stream containing virtually all the raw water contaminants, including an extremely high salt content. On the other hand, an MF plant removing mainly particulate material will generate a much different concentrate stream. If a coagulant or PAC is added upstream of the membrane, the residuals from these chemicals must be handled. The disposal options for all these concentrate streams would be much different.

Various methods are available for predicting concentrate stream quality. Some manufacturers have their own programs for making estimates. Commercially available programs and hand calculations can also be used.[32] All these methods are based on assumed operating conditions and estimates of membrane system performance.

UF and MF plants are typically backwashed or flushed on a regular basis to remove accumulated contaminants from the membrane feed loop and the membrane surface. The quantity of backwash water varies with raw-water quality and the type of membrane system, as some use water only and others use air and water. The supply backwash water is usually just permeate or raw water. The waste backwash contains the same contaminants as the concentrate stream, though concentration of the contaminants may be different.

Virtually all membrane systems require periodic chemical cleaning to remove foulants from the membrane. The volume of chemical cleaners used is much lower than the volume of concentrate and backwash streams. The chemicals used in the cleanings, such as chlorine, sodium hydroxide, and surfactants, normally require that this stream be handled separately.

Membrane facilities also generate wastes from pretreatment steps. Prefilters must be periodically flushed, cleaned, or discarded. Membrane systems requiring more rigorous pretreatment may use conventional treatment processes such as coagulation, flocculation, sedimentation, and/or filtration. These processes and the waste streams associated with them are discussed elsewhere in this book.

Disposal Methods

There are several disposal options for membrane waste streams. Table 15–12 lists the means of concentrate disposal from a survey of 137 operating plants and 52 plants in the planning stages.[32] Note that these plants use either RO, NF, or EDR; no UF or MF plants were operating in the United States at the time of the survey.

Surface Water Discharge Discharge to surface waters is by far the most common method for disposing of membrane concentrates. A number of factors, such as con-

TABLE 15–12. Method of Concentrate Disposal by U.S. Membrane Plants

Type of Disposal	Percentage of Operating Plants	Percentage of Plant in Planning Stages
Surface water discharge	48	60
Sewer discharge	23	21
Land application	12	0
Deep well injection	10	13
Evaporation pond	6	2

(Adapted from *Membrane Concentrate Disposal,* by permission. Copyright © 1993, American Water Works Association and AWWA Research Foundation.)

centrate quality and quantity, characteristics of the receiving water, and regulatory environment, determine the feasibility of the surface discharge. Surface discharge requires an NPDES permit. Water quality and quantity limits, pretreatment conditions, planning requirements, outfall location and design, and monitoring plans must all be considered in applying for a permit. Otherwise, given a nearby receiving water, surface water discharge is relatively simple.

Discharge to Sewer System Discharging concentrates to the sewer is usually limited to smaller membrane plants because of capacity limitations within the collection system and wastewater treatment plant. The concentrate stream must also meet pretreatment requirements to ensure no deleterious effects on the sewer system and wastewater plant. For some concentrates, the cost of pretreatment for sewer discharge can be prohibitive. Some membrane waste streams may be of sufficient quality to bypass a part of the wastewater treatment plant. For example, some concentrates are discharged such that only disinfection is provided. A fee for discharge to the sewer may be required, depending on who controls the water and wastewater agencies.

Ocean Discharge Where a treatment plant is located near the sea, ocean discharge can be an attractive alternative. Ocean discharge is considered separately from surface water discharge due to the general differences in quality, quantity, and uses of these receiving waters. However, as with surface water discharge, an NPDES permit is required. In general, an outfall should be located to maximize mixing and dilution. Contaminants in the concentrate stream, such as heavy metals or high salt concentrations, or conditions such as extremes in pH or low oxygen concentrations, may be harmful to the local marine environment.

Land Application Two land application methods are currently used for membrane concentrate disposal: irrigation systems and rapid infiltration systems. Spray irrigation applies the concentrate to parks, golf courses, and crops and other vegetation. The concentrate waste stream provides a benefit by replacing the use of more valuable water resources. However, spray irrigation is limited by several constraints. The quality of the concentrate must not harm the vegetation, so dilution may be necessary. For crops tolerant of salts, such as golf course grasses and citrus trees, the irrigation water can have a chloride ion concentration no higher than 1,000 mg/L.[31] Adequate land must be also available nearby. Furthermore, local, state, and federal regulations must be met; for example, the concentrate must not adversely affect the groundwater. Often a program is required to monitor the effects of the spray irrigation system.

In rapid infiltration, the concentrate is applied to the land but with little loss to vegetation or evaporation.[32] Typically, a shallow pond is constructed over highly permeable soils. The walls of the pond are lined to prevent horizontal flow. In some cases, the water is recovered through wells or underdrains.

Evaporation Ponds Evaporation ponds are a simple method of concentrate disposal, provided certain conditions are met. The evaporation rate for the area must be relatively high. Also, a fairly large area of land is required near the membrane facility. Evaporation ponds normally have the advantages of being low maintenance and inexpensive. However, the cost advantage of this disposal method can be lost if land costs are high and expensive pond liners are required.

Deep Well Injection Deep-well injection is the pumping of wastes deep into the subsurface. Well depths are generally between 1,000 and 8,000 ft.[32] The objective is to dispose of the waste without contaminating underground drinking water sources. Because of this risk, deep-well injection is strictly regulated.

Regulations Pertaining to Disposal

The regulations and permitting procedures that apply to disposal of membrane concentrate streams vary from state to state, but are based on the framework of federal law. Some of the more commonly applicable regulations and programs are listed below, though not all regulations will apply in every case. A number of other regulations may also apply to a concentrate disposal situation, but less frequently than those mentioned here:[31]

- Clean Water Act (CWA)
- National Pollutant Discharge Elimination System (NPDES)
- National Pretreatment Program
- Safe Drinking Water Act (SDWA)
- Underground Injection Control (UIC)Program
- Coastal Zone Management Act (CZMA)

The CWA requires any plant that discharges residuals to a surface water or ocean to have an NPDES permit. No specific effluent guidelines have been set for water treatment facilities. Permit limits will generally be technology based, though water quality standards may also be implemented if justified by the characteristics and uses of the receiving water.[31] Water treatment plants that discharge to a wastewater treatment plant do not require a NPDES permit. However, the CWA requires that pretreatment requirements be met. Generally, pretreatment is required to ensure that the wastewater facility can meet its NPDES permit limits.

Disposal of membrane concentrates by deep-well injection is regulated under the SDWA by the UIC program. The purpose of the regulations is to prevent contamination of drinking water sources. The SWDA and UIC program may also apply if land application of membrane concentrates is to be used. The irrigation water generally cannot have an adverse affect on the groundwater, users of the groundwater, or the surrounding environment.[30]

Discharge of membrane concentrates to the ocean or surface waters may need to meet requirements of the CZMA. The CZMA, which is administered by the National Oceanic and Atmospheric Administration, applies to the ocean coasts, coasts of the Great Lakes, and some rivers along these coastlines.[30]

REFERENCES

1. Bergman, R. A. ‘‘Overview of Membrane Filtration for Municipal Water Treatment,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.
2. Lyonnaise des Eaux, Water Research Commission of South Africa, AWWARF, ‘‘Water Treatment Membrane Processes,’’ 1996.
3. Michaels, A. S., ‘‘Membranes, Membrane Processes, and their Applications: Needs, Unsolved Problems, and Challenges of the 1990’s,’’ *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 77, Nos. 1–3, 1989.
4. Pohland, H. W., ‘‘Reverse Osmosis,’’ in W. Lorch (ed.), *Handbook of Water Purification,* McGraw-Hill, London, 1981.
5. Rautenbach, R., and Janisch, I., ‘‘Reverse Osmosis for the Separation of Organics from Aqueous Solutions,’’ in S. Stucki (ed.), *Process Technologies for Water Treatment,* Plenum Press, pp. 25–41, 1988.
6. Belfort, G., ‘‘Desalting Experience by Hyperfiltration (Reverse Osmosis) in the United States,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications,* Academic Press Inc., pp. 221–280, 1984.
7. Witmer, F. E., ‘‘Low Pressure RO Systems—Their Potential in WateReuse Applications,’’ in American Institute of Chemical Engineers, L. K. Cecil (ed.), *Complete WateReuse,* pp. 608–618, 1973.
8. Conlon, W. J., ‘‘Membrane Processes,’’ in American Water Works Association, F W. Pontius (ed.), *Water Quality and Treatment,* McGraw-Hill, pp. 709–746, 1990.
9. AWWARF, ‘‘Membrane Filtration for Microbial Removal,’’1997.
10. Gere, A. R., ‘‘Microfiltration Operating Costs,’’ *JAWWA,* Vol. 89, No. 10, October 1997, pp. 40–49.
11. Taylor, J. S., and Jocobs, E. P., ‘‘Reverse Osmosis and Nanofiltration,’’ in *Water Treatment Membrane Processes,* J. Mallevialle et al. (eds.), AWWARF, Lyonnaise des Eaux, and Water Research Commission of South Africa, published by McGraw-Hill, pp. 9.1–9.70, 1996.
12. Morin, O. J., ‘‘Membrane Plants in North America,’’ *JAWWA,* Vol. 86, No. 12, pp. 42–54, 1994.
13. Waypa, J. J., Elimelech, M., and Hering, J. G., ‘‘Arsenic Removal by RO and NF Membranes,’’ *JAWWA,* Vol. 89, No. 10, pp. 102–114, 1997.
14. Brandhuber, P., and Amy, G., ‘‘Mechanisms of Arsenic (V) Removal in Membranes,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.
15. Fu, P., Ruiz, H., Thompson, K., and Spangenberg C., ‘‘Selecting Membranes for Removing NOM and DBP Precursors,’’ *JAWWA,* Vol. 86, No. 12, pp. 55–72, 1994.
16. Reiss, C. R., Taylor, J. S., and Robert, C., ‘‘Successful Nanofiltration of 20 mg/L TOC River Water,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.
17. Amy, G. L., Alleman, B. C., and Cluff, C. B., ‘‘Removal of Dissolved Organic Matter by Nanofiltration,’’ *Jour. of Environmental Engineering,* Vol. 116, No. 1., pp. 200–205, 1990.
18. Allgeier, S. C., ‘‘ICR Membrane Studies: Status Update and Data Analysis,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.

19. Hoffman, J. A. M. H., Noij, Th. H. M., Kruithof, J. C., Schippers J. C., ‘‘Removal of Pesticides with Nanofiltration,’’ AWWA Membrane Technology Conference, Baltimore, MD, 1993.
20. Seacord, T. F., and Crozes, G., ‘‘Low-Pressure Nanofiltration for NOM and Atrazine Removal from the Missouri River Water Supply,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.
21. Peterson, R. J., ‘‘RO/NF Membranes and Performance Characteristics,’’ AWWA Membrane Technology Conference, Baltimore, MD, 1993.
22. Chellam, S., Jacangelo, J. G., Bonacquisti, T. P., and Schauer, B. A., ‘‘Effect of Pretreatment on Surface Water Nanofiltration,’’ *JAWWA,* Vol. 89, No. 10, pp. 77–89, 1997.
23. Ridgway, H. F., and Flemming, H.-C., ‘‘Membrane Biofouling,’’ in *Water Treatment Membrane Processes,* J. Mallevialle et al. (eds), AWWARF, Lyonnaise des Eaux, and Water Research Commission of South Africa, published by McGraw-Hill, pp. 6.1–6.62, 1996.
24. Robert, C., Reiss, C. R., Taylor, J. S., and Langenderfer N., ‘‘Surface Water Treatment by Integrated Membrane Systems with In-Line Coagulation,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.
25. Braghetta, A., and DiGiano, F. A., ‘‘Organic Solute Association with Nanofiltration Membrane Surface: Influence of pH and Ionic Strength on Membrane Permeability,’’ AWWA Annual Conference, New York, NY, 1994.
26. Ventresque, C., Turner, G., and Bablon, G., ‘‘Nanofiltration: from Prototype to Full Scale,’’ *JAWWA,* Vol. 89, No. 10, pp. 65–76, 1997.
27. Agbekodo, K. M., Legube, B., and Coté P., ‘‘Organics in NF Permeate,’’ *JAWWA,* Vol. 88, No. 5., pp. 67–74, 1996.
28. Escobar, I. C., and Randall A. A., ‘‘Influence of NF on Distribution System Biostability,’’ *JAWWA,* Vol. 91, No. 6, pp. 76–89, 1999.
29. Katz, W. E., ‘‘Desalination by ED and EDR—State of the Art in 1981,’’ *Desalination*, Vol. 42, No. 2, pp. 129–139, 1982.
30. Buros, O. K., *The U.S.A.I.D. Desalination Manual,* U.S. Agency for International Development, Washington, DC, August 1980.
31. AWWA Technology Transfer Handbook: ‘‘Management of Water Treatment Plant Residuals,’’ 1996.
32. Mickley, M. et. al., ‘‘Membrane Concentrate Disposal,’’ AWWA Research Foundation, Denver,1993.

SUPPLEMENTARY READING

Abaticchio, P., et al., ‘‘Characterization of Ultrafiltration Polymeric Membranes,’’ *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 78, No. 2, pp. 235–253, 1989.

Adham, S. S., et al., ‘‘Predicting and Verifying Organics Removal by PAG in Large Scale UF Systems,’’ Proceedings of the Membrane Technology Conference AWWA–IWSA, Baltimore, 1993.

Anselme, C., et al., ‘‘Drinking Water Production by Ultra-Filtration and PAC Adsorption and First Year of Operation for a Large Capacity Plant,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.

AWWA Membrane Technology Research Committee Committee Report: ‘‘Membrane Processes in Potable Water Treatment,’’ *JAWWA,* Vol. 84, 1:59–67, 1992.

AWWA Water Desalting and Reuse Committee, ‘‘Committee Report: Membrane Desalting Technologies,’’ *JAWWA,* Vol. 81, No. 11, pp. 30–37, 1989.

Baier, J. H., Lykins, B. W., Fronk, C. A., and Kramer S. J., ‘‘Using Reverse Osmosis to Remove Agricultural Chemicals from Groundwater,’’ *JAWWA*, Vol. 79, No.8, pp. 55–60, 1987.

Belfort, G. ‘‘Desalting Experience by Hyperfiltration (Reverse Osmosis) in the United States,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications,* Academic Press Inc., pp. 221–280, 1984.

Benjamin, M. M., ‘‘Removal of NOM and Natural Colloids with the HIOP/UF System: Bench and Pilot Studies,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.

Brink, L. E. S., and Romijn, D. J., ‘‘Reducing the Protein Fouling of Polysulfone Surfaces and Polysulfone Ultrafiltration Membranes: Optimization of the type of Presorbed Layer,’’ *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 78, No. 2, pp 209–231, 1989.

Buros, O. K., ‘‘Desalting Practices in the United States,’’ *JAWWA*, Vol. 81, No. 11, pp. 38–42, 1989.

Campos, C., et. al. ‘‘Effect of Process Design and Operating Procedures on the Performance of PAC/UF Systems for the Removal of Organic Compounds,’’ AWWA Membrane Technology Conference, Long Beach, CA, 1999.

Collins M. R., et al., ‘‘Assessment of MS2 Bacteriophage Virus, Giargia Cyst and Crypyosporidium Oocyst Removal by HF UF Polysulfone Membranes,’’ Submitted to Koch Membrane Systems, 1994.

Conlon, W. J., and McClellan S. A., ‘‘Membrane Softening: A Treatment Process Comes of Age,’’ *JAWWA*, Vol. 81, No. 11, pp. 47–51, 1989.

Conlon, W. J., Hornburg, C. D., Watson, B. M., and Kiefer C. A., ‘‘Membrane Softening: The Concept and Its Application to Municipal Water Supply,’’ *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 78, No. 2, pp. 157–174, 1990.

Cruver, J. E., ‘‘Reverse Osmosis for Water REuse,’’ in *Complete WateReuse,* American Institute of Chemical Engineers, L. K. Cecil (ed.), pp. 619–630, 1973.

Dykes, G. M., and Conlon W. J., ‘‘Use of Membrane Technology in Florida,’’ *JAWWA*, Vol. 81, No. 11, pp. 43–46, 1989.

Jacangelo, J. G., et al., ‘‘Assessing Hollow Fiber Ultrafiltration for Particulate Removal,’’ *JAWWA*, Vol. 81, No. 11, pp. 68–75, 1989.

Jonsson, G., and Boesen, C. E., ‘‘Polarization Phenomena in Membrane Processes,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications*, Academic Press Inc., pp. 101–130, 1984.

Laine, J. M., et al., Ultrafiltration of Lake Water; Effect of Pretreatment on Partitioning of Organics, THMFP, and FLUX,’’ *JAWWA* 82 (12): 82–87, 1990.

Laine, J. M., Hagstrom. J. P., Clark, M. M., and Mallevialle, J., ‘‘Effects of Ultrafiltration Membrane Composition,’’ *JAWWA*, Vol. 81, No. 11, pp. 61–67, 1989.

Luck, W. A. P., ‘‘Structure of Water and Aqueous Systems,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications,* Academic Press Inc., pp. 21–72, 1984.

Madani, A. A., ‘‘Economics of Desalination for Three Plant Sizes,’’ *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 78, No. 2, pp 187–199, 1989.

Rautenbach, R., and Grøschl, A., ‘‘Separation Potential on Nanofiltration Membranes,’’ in *Desalination: The International Journal on the Science and Technology of Desalting and Water Purification,* Vol. 77, Nos. 1–3, 1989.

Reinhard, M., Goodman, N. L., McCarty, P. L., and Argo D. G., ‘‘Removing Trace Organics by Reverse Osmosis Using Cellulose Acetate and Polyamide Membranes,’’ *JAWWA*, Vol. 78, No. 4, pp. 163–174, 1986.

Rogers, A. N., ‘‘Design and Operation of Desalting Systems Based on Membrane Processes,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications,* Academic Press Inc., pp. 437–477, 1984.

Sammon, D. C., ‘‘Hyperfiltration Membranes, Their Stability and Life,’’ in G. Belfort (ed.), *Synthetic Membrane Processes: Fundamentals and Water Applications,* Academic Press Inc., pp. 73–79, 1984.

Taylor, J. S., Mulford, L. A., Duranceau, S. J., and Barrett W. M., ‘‘Cost and Performance of a Membrane Pilot Plant,’’ *JAWWA*, Vol. 81, No. 11, pp. 52–60, 1989.

CHAPTER 16

Oxidation

INTRODUCTION

Strong oxidants may be applied at one or more locations within a water treatment system to achieve a variety of objectives:

- Control of zebra mussels
- Control of biofouling within the treatment plant
- Inactivation and removal of algae
- Oxidation of iron and manganese
- Reduction of off-tastes and odors
- Color removal
- Improvement of coagulation and filtration
- Reduction of disinfection by-product precursors
- Oxidation of pesticides and other synthetic organic chemicals
- Disinfection
- Prevention of regrowth and maintenance of biological stability in the distribution system

Figure 16–1 illustrates typical application points for oxidants and the process objectives normally associated with these locations.

This chapter reviews the fundamentals of oxidation chemistry, identifies the principal oxidants used in water treatment, and describes the applications of this process. Two major applications of oxidants—iron and manganese removal, and disinfection—are sufficiently important to warrant separate chapters (see Chapters 14 and 19), and are not discussed here. Also, control of water quality in distribution systems is addressed in Chapter 22.

OXIDATION CHEMISTRY

This section provides an abbreviated overview of oxidation chemistry. More thorough treatment of this subject may be found in several references.[1–3]

Oxidation Half-Reactions

A chemical substance is oxidized when it loses electrons to a second substance. This loss of electrons increases the oxidation state (valence) of the substance. Simultaneously, a second compound in the reaction (the oxidizing agent) gains electrons. This second compound is said to be reduced and its valence is decreased. This process is termed an oxidation-reduction, or "redox," reaction. For example, the ferrous ion (FeII) is oxidized when it is changed to ferric ion (FeIII), as in the half-reaction

$$Fe^{++} \rightarrow Fe^{+++} + e^{-} \qquad (16\text{–}1)$$

In contrast, the oxidizing agent chlorine may be reduced to the chloride ion as follows:

$$Cl_2 + 2e^{-} \rightarrow 2Cl^{-} \qquad (16\text{–}2)$$

Reactions such as 16–1 and 16–2 are called half-reactions or partial reactions because they represent only half of the reaction that takes place. These partial reactions are also called half-cell reactions, because they represent the reaction taking place in one half of a suitable electrolytic cell while a complementary reaction is taking place in the other half of the cell at the same time. Combining the two half-reactions and

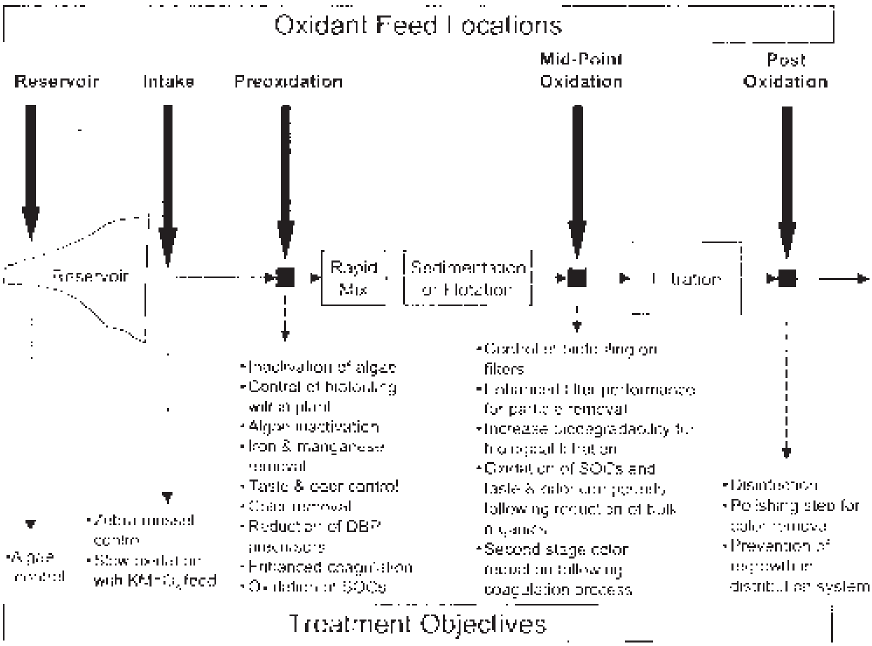


Fig. 16-1. Typical application points for oxidants (Reprinted with permission from *Environmental Science and Technology,* Vol. 21, p. 224. Copyright 1987, American Chemical Society.)

balancing them with respect to electron exchange yields the following complete redox reaction:

$$Cl_2 + 2Fe^{++} \rightarrow 2Cl^- + 2Fe^{+++} \qquad (16\text{–}3)$$

A variety of oxidizing agents are used in water treatment. These are elements or compounds that contain some atom that can gain electrons, and thus increase the positive valence number of some other element or compound. The oxidizing ability of a reagent is generally expressed in terms of the reversible oxidation (or reduction) potential. The standard electrode potential, E^0, is defined as the potential of a half-cell at 25°C in which all the ions are at unit activity and all gases are at partial pressures of one atmosphere. Standard electrode potentials for chemical oxidants used in water treatment are listed in Table 16–1. By convention, the potentials are listed as reduction potentials; that is, the half-cell reactions are formulated as reduction reactions. In this form, the stronger the oxidizing agent, the greater the electrode potential.

Thermodynamic Potential

Thermodynamic potentials may be used to calculate the overall free energy change of a redox process to determine whether the reaction should occur spontaneously. A useful illustration is provided by Glaze[4] in his discussion of permanganate oxidation of oxalic acid, a relatively refractory compound. Reactions 16–4 through 16–6 describe this process:

$$MnO_4^- + 4H^+ + 3e^- \rightarrow MnO_2\downarrow + 2H_2O \qquad E^0 = 1.70 \qquad (16\text{–}4)$$

$$H_2C_2O_4 \rightarrow 2CO_2 + 2H^+ + 2e^- \qquad E^0 = 0.49 \qquad (16\text{–}5)$$

$$2MnO_4^- + 3H_2C_2O_4 + 2H^+ \rightarrow 2MnO_2\downarrow + 4H_2O + 6CO_2 \qquad E^0 = 2.19 \qquad (16\text{–}6)$$

E^0 for the net reaction (2.19 volts) is calculated from the sum of the E^0 values for the

TABLE 16–1. Standard Half-Cell Potentials for Chemical Oxidants Used in Water Treatment

Oxidant	Reduction Half-Reaction	E^0_{red} volts
Ozone	$\frac{1}{2}O_{3(aq)} + H^+ + e^- \rightarrow \frac{1}{2}O_{2(aq)} + \frac{1}{2}H_2O$	2.08
Hydroxyl radical	$OH + H^+ + e^- \rightarrow H_2O$	2.85
Hydrogen peroxide	$\frac{1}{2}H_2O + e^- \rightarrow H_2O$	1.78
Permanganate	$\frac{1}{3}MnO_4^- + \frac{4}{3}H^+ + e^- \rightarrow \frac{1}{3}MnO_{2(s)} + \frac{2}{3}H_2O$	1.68
Chlorine dioxide	$ClO_2 + e^- \rightarrow ClO_2^-$	0.95
Hypochlorous acid	$\frac{1}{2}HOCl + \frac{1}{2}H^+ + e^- \rightarrow \frac{1}{2}Cl^- + \frac{1}{2}H_2O$	1.48
Hypochlorite ion	$\frac{1}{2}OCl^- + H^+ + e^- \rightarrow \frac{1}{2}Cl^- + \frac{1}{2}H_2O$	1.64
Hypobromous acid	$\frac{1}{2}HOBr + \frac{1}{2}H^+ + e^- \rightarrow \frac{1}{2}Br^- + \frac{1}{2}H_2O$	1.33
Monochloramine	$\frac{1}{2}NH_2Cl + H^+ + e^- \rightarrow \frac{1}{2}Cl^- + \frac{1}{2}NH_4^+$	1.40
Dichloramine	$\frac{1}{4}NHCl_2 + \frac{3}{4}H^+ + e^- \rightarrow \frac{1}{2}Cl^- + \frac{1}{4}NH_4^+$	1.34
Oxygen	$\frac{1}{4}O_{2(aq)} + H^+ + e^- \rightarrow \frac{1}{2}H_2O$	1.23

Source: See reference 3. (From American Water Works Association, *Water Quality and Treatment,* 5th ed. Copyright © 2000. Reproduced with permission of The McGraw-Hill Companies.

two half-reactions. This equates to a free energy charge of −303 kilocalories when calculated from $-nFE^0$, where n is the total number of electrons (6) transferred and F is the Faraday constant. Such a high negative free energy indicates that the reaction should occur spontaneously. In real water treatment applications, however, the reaction is too slow for practical use, a condition that exists for most organic oxidation processes.

Furthermore, some reactions that seem feasible from a thermodynamic perspective never occur because intermediate energy requirements along the reaction pathway are too high (see Fig. 16–2). In these cases, energy input must be provided to meet the activation energy requirements.

Equation 16–6 indicates that the oxidation of oxalic acid will proceed to form carbon dioxide. In practice, this usually does not occur and a series of partially oxidized by-products are formed. In theory, all of the oxidants considered in this chapter are capable of oxidizing organic substances to CO_2, but in drinking water applications, conditions are seldom extreme enough to reach this goal and the oxidation process is usually incomplete.[5]

Reaction Kinetics

While the thermodynamic potential indicates whether a reaction will proceed, it provides no indication whether the reaction will occur within a time frame acceptable for water treatment applications. This requires consideration of chemical kinetics.

The basic rate law for a chemical reaction (shown in Equation 16–7) is based on collision theory and is presented in Equation 16–8:

$$aA + bB \rightarrow cC + dD \qquad (16\text{–}7)$$

$$\text{Rate} = k_{fa} \cdot [A]^a\,[B]^b \qquad (16\text{–}8)$$

where the capital letters represent chemical species, the small letters are the stoichi-

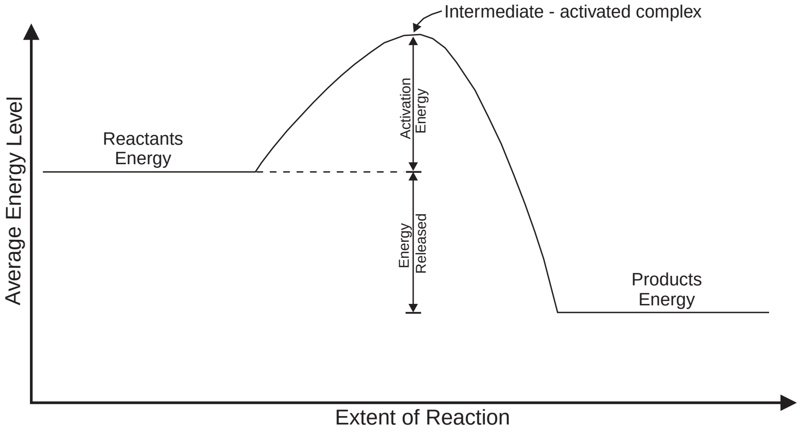


Fig. 16–2. Schematic reaction pathway energy path showing activation energy required to allow reaction to proceed

ometric coefficients, and k_{fa} is the forward rate constant for species A. For real-world systems, reaction rates must be determined experimentally—they are not well predicted by theory. Empirical studies also can determine the effects of changes in environmental variables (reactant concentrations, temperature, catalysts, etc.) on reaction kinetics.

Rate equations may be complex, indicating that the mechanism of a chemical reaction consists of multiple steps. Moreover, simple rate equations do not mean that a mechanism is necessarily simple. For example, decomposition of ozone has an extremely complex mechanism, yet the rate law for the overall decomposition is quite simple.[6]

Catalysts

In many cases, the rate of a chemical reaction can be increased by the presence of catalysts. Catalysts are compounds that increase reaction rates without being formed or consumed by the reaction. They generally provide an alternative pathway to a reaction with a lower activation energy. In some cases, the product of a reaction is a catalyst, in which case the reaction is termed ‘‘autocatalytic’’—it will accelerate as the reaction proceeds.

In oxidation processes for drinking water treatment, important catalytic processes involve the participation of acids and bases. Also important are catalysts that initiate free-radical chain reactions, such as the decomposition of ozone by hydroxide.

Effect of pH and Temperature

Oxidation reaction rates may be affected by pH because the oxidation agents may change form or mechanism under basic or acidic conditions. An example is ozone, which at high pH decomposes to produce the highly reactive hydroxyl radical; consequently, rates of reactions are often observed to change at high pH levels. Similarly, in aqueous solutions, the relative concentrations of different chlorine species shift with changing pH, with each species having different reaction rates.

Temperature also impacts the rate of reaction, as generally the rate constant (k) of a chemical reaction will increase with increasing temperature according to a logarithmic function.

To determine the effect of pH, temperature, and other water quality variables, it is generally necessary to conduct pilot studies that reasonably simulate anticipated full-scale conditions.

OXIDANTS USED IN WATER TREATMENT

Chlorine

Chlorine is the most commonly applied oxidant in water treatment. It is generally used in two forms: as a gaseous element, or as a solid or liquid hypochlorite compound. Information on the nature of alternative chlorine forms and their method of application may be found in Chapter 19.

When chlorine is dissolved in water, it reacts to form hypochlorous and hydrochloric acids:

$$Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^- \tag{16–9}$$

This reaction is essentially complete within a few seconds. The hypochlorous acid ionizes or dissociates instantaneously into hydrogen and hypochlorite ions:

$$HOCl \leftrightarrow H^+ + OCl^- \tag{16–10}$$

These reactions form the basis for the use of chlorine in water treatment applications.

The extent of hypochloride dissociation is controlled by the $[H^+]$ in Equation 16–10. Figure 16–3 illustrates the relative amounts of Cl_2, HOCl and OCl^- as a function of pH. At pH values > 2 and with $[Cl_2]_t < 100$ mg/L, little molecular Cl_2 is present. Between pH 2 and pH 7, hypochlorous acid is the predominant species, and hypochlorite ion is predominant at high pH values.

Hypochlorite salts also ionize in water and yield hypochlorous acid:

$$Ca(OCl)_2 + 2H_2O \rightarrow 2HOCl + Ca^{++} + 2OH^- \tag{16–11}$$

$$NaOCl + H_2O \rightarrow HOCl + Na^+ + OH^- \tag{16–12}$$

Once dissolved, the distribution of hypochlorite and hypochlorous acid is controlled by pH, total chlorine concentration, and temperature in the same manner as that described for gaseous chlorine addition.

Chlorine is a versatile oxidant that historically has been applied for such uses as control of biofouling in basins and filters, taste and odor control, oxidation of reduced

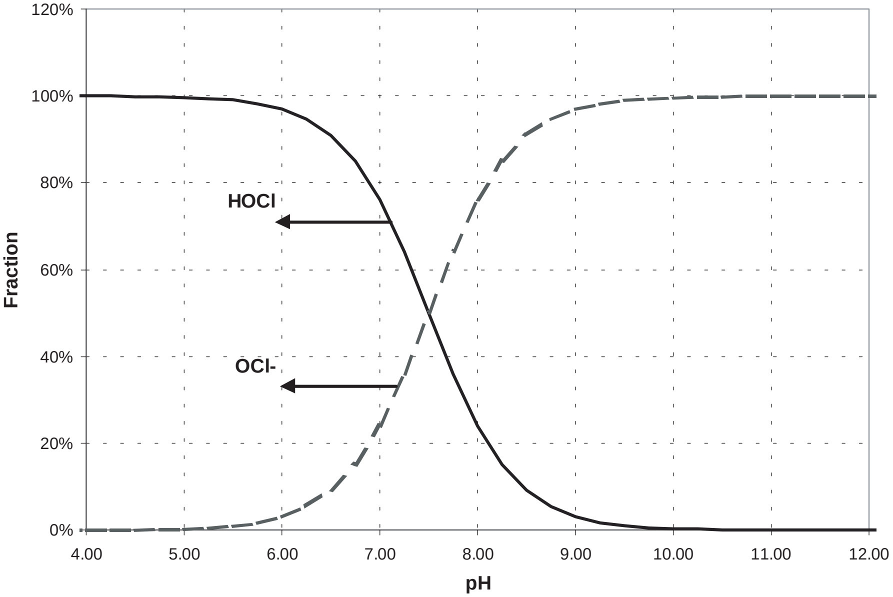


Fig. 16–3. Distribution of chlorine species as a function of pH

metals, color removal, and improvement of flocculation. Unfortunately, chlorine reacts with natural organic material in the water to form harmful by-products. By-products of significant health concern include low-molecular-weight chlorinated and brominated compounds, such as trihalomethanes, haloacetic acids, chloropicrin, and halogenated acetonitriles. Bromine-substituted compounds result from chlorination of waters containing bromide.

Chlorine also reacts with organic material, particularly in surface waters, to form small quantities of many other halogenated organic compounds of demonstrated or potential health concern. The parameter total organic halogen (TOX) has been used to measure the combined levels of these by-products. TOX levels in chlorinated water depend on the level of total organic carbon in the water, but substantial variations occur depending on the source of the water and the pH at which chlorine is applied.[6]

Many halogenated compounds are regulated, often to very low concentrations. These restrictions, along with general public health concerns, have caused water suppliers to reduce the use of chlorine for oxidation applications, and to turn to other oxidants or other treatment techniques to achieve their water quality goals. Nonetheless, chlorine remains a useful tool for well-treated waters or high-quality source waters where chlorine demand is low.

Chloramines

Chloramines are formed by the reaction of ammonia with aqueous chlorine. Compared to the other chemicals described in this chapter, chloramines are weak oxidants. They have been found ineffective for oxidation of manganese and most taste and odor compounds.[7] The principal role of chloramines in water treatment is for disinfection and protection of water quality in distribution systems.

Chlorine Dioxide

Chlorine dioxide is a powerful oxidant that has been used for water treatment applications in the United States since 1944. It is an explosive gas, but is stable in water in the absence of light and elevated temperature. Because of its instability, it is normally generated at the point of use. Several techniques are available for this purpose, which are described in Chapter 19. The chlorine dioxide generation process may produce small quantities of chlorine, with the level of impurity determined by the method chosen and level of control used. The significance of the chlorine impurity depends on the overall water treatment objectives. In waters with high THM formation potential, high-purity chlorine dioxide may be required.

In drinking water, the predominant reaction products are chlorite, chlorate, and chloride. Approximately 50 to 70 percent of the chlorine dioxide will initially react to form chlorite. A fraction of the chlorite will, in turn, be reduced to chloride.[8] Due to health concerns associated with chlorite, EPA established an MCL of 1 mg/L for this constituent as part of the Stage 1 Disinfectants/Disinfection By-Products Rule. This rule also established a Maximum Disinfectant Residual Level of 0.8 mg/L (as ClO_2) for chlorine dioxide. Consequently, use of chlorine dioxide is limited to applications in which demand is low.

Chlorine dioxide reacts with a wide variety of organic and inorganic chemicals in water treatment systems.[9,10] Typical applications include iron and manganese oxidation, color removal, and oxidation of certain organic and sulphurous compounds that cause off-tastes and odors. Several important reactions (or lack thereof) are associated with chlorine dioxide:

- Chlorine dioxide does not react to form THMs. Even though some chlorine impurity results from the generation process, THM formation is lowered compared to use of chlorine as the oxidant.
- Organic compounds are oxidized by chlorine dioxide by a variety of mechanisms, and in some cases chlorine substitution occurs in the products.[11]
- Chlorine dioxide does not react with ammonia, but will react with other amines.
- Chlorine dioxide does not react by breaking carbon–carbon bonds, so mineralization of organics typically does not occur.
- In contrast to ozone, high-purity chlorine dioxide does not oxidize bromide ion into bromate ion, unless photolyzed.[12] Neither does it produce appreciable amounts of aldehydes, ketones, ketoacids, or other DBPs associated with ozonation of organic matter.
- Because chlorine dioxide reacts selectively with phenols, it has become an accepted oxidant for polishing waters where a risk for pollution with phenols has to be accounted for, or where consumers reject waters that contain a taste of chlorophenols.[13]
- The reduction of chlorine dioxide depends heavily on pH and the nature of the reducing agent. At neutral or alkaline pH, chlorine dioxide uses only 20 percent of its oxidizing capacity. At low pH, it uses all of its oxidizing capacity.
- Chlorine dioxide does not react with water as chlorine does, making it easily separable from water with mild aeration, leaving no residuals.

Potassium Permanganate

Potassium permanganate ($KMnO_4$) was first used for drinking water treatment in 1913 (London), and the first U.S. application occurred in 1927 (New York). It has been applied for a variety of objectives, including algae control, color removal, oxidation of iron and manganese, oxidation of phenol, cyanide removal, and taste and odor control.

According to thermodynamic principles, potassium permanganate treatment should be able to oxidize virtually any organic compound and many of the inorganic pollutants of concern.[6] In some waters, permanganate enhances the removal of TOC, possibly by adsorption on manganese dioxide (MnO_2) flocs.[14,15] It is generally not capable of oxidizing THM precursors completely.[16] Potassium permanganate is typically applied as pretreatment, followed by filtration. Addition to finished water is unacceptable because of the pink color of the compound itself or the brown color of oxides. If excessive permanganate is used in the oxidation process, it will pass though the downstream filters and enter the distribution system.

Permanganate may be fed into water as a solid or as a solution prepared on site. Typically, it is applied during flash mixing. Required contact times depend on the treatment objectives. For iron and manganese removal, required contact times are typically 5 to 10 minutes, whereas contact times for taste and odor control may be as long as 1 to 2 hours. To extend contact times, many utilities add $KMnO_4$ to the raw-water pipelines at their sources.

Ozone

Ozone is an unstable gas that is manufactured on site at concentrations of 1 percent to 3 percent for air-supplied generators and 3 percent to 6 percent for oxygen-supplied units. The ozone-rich gas is then dissolved in the liquid stream. A variety of dissolution methods may be used, with the most common being a baffled bubble contactor.

Ozone is unstable in aqueous solution and decomposes spontaneously by a complex mechanism that involves the formation of hydroxyl radicals. As shown in Figure 16–4, ozone reacts in two modes in aqueous solution:[17]

- Direct oxidation of compounds using molecular ozone ($O_{3(aq)}$)
- Oxidation of compounds by hydroxyl radicals produced during the spontaneous decomposition of ozone.

Direct reactions involving molecular ozone are very selective, with ozone reacting rapidly with some species but slowly for others. In contrast, the hydroxyl radical is nonselective, reacting rapidly with a large number of species. The OH radical is the more powerful oxidant.

Decomposition of ozone may be initiated by a number of water constituents, including hydroxide ion (high pH), natural organic matter (NOM), ferrous iron, and others. In fact, the hydroxyl radical reacts rapidly with molecular ozone, contributing to the autocatalytic rate of ozone decomposition. Decomposition may also be initiated by the addition of hydrogen peroxide or irradiation with UV light, as discussed in the section on advanced oxidation processes.

In natural waters, the lifetime of molecular ozone is extended by the presence of bicarbonate and carbonate ions, measured as alkalinity. These constituents scavenge hydroxyl radicals and form carbonate radicals. Therefore, waters with high concentrations of bicarbonate and carbonate alkalinity will maintain molecular ozone residuals for longer periods than will low-alkalinity waters. The scavenging activity of the carbonate species increases at higher pH, partially offsetting the rapid rate of hydroxide-induced ozone decomposition at higher pH values.

Ozonation of natural organic material in water forms by-products, particularly for surface waters. Typical products include aldehydes, aliphatic and aromatic carboxylic acids, quinones, and peroxides. Generally, these products are not considered to cause significant health effects at the concentrations produced.

Ozone can form bromate and brominated by-products when applied to waters with moderate concentrations of bromide. One study found that 7 percent of the raw-water

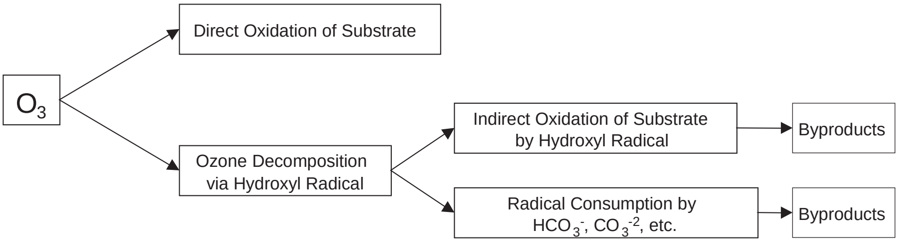


Fig. 16-4. Two modes of ozone oxidation in water

bromide becomes incorporated as TOX following ozonation under typical drinking water conditions.[18]

Ozonation can sharply increase the biodegrability of organic material. To prevent problems associated with high levels of assimilative organic carbon (AOC) in distribution systems (see Chapter 22), ozonation should not be used as a post-oxidation process. Often, ozonation is followed by a biologically active filtration step to reduce the AOC levels, as described in Chapter 12.

Advanced Oxidation Processes

The general term ‘‘advanced oxidation process’’ (AOP) is commonly used to describe processes that use highly reactive radicals as oxidants. These strongly oxidizing, unselective radicals (including hydroxyl radicals) can be produced as intermediate compounds by several processes:

- Ozonation at high pH
- Ozonation with addition of H_2O_2
- Ozonation with irradiation by ultraviolet light
- H_2O_2 with irradiation by ultraviolet light
- Ozonation in the presence of a solid catalyst such as titanium dioxide

The various AOP processes are similar mechanistically, but differ enough to make one or the other more practical, depending on water quality or treatment objectives.

By far, the most widely applied AOP has been the H_2O_2/O_3 process, sometimes referred to as peroxone. When added to water, the hydrogen peroxide dissociates into the hydroperoxide ion (HO_2^-) which then reacts with molecular ozone to produce the hydroxyl radical (OH), the superoxide ion (O_2^-), and molecular oxygen. The H_2O_2 may be applied prior to, at the same time as, or lagging the ozone feed point, depending on the process objectives.

The O_3/UV process is the second-most-applied AOP. In this system, the UV radiation provides the energy to split the hydrogen peroxide into two hydroxyl radicals. The process tends to be slower than the H_2O_2/O_3 reaction mechanism. UV radiation of waters containing molecular ozone also will form H_2O_2, promoting hydroxyl radical formation, as described above.

The other AOPs are less commonly used. Ozonation at high pH may require high ozone doses to offset the impacts of radical scavenging by carbonate ions; consequently, supplementing the ozone with H_2O_2 or UV irradiation may be more economical. The H_2O_2/UV process has generally been found to be less effective than other AOP processes for oxidation of refractory compounds. Use of solid catalysts has been evaluated only at laboratory scale and its economic viability is uncertain.[19,20]

AOPs have been found to be particularly effective for reduction of refractory micropollutants such as pesticides, chlorinated organics, and certain taste and odor compounds. The performance of an AOP is affected by such water quality parameters as pH, total organic carbon, and other chemical species that can act as initiators, promoters, or inhibitors of the chain reaction process. As described earlier, carbonates and bicarbonates are powerful radical inhibitors, and the efficiency of the AOP will decrease quickly with increasing alkalinity. Generally, AOPs work well if the alkalinity is low (<100 mg/L $CaCO_3$). Consequently, in some waters, it may be necessary to adjust the pH or even to use softening treatment before the AOP. High levels of NOM can also increase scavenging of hydroxyl radicals, making application of AOPs less effective and less economical in these conditions. Again, pretreatment to reduce the bulk organic level prior to oxidation may be needed.

AOPs employing UV can be adversely affected by the presence of color or turbidity, which can reduce the UV transmittance. Key design considerations include light intensity and contact time.

APPLICATION OF OXIDANTS IN WATER TREATMENT

The following sections describe applications for oxidants in drinking water treatment. Most cases involve the oxidation of organic or inorganic materials to achieve a desired treatment objective. In some applications, the oxidant is used as a biocide. The two most common uses of oxidants—disinfection and iron/manganese removal—are discussed in Chapters 19 and 14 and are not addressed here.

Control of Zebra Mussels

As described in Chapter 8, proliferation of zebra mussels in U.S. surface waters has created serious problems associated with clogged intakes and structures, and water quality degradation.[21,22] To control zebra mussels, a variety of oxidants have been tested with varying degrees of success. The most extensive applications have involved free chlorine. Investigators report success using continuous chlorination in which a free residual of 0.5–2.0 mg/L is maintained throughout the intake pipe.[23] Typically, the chlorine dosage requirement will vary seasonally for a variety of reasons, with one of the major factors being temperature. Application of chlorine is often convenient for municipal water suppliers who use chlorine or chlorine solutions as disinfectants and have facilities that can be modified or expanded. Either gaseous chlorine or hypochlorite may be used.

Relocating the chlorine injection point to the intake will increase THM production by increasing the exposure of the chlorine to organics and lengthening contact time. Consequently, use of chlorine as a zebra mussel control agent is a workable alternative only if chlorine by-products can be kept to an acceptable level.

To reduce THM formation, other approaches may be used. A number of systems along the Great Lakes are supplementing chlorination with potassium permanganate to hold THM formation in check; including some systems that use $KMnO_4$ exclusively for this purpose.[24] Unfortunately, potassium permanganate control is considerably more expensive than chlorination. Chlorine dioxide is reported to be especially effective in controlling zebra mussels,[25] and at least one investigator has found ozone to be an effective control chemical.[26]

Control of Biofouling Within the Treatment Plant

Oxidants may be used when it is necessary to control biological growth within the treatment plant. Basins and filters that are open to sunlight are particularly susceptible to growth of algae and other microorganisms. Growth may be minimized by applying chemical oxidant in sufficient quantity to maintain a residual throughout the susceptible treatment processes. Alternatively, occasional doses at high concentrations may be used to ‘‘shock’’ the system. Minimization of biofouling in membrane processes is a specific control technique that is addressed in Chapter 15.

Historically, chlorine has been the oxidant of choice for in-plant biofouling control; however this practice may lead to elevated THM concentrations. Other oxidants such as potassium permanganate, chlorine dioxide and ozone are effective biocides, but are rarely used in this application due to the cost and difficulty of sustaining a continuous residual. In fact, the tendency of ozone and chlorine dioxide to make organic material more biodegradable may actually worsen biofouling downstream of their addition points. If this occurs, periodic shock dosing with chlorine or chloramines may provide effective control. Alternatively, the plant may choose to deliberately operate filters in a biological mode to reduce AOC concentrations (see discussion later in this chapter and in Chapter 12).

Control of Algae

Algae and plankton can undergo explosive growth in raw-water supplies, particularly in lakes and reservoirs. These episodes can wreak havoc on treatment plant operation and impair the quality of the finished water. A number of undesired impacts have been identified:[27]

- Poor coagulation and flocculation due to sudden variations in raw-water pH, related to the photosynthetic activity of the algae. Flocculation also becomes difficult because a number of these organisms behave like negatively charged biocolloids.
- Poor settling characteristics connected with poor flocculation and the low apparent density of the algae
- Short filter runs due to clogging in the bottom of filters or blanketing on the surface, depending on the size and morphology of the different species. Filter malfunction may also be due to gas release in the filters from water that is supersaturated in oxygen. This condition is caused by the photosynthetic activity of the algae.
- Various odors and flavors linked to metabolites excreted by plankton in the water
- Breakthough of algae into the finished water, where the increased organic matter may contribute to bacterial regrowth

To control the impacts of algae and plankton, a variety of reservoir management and treatment techniques have been applied with varying success. Within reservoirs, the oxidant potassium permanganate has been used as an alternative algaecide to copper sulfate. Its effective dose ranges from 0.4 to 4.0 mg/L. The higher dosages and cost of permanganate have restricted its application for reservoir algal control.

Within the treatment plant, available control methods include microscreening, filtration without adding a flocculant, flotation, flocculation in a filter, oxidation, or combinations of these processes. Historically, chlorine has been the oxidant used, with the applied dose being the breakpoint dose. Chlorine dioxide is also effective for this purpose. However, because of regulatory restrictions on these oxidants and their by-products, attention in recent years has focused on use of ozone.

Ozone can inactivate some algae or reduce their growth.[27] Some researchers have shown that algae or plankton must first be inactivated to allow increased removal by flotation and filtration.[28] Other studies have demonstrated ozone's ability to improve flocculation and filtration performance, especially during periods of algae bloom.[29,30] For example, in Mont-Valerien, France, studies of preozonation before coagulation, flocculation, and rapid sand filtration found that use of a 1.6 mg/L ozone dose improved the removal of algae cells by about one log.[31] In Antwerp, Belgium, preozonation and prechlorination were compared for a direct filtration plant. Whereas the prechlorination train provided slightly better algae removal, the preozonation train needed lower coagulant dosages, achieved significantly longer filter runs, and produced lower average filter effluent turbidities. When used as a coagulant aid, the contact time should be short (3 minutes or less) and the injected ozone dose lower than that needed to obtain a residual of ozone in the treated water. Too high a dose makes the particles more stable and less efficiently removed.

Ozonation may also be directly combined with a flotation step in a process termed ozoflotation.[27] This process, shown in Figure 16–5, is divided into two compartments. In the ozonation compartment, a large number of ozone gas bubbles is obtained by sweeping porous diffusers with a supplemental water stream. The larger bubbles rise to the surface of the compartment as in a conventional ozonation column. The action of the water sweep entrains the finer bubbles and carries them horizontally into a flotation compartment. Here, the fine bubbles cause flotation of algae and other suspended material. This preozonated, floated water is then collected from the bottom of the flocculation compartment and sent on to the filtration stage. This process has been applied to several European water plants with reported success. In Autun, France, it was found that ozoflotation achieved a significant drop in the filter-clogging index, and that the drop increased in proportion to the ozone dosage.[27] In Hull, England, ozoflotation achieved 70 percent removal of *Anabaena* and 90 percent removal of *Aphanizomenon*.[32]

Metals Removal

As described in Chapter 14, chemical and biological oxidation processes are routinely used to reduce concentrations of iron and manganese. Little information is available

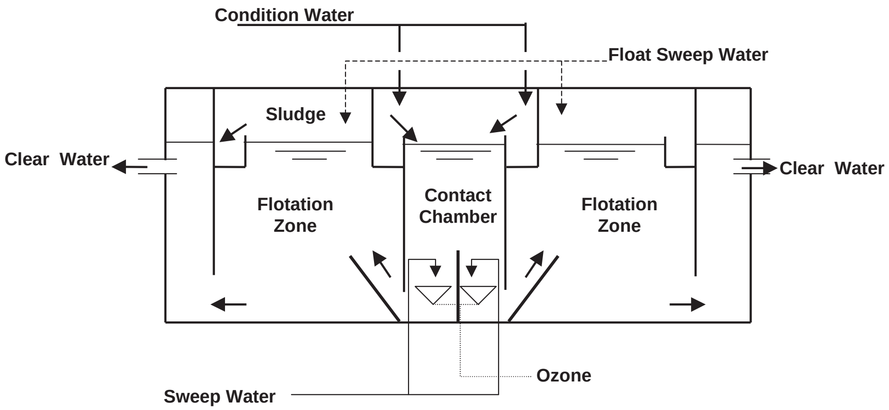


Fig. 16–5. The Ozoflot process flow schematic

regarding the use of oxidation properties to remove other metal species. One study comparing use of chlorine and ozone for removal of iron and manganese found that use of ozone improved removal of zinc and cadmium.[33] The ozone oxidized these metals to the 2 valence state, enhancing their precipitation and subsequent removal by filtration. At an ozone dose of 2 mg/L, removal effectiveness increased with contact time, with nearly complete removal occurring at a contact time of 10 minutes, significantly longer than that needed for iron and manganese.

Taste and Odor Control—Hydrogen Sulfide

Hydrogen sulfide (H_2S) is a common source of taste and odor problems, particularly for groundwater supplies. Information on the source, occurrence, and water quality impacts of hydrogen sulfide is presented in Chapter 2. Common methods of removing hydrogen sulfide from water are air stripping and oxidation. Aeration is discussed in Chapter 9. Adsorption by activated carbon (see Chapter 17) is also effective, but expensive. The effectiveness of water treatment oxidants is presented below.

Chlorine Chlorination may be used to oxidize hydrogen sulfide, particularly in groundwaters of low organic strength where the potential for THM and TOX formation is low. The chemistry for oxidation of hydrogen sulfide by chlorine is complex, proceeding to either elemental sulfur, sulfate or both, as follows.[23]

$$H_2S + Cl_2 \rightarrow 2HCl + S\downarrow \tag{16–13}$$

and

$$H_2S + 4Cl_2 + 4H_2O \rightarrow 8HCl + H_2SO_4 \tag{16–14}$$

Theoretically, the two equations require 2.1 and 8.5 parts of chlorine for each part of H_2S, respectively. Factors that affect these reactions include pH, temperature, and reaction time.

Studies of taste and odor removal with chlorination have generally shown this oxidant to be very effective with low levels of hydrogen sulfide; however, when the hydrogen sulfide levels are too high, the technique becomes very complex.[34] Chlorination must then be preceded by air stripping, or heavy chlorination must be followed by complete dechlorination and subsequent rechlorination.[35]

Ozone Ozone may be used to oxidize hydrogen sulfide. To be effective, the molecular ozone pathway must be used; consequently, the pH of the water should be kept at 7 or less. The reaction equation is:

$$H_2S + 4O_3 \rightarrow H_2SO_4 + 4O_2 \tag{16–15}$$

Therefore, 5.7 parts of ozone are required to oxidize one part of hydrogen sulfide.

Potassium Permanganate Potassium permanganate reacts with hydrogen sulfide as follows:

$$4KMnO_4 + 3H_2S \rightarrow 2K_2SO_4 + 3MnO + MnO_2 + 3H_2O + S\downarrow \quad (16\text{–}16)$$

The mass ratio of potassium permanganate to hydrogen sulfate is 6.2:1. Once again, the process is pH sensitive, and should be conducted in the pH range of 6.5–7.[36]

Removal of Colloidal Sulfur Elemental sulfur exists in water in a virtually insoluble form that may be removed by filtration. Unfortunately, oxidized sulfur may also exist in colloidal form, imparting a milky blue turbidity to the water (the Tyndall effect). In this form, the sulfur may not be removed satisfactorily by ordinary filtration processes and odor problems will remain.[34] Monscvitz and Ainsworth have recommended the use of sodium pyrosulfite or sulfur oxide to treat this condition.[37]

Taste and Odor Control—Organics

Many objectionable tastes and odors resulting from organic material in water can be mitigated by judicious application of a preoxidant. This section provides a brief review of the principal organic sources of off-tastes and odors, and describes the relative effectiveness of oxidants to reduce or eliminate these problems.

Organic Taste and Odor Sources In most cases, off-tastes are caused by organic compounds present in very small quantities, on the order of nanograms per liter. Typically, these compounds are naturally occurring, but taste and odor problems may also result from man-made materials discharged to the source water. A detailed discussion of odor sources may be found in several references.[27,38–40]

The most frequent source of taste and odor in water supply are secretions from blue-green algae and *Actinomycetes*. These odors are particularly prevalent for water supplies that rely on impoundments, and may come on suddenly and strongly as the result of rapid growth of the microorganism. The most common and well-known odor-causing compounds associated with these microorganisms are the alicyclic alcohols geosmin and 2-methylisoborneal (MIB). These compounds impart objectionable odor at very low concentrations (20 ng/L) Consequently, the target concentration for these compounds is typically set at 5 ng/L.

In addition to geosmin and MIB, numerous other compounds are released by different types of algae. These include phenols, aliphatic alcohols, aldehydes, aromatics, ketones, alkanes, esters, thioesters, and sulfides.[27] They may cause odors in their natural form or following oxidation processes as chlorination.

Other natural sources of organic odors include seasonal leaf fall, agricultural drainage and runoff. Decaying vegetation may result in brown-colored, sweet-smelling water. These effects are due to dissolved glucosides such as tannin.[34]

Anthropogenic sources of odor-causing compounds include industrial chemical spills, illegally discharged industrial chemicals, and pollution by sewage. The most offensive industrial discharges typically result from the manufacture of chemicals, dyes, medicinal products, ammonia recovery, wood oil, phenols, cresols, petroleum products, textiles, and paper products.[23] Historically, phenol has received the most attention because of the intensification of off-tastes resulting from chlorination. While pollution control regulations have dramatically reduced the occurrence of odor-causing industrial discharges, water purveyors must remain vigilant to this potential source. This concern is heightened by the fact that roughly 3,000 new chemicals are introduced to the environment each year.

Oxidation Treatment-General Effective treatment techniques for control of organic tastes and odors include oxidation, adsorption, or a combination of these processes. Generally, oxidation followed by filtration is the most cost-effective approach, but particularly tough applications may require a combined oxidation/adsorption process to lower tastes and odors to acceptable levels.

The following sections discuss the potential applicability of alternative oxidants for taste and odor control. Although general conclusions can be drawn about their comparative performance, there is no definitive guide as to which oxidant to apply for each odor type or source. This is due in part to the transient nature of taste and odor episodes and the potential that there may be multiple origins to the odor problem. Also, water quality characteristics and competing oxidant demands may significantly impact the effectiveness or cost of different oxidants. Finally, oxidants have the potential for creating new taste and odor problems. For these reasons, bench or pilot studies should be conducted to establish design criteria and select the appropriate oxidant for specific taste and odor problems.

Chlorine The occasional success of heavy chlorination to control tastes and odors from algal growth or seasonal reservoir conditions has been reported.[39,41,42] However, these applications required prechlorination at sufficient doses to produce a free chlorine residual of 1–5 mg/L. Regulations on chlorinated organics renders this high-dose prechlorination practice impractical for most source waters.

At lower doses, chlorine has been found most effective with organic sulfides, disulfides and mercaptans.[34] At practical dosage rates and contact times, chlorine was found ineffective for oxidation of the algae products 2-isopropyl-3-methoxy-pyrazine [IPMP] and 2-isobutyl-3-methoxypyrazine [IBMP],[43] and could not oxidize geosmin and MIB even at extreme doses and contact times.[43,44]

Chlorination itself is a leading cause of odor complaints in potable water, either by itself or through its by-products. Studies have shown that the odor threshold for chlorine in water at neutral pH is about 0.2 mg/L.[45] The threshold increases to about 0.5 mg/L at pH 9.0. The odor threshold for certain reaction products of chlorine, such as nitrogen trichloride, is much lower.

For odors of certain industrial or algal origin, chlorination often increases odor intensity or character. In this case, superchlorination to a free residual is necessary, followed by partial dechlorination. An example of this is the reaction of phenolic compounds with chlorine. At low chlorine doses, chlorophenol compounds are formed and impart an objectionable medicinal taste to the water. As the chlorine dose increases, the taste-producing intensity of the water increases up to a maximum, after which greater chlorine doses reduce and finally eliminate the chlorophenolic tastes.[34,42]

Chlorine Dioxide Chlorine dioxide has been used effectively to destroy taste-producing phenolic compounds, and will eliminate chlorophenol taste caused by prechlorination. Chlorine dioxide also oxidizes some other off-taste and odor-causing compounds such as mercaptans and disubstituted organic sulfides. Studies in Los Angeles achieved removal efficiencies for IPMP, IBMP, and TCA of greater than 50 percent for a practical range of chlorine dioxide dosages and contact times.[43] Generally, chlorine dioxide has been found ineffective for oxidation of geosmin and MIB.[44]

As described earlier, restrictions on chlorine dioxide by-products in the finished water limit use of this chemical to applications requiring low dosage rates. For surface and groundwaters containing significant organic matter, the overall oxidant demand may preclude use of chlorine dioxide unless upstream treatment steps are used to substantially reduce the organic loading.

Residual chlorine dioxide may impart a taste to the finished water, with adverse odors occurring at concentrations of 0.4–0.5 as ClO_2. Also, application of chlorine dioxide has been found to occasionally cause odors described as kerosene or cat urine. This odd phenomenon has been attributed to the reaction of chlorite, free chlorine, and the volatile organic chemicals released from new carpeting.[46]

Potassium Permanganate Potassium permanganate has been frequently applied for taste and odor control, and is reported to be effective for certain industrial and algal odors.[47] Reported dosages range from 0.5 to 10 mg/L, but are more commonly in the range of 1 to 3 mg/L. Required contact times have ranged from 10 minutes to 1–2 hours, depending on the nature and intensity of the odor source and the oxidant dosage. Oxidation of odor compounds with potassium permanganate is most effective at alkaline pH. At high dosages, potassium permanganate has been able to decrease the concentrations of certain taste- and odor-producing metabolites of actinomycetes,[48] but is not effective for removal of geosmin and MIB.[44] For the metabolites removed; however, the mechanism may not be oxidation. Because of the sorptive properties of manganese dioxide (MnO_2) for certain organic substances and metal ions, it is hypothethized that its formation may play an important role in the extent of organics removal during the $KMnO_4$ treatment process.[49] Work by Lalezary and others indicates that MnO_2 adsorption may be significant for removal of 2,3,6-trichloroanisol [TCA], IPMP, and IBMP, but is not significant for removal of geosmin and MIB.[43]

Ozone Used by itself, ozone can sometimes solve taste and odor problems. Ozone preoxidation has been found effective at reducing levels of unsaturated odor-producing compounds such as 2,4-decadienal,[44] TCA, IPMP, and IBMP.[43]

If the odor-causing compounds are saturated, ozonation may have little effect, unless it occurs in waters that support the hydroxyl radical pathway. Ozonation studies using highly purified water found little oxidation of geosmin and MIB;[43] however, several studies using natural waters have shown ozone to be quite effective at removing these compounds.[44,50–52] The OH radical is a much more powerful oxidant than the O_3 molecule, especially toward aliphatic molecules such as geosmin and MIB.

The ozonation process changes the characteristics of the organic material and the resulting odor and taste. Consequently, when treating waters with significant quantities of organic matter, the impact of ozone on odor levels will be variable and dependent on overall treatment conditions. For some waters, ozonation has been found to increase the odor level through the formation of aldehydes, giving the water a fruity taste.[53]

The effective dosage of ozone ranges from 0.5 to 5 mg/L, with 2 mg/L being a typical value. When concentrations of odor-producing compounds are high, oxidation with ozone alone may not be the most economical process choice. In this case, modification to an advanced oxidation process should be considered. Alternatively, application of GAC filtration downstream of preozonation or intermediate ozonation may provide the best overall treatment solution.

AOPs Extensive studies conducted at the Metropolitan Water District of Southern California compared the ability of oxidation processes to remove odor-causing compounds, including geosmin and MIB.[54] Two AOP processes (H_2O_2/O_3 and O_3/UV) were found to be most effective for geosmin and MIB, achieving observed removal efficiencies of 70 to 99 percent when treating initial concentrations of 100 ng/L for both compounds. Ozone alone was nearly as effective as the AOPs, but at higher dosage rates. To achieve 90 percent removal by ozone alone required a dosage of greater than 4.0 mg/L, as compared with an ozone dose of 2.0 mg/L using H_2O_2/O_3.[54] The effectiveness of ozone alone was apparently due to the fact that constituents in the natural waters tested react with ozone to form highly reactive radicals, probably the OH radical.

Of the AOPs tested, H_2O_2/UV was the least effective. This appears due to the fact that H_2O_2 is a weak absorber of 254-nm light, though it does form OH radicals in the process. High concentrations of H_2O_2 would be needed to absorb enough UV light to produce OH radicals in sufficient concentrations to oxidize the taste and odor compound completely. High concentrations of H_2O_2 would probably be unacceptable for water treatment, unless the residual H_2O_2 could be removed prior to water entering the distribution system.[44]

A Japanese study compared ozone alone with O_3/UV for removal of musty odor sources. Elimination of 100 percent of the geosmin (initial concentration 22 ng/L) and 90 percent of the MIB (initial concentration 130 ng/L) was achieved with an ozone dose of 5 mg/L alone or 4 mg/L if combined with UV.[55]

In Connecticut, the South Central Regional Water Authority experienced a taste and odor problem described as chlorinous at their Lake Gaillard Water Plant. The plant used chlorine as a post-disinfectant; with no preoxidation employed. Analyses of the water did not indicate geosmin, MIB, or other conventional odor-causing compounds. Through testing of alternative preoxidation and post-disinfection processes, investigators found that preoxidation with ozone/peroxide followed by post-chloramination provided odor-free water with a TTHM of 0 μg/L. Use of ozone alone with post-chloramination or use of ozone/peroxide with post-chlorination both failed to significantly improve taste and odor characteristics.[56]

Color Removal

Colored water, like off-taste and odor, is an undesired aesthetic quality that generates consumer complaints. True color implies the presence of substances that absorb light of wavelength 400–800 nm or possibly fluoresce in the range of 200–400 nm. Both organic and inorganic substances may contribute to color, with the relative importance of different components varying widely amongst different source waters. In most cases, color is caused by natural organics (humic substances). These may consist of polyaromatic structures, substituted aromatic structures, polyenes, condensed heterocyclic molecules, or complex ions.[27] In the United States, color is defined in Pt-Co units.

A variety of techniques may be used for color removal. Conventional treatment or direct filtration may achieve color-reduction efficiencies of 70 percent or greater. Carbon adsorption and reverse osmosis also are effective, but likely are cost-prohibitive. For many applications, the process of choice for color removal is preoxidation in combination with chemical coagulation, flocculation, and filtration. Alternatively, to reduce oxidant demand and by-product formation, midpoint oxidation could be practiced following initial removal of bulk organics by coagulation and sedimentation or flotation. Chorine, chlorine dioxide, and ozone have proved to be the best oxidants for color removal.

Chlorine True color removal by chlorine is most effective in the acid pH zone, between pH 4.0 and 6.8. Color removal by chlorination is usually instantaneous, and temperature does not appear to be a factor. There is no rule of thumb for predicting the optimum chlorine dose, and the effectiveness of the application will vary with local conditions.[23]

Given concerns over THM and TOX formation, use of chlorine as a preoxidant for highly colored waters is probably no longer practical. Post-chlorination following initial color removal by coagulation and filtration may be applicable as a polishing step.

Chlorine Dioxide Chlorine dioxide is a strong bleaching agent that has proved effective at removing color from water when used as a preoxidant. Unfortunately, water containing significant color would likely require elevated dosages of chlorine dioxide. Given the restrictions on the by-products for this oxidant, application of chlorine dioxide for color removal appears limited.

Ozone Ozone is the most effective oxidant for color removal. Comparative studies using ozone and AOPs have shown ozone to be the superior oxidant for color removal, demonstrating that the molecular zone pathway is the preferred oxidation mode.[54,57]

To achieve color removal, ozone can be applied as a preoxidant, intermediate oxidant, or both. To prevent increased biodegradability of organic carbon, post-ozonation should be avoided. According to some researchers, ozone doses of 1 to 3 mg of ozone per mg of carbon will lead to nearly complete color removal.[27] Consequently, high ozone dosages may be needed for effective treatment. Also, beyond a certain threshold, some residual color has proven difficult to remove.

A study of colored groundwater in Southern California using ozone as a preoxidant achieved a color reduction from 32–57 Pt-Co to 1–4 Pt-Co, with ozone doses of 4–5 mg/L and residuals of 0.4–0.5 mg/L following 5 min of contact time.[54] A similar study comparing an in-line ozone-dissolving system with a conventional bubble contactor found that influent color levels of 50 Pt-Co could be reduced to the target level of less than 10 Pt-Co using a 4 mg/L ozone dose in the in-line system and a 6 mg/L dose in the conventional system.[58]

In Myrtle Beach, South Carolina, highly colored surface water was preoxidized with ozone. Using a 10 mg/L ozone dose, the color level was reduced from 150–450 Pt-Co to an average of 5 Pt-Co across the treatment plant. However, recoloration was observed 1–2 hours downstream of the preozonation contactor, a condition that can occur if the ozone dose or contact time is insufficient. This problem was addressed by adding an intermediate ozonation step downstream of sedimentation.[59]

Improvement of Coagulation and Filtration

Preoxidation has been reported to improve coagulation and settling properties in certain waters, reducing treatment costs and improving treated water quality for some parameters. While chlorine and chlorine dioxide have been found useful as a coagulant aid, most attention has focused on the use of ozone and ozone-containing AOPs for this application. Preozonation has been used as a coagulant aid for many years in Europe, and has been increasingly used for this purpose in the United States since the mid-1980s.

Reported benefits of preozonation include:

- Reduced coagulant dose to achieve a desired settled water or filtered water quality
- Larger floc size and greater floc-settling velocities in conventional coagulation-settling facilities

- Extended filter run times
- Increased filter application rates

Taken together, these benefits may significantly reduce both the capital and operating costs for treatment systems. In Europe, preozonation has been applied extensively in conventional coagulation-settling filtration systems, whereas United States applications have focused primarily on direct filtration applications, due largely to success at the such large facilities as the Los Angeles Aqueduct Filtration Plant.[60] One full-scale study of diatomaceous earth filtration found that preozonation significantly extended filtration runs while meeting filtered water turbidity goals.[61]

With respect to water quality objectives, the coagulant aid properties of preozonation have largely been associated with attainment of settled- or filtered-water turbidity goals.[62] Practice has shown that preozonation can improve coagulation for some waters but not for others. Also, for waters amenable to this process, seasonal variations can render preozonation temporarily ineffective.

When the goal is reduction of organic matter, findings are more varied and controversial. Some researchers have found that preozonation may improve TOC removal, while others have found the opposite effect.[61,63] Work by Edwards and Benjamin[64,65] suggests that preozonation hinders TOC removal at conventional ratios of coagulant dose to TOC concentrations. This work also indicates that preozonation can increase the residual concentration of metals present after coagulation when alum and iron salts are used as the coagulants.

In cases where preozonation is desired for other treatment objectives, potential detrimental impacts to TOC removal can be offset by increased coagulant dosages, optimization of pH, use of different coagulants (polymeric metal salts, etc.) or application of biologically active filtration downstream of the ozonation step.

The coagulation impacts of ozonation appear complex and varied, and the mechanisms are unclear. Based on theoretical considerations and some empirical observations, a number of explanations have been proposed, including loss of organic coating, increased aluminum complexation, increased calcium complexation, organic polymerization, breakup of iron and manganese complexes, and reactions with algae.[66]

An AWWARF study[62] investigating preozonation as a coagulant aid offered the following observations:

- The impacts of preozonation on subsequent coagulation are highly dependent on the nature of the raw water and its organic and inorganic constituents.
- Iron, organic matter, and algae are all important in determining ozone’s effect on subsequent coagulation.
- Efforts to find a link between high calcium concentrations and greater benefits of preozonation were unsuccessful.
- In lakes and reservoirs of low to intermediate color, seasonal changes in the effect of preozonation may occur due to changes in algae and iron.
- Moderately colored waters containing clay turbidity may have reduced polymer requirements following ozonation. This effect is most likely to occur with waters containing high-molecular-weight organics.
- The presence of high levels of iron or certain species of algae may render a water more susceptible to the coagulating effects of ozone. The impact of preozonation on waters high in iron is dependent on both the iron concentration and the chem-

ical form of the iron. Hydrolyzed iron colloids are susceptible to the coagulating effects of ozone, whereas complexed, reduced iron is not.

- Ozone's effects on treatment of reservoir waters depends on season, algae type, and algae concentration. Ozonation at doses of 3 mg/L or less does not cause extensive lysing of algal cells.
- Preozonation is more likely to improve subsequent coagulation when cationic polyelectrolytes are used as coagulants than when alum is used alone. The beneficial effects of ozone on subsequent coagulation with polymer are more likely to be seen in waters containing high-molecular-weight NOM.

The lack of a theoretical basis for the coagulating effect of ozone has hindered its application for this purpose. It is unlikely that the full benefits of preozonation can be realized before a better understanding of the underlying mechanisms of ozone's interactions with subsequent treatment processes is obtained.[62]

The ozone coagulation process requires ozone dosages between 0.2 and 2.0 mg/L, or about 0.1 to 0.5 mg O_3/mg DOC.[63] An economic evaluation is needed to determine whether the cost of the ozone system is more than offset by coagulant savings, improved filtration performance, and other benefits of preozonation (preoxidation, improved biodegradability, disinfection credit, etc.).

One study by the Metropolitan Water District of Los Angeles compared the relative performance of ozone and the O_3/H_2O_2 AOP with respect to particle destabilization, particle aggregation and filtered water turbidity.[15] Overall, ozone/peroxide was found to be more effective as a coagulant aid than ozone alone.

Reduction of THM and TOX Precursors

A key water quality goal is the control of THM and TOX formation to levels below increasingly stringent regulatory limits. Realizing this goal requires reducing the quantity of organic (humic) material present in the water, reducing the amount of chlorine added, or both. As described in Chapter 3, a variety of management strategies are available for this purpose.

One approach is to replace use of chlorine with ozone, chlorine dioxide, or $KMnO_4$ for the preoxidation applications discussed in this chapter. Chlorination could then take place following reduction of organic mass through conventional treatment or direct filtration.

A second approach is to enhance removal of DBP precursors through use of biological treatment. In this case, ozone or chlorine dioxide may be applied as a preoxidant or midpoint oxidant to increase the biodegradability of the NOM. This step would be followed by a biologically active filter. This process is described in Chapter 12.

Oxidation of Synthetic Organic Chemicals

In many cases, strong oxidants can react with synthetic organic chemicals (SOCs) in drinking water to create products that are nontoxic or more amenable to removal through subsequent biological or adsorption processes. These reactions are highly dependent on the nature of the organic compound, the specific oxidant in question, other constituents in the water, pH, and temperature. When treating SOCs, ozonation and AOPs are the most widely applied oxidation processes, although chlorine dioxide, chlorine, and potassium permanganate may prove useful for specific applications.

The ability of an oxidant to remove a specific micropollutant is largely dependent on the rate constant. For conventional preozonation following the molecular pathway, SOCs with rate constants of greater than 10^5 or 10^6 $M^{-1}s^{-1}$ will be degraded if a measurable ozone residual is maintained. In the case where the rate constant is below 10^2 $M^{-1}s^{-1}$, use of an AOP should be considered; however, some natural waters promote decomposition of ozone to the point where relatively refractory SOCs will degrade. Conversely, water quality conditions that favor scavenging of hydroxyl radicals may impair the ability of AOPs to degrade SOCs. For intermediate values, it is necessary to conduct pilot-plant tests under actual source water conditions to assess performance capabilities.[27] Rate constants for reactions of ozone with a range of inorganic and organic compounds have been developed by Hoigne, Bader and others.[67–69]

In waters with relatively high levels of NOM, the oxidant demand of the bulk organic material must be overcome before the micropollutant can be effectively degraded. In addition, high-NOM waters tend to impair the effectiveness of AOPs by promoting radical scavenging.

The following sections present empirical results for the use of oxidants to degrade pesticides and MTBE—two SOC issues currently of high concern to water purveyors.

Pesticides Removal of pesticides and herbicides is of increasing interest as greater numbers of these toxins become regulated, often to very low concentrations. U.S. standards for regulated pesticides are listed in Chapter 1. European standards limit the concentration of an individual pesticide or herbicide to no more than 100 ng/L and limit the total concentration of all pesticides to 500 ng/L. Particular focus has been placed on the removal of atrazine, one of the most heavily used herbicides in the United States and Europe, and a compound that results in early breakthrough when using activated carbon treatment. However, there is a wide range of pesticides and herbicides employed in the agricultural industry, as discussed in Chapter 3. As controls on one type of compound become more restrictive, use of alternative chemicals increases to meet agricultural production requirements. This proliferation of compounds, each with differing properties that impact treatability, pose a serious challenge to the water purveyor.

A number of studies have been conducted on the ability of oxidation processes to reduce pesticide concentrations in water.[70–77] Two studies that appear representative of this work are described below. Both efforts evaluated the ability of ozone alone and an AOP (H_2O_2/O_3) to treat a range of pesticides under typical water treatment conditions.

Meijers and coworkers[78] evaluated oxidation processes for removal of 23 pesticides spiked into source water from the River Meuse. At neutral pH, using typical ozone doses required for disinfection (O_3/DOC = 0.55 g/g), ozone alone was found to be a poor barrier against pesticides, providing effective degradation of only six compounds: dimethoate, chlortoluron, diuron, isoproturon, metoxuron, and vinclozolin. Increasing the ozone dose (O_3/DOC = 1.0 g/g) resulted in an effective barrier for 50 percent of the pesticides, expanding the list of impacted compounds to include diazinon, parathion-methyl, linuron, methabenzthiazuron, metobromuron, MCPA, and MCPP. With ozone alone, pesticides were degraded more effectively at high pH and temperature, reflecting the impacts of the radical oxidation and improved kinetics.

With advanced oxidation, 21 of the 23 pesticides were effectively degraded, including atrazine, propazine, simazine, chlorfenvinphos, tetrachlorvinphos, 2,4-D, 2,4-DP and 2,4,5-T. Only dicamba and didikegulac were resistant to AOP treatment. Moderate oxidant dosages were used for the AOP process: O_3/DOC = 1.4 g/g (3.0

mg/L) and H_2O_2/O_3 = 0.5 g/g. pH was found to have a minor effect on the degradation of pesticides by the AOP, and higher hydrogen peroxide dosages showed no improvement in degradation for this source water.

Roche and Prados[79] tested ozone alone and H_2O_2/O_3 for the removal of 11 pesticides from water with the following characteristics: TOC = 2.1 mg/L, alkalinity = 240 mg/L as $CaCO_3$, UV_{254} = 0.034/cm, pH = 8.3, and ozone demand = 0.5 mg/L. Ozone alone was effective at reducing only three of the pesticides to target concentrations: terbutryn, isoproturon, and aldicarb. This was achieved at an ozone dose of 1 mg/L and a 10-min contact time. H_2O_2/O_3 using a hydrogen peroxide dose of H_2O_2/O_3 of 0.4 g/g provided effective treatment of eight pesticides, including the following compounds, listed in descending order of reactivity: malathion, aldrin, M. parathion, linuron, and atrazine. For three pesticides—lindane, HCB, and -endosulfan—removals observed were low for both oxidant systems, requiring treatment by activated carbon.

Comparisons of alternative AOP processes (UV/O_3, UV/H_2O_2, H_2O_2/O_3, O_3 at high pH) for control of selected pesticides have produced varying results, indicating the importance of source water characteristics.

Oxidation of pesticides may be incomplete, producing intermediate degradation products. For example, incomplete oxidation of atrazine may yield substantial amounts of deethylatrazine and deisopropylatrazine. Such by-products are potential candidates for future regulatory action and must be taken into account when evaluating treatment alternatives.

MTBE Methyl tertiary butyl ether (MTBE) is the most common oxygenated fuel additive used in reformulated gasoline. It has been found in an increasing number of groundwater supplies and is difficult to remove in conventional treatment systems. Pilot-scale investigations have demonstrated that both ozone alone and H_2O_2/O_3 can remove MTBE from California groundwater sources.[80] H_2O_2/O_3 provided greater reductions, achieving 78 percent removal of MTBE (23 g/L influent concentration) with a 4-mg/L ozone dose and 1.3-mg/L of hydrogen peroxide; however, unacceptable levels of bromate were produced due to high concentrations of bromide in the raw water.

SELECTION OF AN OXIDATION PROCESS

Selecting the appropriate type(s) of oxidants and their application point(s) within the water treatment process requires consideration of a wide range of factors, including:

- *Oxidation treatment objectives*—particularly if multiple objectives are involved. Different oxidants exhibit varying versatility, and some objectives may require conflicting oxidant selection or process selection. For instance, treatment of a reservoir water containing manganese and refractive pesticides favors the molecular ozone pathway for one objective and the hydroxyl radical pathway for the other. Through careful design, both mechanisms may be used through sequential addition of ozone and hydrogen peroxide or UV, or through use of preozonation followed by midpoint application of an AOP.
- *Source water quality.* As previously described, water quality parameters such pH, alkalinity, ferrous iron, and NOM concentration may promote or inhibit the molecular

TABLE 16-2. Comparative Effectiveness of Oxidants for Water Treatment Applications

Treatment Challenge	Chlorine	Chloramines	Chlorine Dioxide	Potassium Permanganate	Ozone	Advanced Oxidation Process
Zebra mussels	E	N	E	E	M*	U
Biofouling	E	M	M†	M	N‡	N‡
Algae	M	N	M	M	E	M*
Iron	E	N	E	E	E	E
Manganese	M	N	E	E	E	M*,§
Hydrogen sulfide	E	N	M	M	E§	N1
Organic taste and odor	M//	N	M//	M//	M#	E
Color	E	N	E	M	E	M*,§
Flocculation aid	M	N	U	M	E**	E**
THM/TOX control	N	N	E††	M	E‡‡	E‡‡
Synthetic organics	M§§	N	M§§	M§§	M§§	E

Adapted from reference 6.

E = effective; M = moderately effective; N = not effective; U = unknown.

*Limited data.

†May worsen biofouling by increasing biodegradability of organic material

‡May worsen biofouling unless two-stage ozonation or biological filtration used.

§Molecular ozone pathway is more effective.

//Generally ineffective for geosmin and MIB.

#Ineffective for geosmin and MIB unless conditions promote radical pathway.

**Effective for some waters, but not others.

††Assumes low chlorine impurities.

‡‡May increase THM/TOX for some waters, particularly at low oxidant dose.

§§Effectiveness highly dependent on the specific compound.

TABLE 16–3. Advantages and Disadvantages of Oxidants Used in Water Treatment

Oxidant	Advantages	Disadvantages
Chlorine	• Strong oxidant • Economical • Versatile • Simple operation • Provides persistent residual that is easy to measure and control • Extensive track record of successful use	• Reacts with naturally occurring organics and inorgancs in water to form undesirable DBPs • Hazards associated with using chlorine gas • May cause taste and odor problems
Chloramines	• No THM formation • Relatively simple operation • Provides persistent residual • Extensive track record of successful use	• Weak oxidant • May lead to nitrification problems in distribution system • Produces TOX
Chlorine Dioxide	• Strong oxidant • Versatile • Does not produce halogenated DBPs (unless chlorine present as impurity) • Does not react with ammonia • Residual may be stripped by aeration	• Relatively high cost • Must be generated on-site • Produces chlorite, a regulated DBP • Difficult to maintain a residual • May produce undesirable odors • May produce biodegradable organic material that must be controlled
Potassium Permanganate	• Simple operation • Does not produce halogenated DBPs	• Only moderately strong oxidant • High chemical cost • Slow reaction time for some applications • Can lead to pink water if dosage not carefully controlled • Produces manganese dioxide which must be removed • Does not provide a persistent residual

Ozone	• Very strong oxidant • Does not produce halogenated DBPs except in bromide-rich waters	• Relatively high cost • Must be generated on-site • Relatively complex operation • Produces bromate in bromide-rich waters • Does not provide a persistent residual • Produces biodegradable organic material that must be controlled
Advanced Oxidation Processes	• Very strong oxidant • Does not produce halogenated DBPs except in bromide-rich waters	• Relatively high cost • Relatively complex operation • Does not provide a persistent residual • Produces biodegradable organic material that must be controlled • Consumption of hydroxyl radicals by alkalinity or NOM may require high oxidant dosages • For AOPs using UV, effectiveness is impaired by color or turbidity

Source: Adapted from Reference 6.

or radical pathway for ozonation and AOPs. Presence of color and turbidity will impair the effectiveness of AOPs using UV. Waters with high NOM levels impart high oxidant demands, which may preclude use of chlorine and chlorine dioxide due to limitations on by-products formed by these oxidants. Changing pH may alter the form and reaction rate of the oxidant. Many of these water quality impacts may be mitigated through pretreatment or chemical adjustment of the source water, but the cost of these added measures may make such approaches unattractive.

- *Impacts of by-product formation.* The potential for the oxidation process to ultimately result in unacceptably high levels of regulated by-products such as THMs, TOX, and bromate must be carefully considered. This evaluation must consider the synergistic effect of the oxidation process and subsequent disinfection steps. In some cases, preoxidation may reduce formation of halogenated organics, while in others it may increase formation. Also, for ozone and chlorine dioxide, oxidation will increase AOC levels in the treated water unless this material is removed through subsequent processes such as GAC, conventional filtration, or biologically active filtration.
- *Cost.* When considering costs, the total system cost for implementing an oxidation must be considered, including necessary pretreatment or post-treatment steps to ensure the effectiveness and mitigate the impacts of a particular oxidation method.
- *Compatibility with operator skills.* The different oxidation processes vary in terms of operational and maintenance complexity, ranging from simple chemical feed systems to mechanically complex systems such as O_3/UV. The chemicals employed also vary in terms of hazard potential and safety requirements. The process choice must be consistent with both the skill level and operational philosophy of the water purveyor.

For some applications, such as iron and manganese removal from groundwater, oxidant selection and plant design can be based on proven removal mechanisms and extensive operational experience. In many other applications, such as removal of refractory taste and odor compounds or pesticides, pilot testing is needed to fully assess treatment options and to optimize design criteria for full-scale application.

Table 16–2 summarizes the relative effectiveness of the different oxidants for the various oxidation goals described in this chapter. Their comparative effectiveness for disinfection and iron/manganese removal is described in Chapters 19 and 14, respectively. It must be noted that Table 16–2 provides a generalized comparison that does not hold for all source waters or treatment conditions.

Table 16–3 provides a brief summary of the comparative advantages and disadvantages of the oxidants with respect to operational considerations, regulatory restrictions, and other issues.

REFERENCES

1. Stumm, W., and Morgan, J. J., *Aquatic Chemistry,* Wiley-Interscience, New York, 1970.
2. Snoeyink, V. L., and Jenkins, D., *Water Chemistry,* John Wiley & Sons, New York, 1980.
3. AWWA, “Chemical Oxidation,” 5th ed., *Water Quality and Treatment,* American Water Works Association, McGraw-Hill, 1999.
4. Glaze, W. H., “Oxidation of Organic Substances in Drinking Water,” *Control of Organic Substances in Drinking Water* (B. Berger, ed.), Noyes Data Corporation, Park Ridge, NJ, 1987.

5. Glaze, W. H., ‘‘Drinking Water Treatment with Ozone,’’ *Environ. Sci. Tech.,* Vol. 21, p. 224, 1987.
6. AWWA, ‘‘Chemical Oxidation,’’ 4th ed., *Water Quality and Treatment,* American Water Works Association, McGraw-Hill, 1990.
7. Krasner, S. W., Barrett, S. E., Dale, M. E., and Hwang, C. J., ‘‘Free Chlorine Versus Monochloramine in Controlling Off-Tastes and Odors in Drinking Water,’’ AWWA Annual Conference, Denver, June 1986.
8. Aieta, E. M., Berg, J. D., ‘‘A Review of Chlorine Dioxide in Drinking Water Treatment,’’ *JAWWA,* 78(6), June 1986.
9. Masschelein, W. J., *Chlorine Dioxide,* Ann Arbor Science, Ann Arbor, MI, 1979.
10. Gordon, G., Kieffer, R. G., and Rosenblatt, D. H., ‘‘The Chemistry of Chlorine Dioxide,’’ in *Progress in Inorganic Chemistry,* Vol. XV (S. J. Lippard, ed.), John Wiley & Sons, New York, 1972.
11. Stephens, A. A., ‘‘Reaction Products of Chlorine Dioxide,’’ *Environmental Health Perspective,* Vol. 46, 1982.
12. Gordon, G., Emmert, G. L., and Bubnis, B., ‘‘Bromate Ion Formation in Water When Chlorine Dioxide Is Photolyzed in the Presence of Bromide Ion,’’ AWWA Water Quality Technology Conference, New Orleans, November 1995.
13. Hoigne, J., and Bader, H., ‘‘Kinetics of Reactions of Chlorine Dioxide (OClO) in Water—I. Rate Constants for Inorganic and Organic Compounds,’’ *Wat. Res.* 28(1), pp. 45–55, 1994.
14. Carlson, M. A., Hoehn, R. C., Knocke, W. R., and Hair, D. H., ‘‘Experiences with the Use of Chlorine Dioxide and Potassium Permanganate as Preoxidants for Thihalomethane and Manganese Control,’’ AWWA Annual Conference, Denver, June 1986.
15. Paode, R. D., Chandrakanth, M. S., Amy, G. L., Gramith, J. T., and Ferguson, D. W., ‘‘Ozone Versus Ozone/Peroxide Induced Particle Destabilization and Aggregation: A Pilot Study,’’ *Ozone Sci. & Eng.,* Vol. 17, pp. 25–51, 1995.
16. Singer, P. C., Borchardt, J. H., and Colthurst, J. M., ‘‘The Effects of Permanganate Pretreatment on Trihalomethane Formation in Drinking Water,’’ *JAWWA* 72(10), October 1980.
17. Hoigne, J., and Bader, H., ‘‘The Role of Hydroxyl Radical Reactions in Ozonation Processes in Aqueous Solutions,’’ *Wat. Res.* Vol. 10, pp. 377–386, 1976.
18. Song, R., Westerhoff, P., Minear, R., and Amy, G., ‘‘Bromate Minimization During Ozonation,’’ *JAWWA* 89(6), 1997.
19. Allemane, H., Delouane, B., Paillard, H., and Legube, B., ‘‘Comparative Efficiency of Three Systems (O_3, O_3/H_2O_2, and O_3/TiO_2) for the Oxidation of Natural Organic Matter,’’ *Ozone Sci. & Eng.,* Vol. 15, pp. 419–432, 1993.
20. Pines, D., Humayan, R., Reckhow, D., and Spangenberg, C., ‘‘A Catalytic Oxidation Process for Removal of Ozone By-Products,’’ AWWA Water Quality Technology Conference, 1994.
21. O’Neill, C. R., Jr., and MacNeill, D. B., ‘‘Dreissens Polymorpha: An Unwelcome New Great Lakes Invader,’’ Control Resources, New York State Sea Grant, November 1989, revised February 1990.
22. Lange, C., ‘‘The Impact of the Freshwater Macrofouling Zebra Mussel (Dreissens Polymorpha) on Drinking Water Supplies,’’ AWWA Water Quality Technology Conference, 1992.
23. White, G. C., *Handbook of Chlorination and Alternative Disinfectants,* 4th ed., John Wiley & Sons, 1999.
24. San Giacomo, R., and Randell, N. G., ‘‘Invasion of the Zebra Mussels,’’ *Civil Engineering,* pp. 56–58, May 1993.
25. Griese, M. H., and Rosenblatt, A. A., ‘‘Chlorine Dioxide’s New Phase,’’ AWWA Annual Conference and Exposition, Toronto, Ontario, June 1996.
26. LePage, W. L., and Bollyky, L. J., ‘‘A Proposed Treatment for Dreissena Polymorpha,’’ Proc. of Spring Conference of International Ozone Association Pan American Committee, Shreveport, LA, 1990.

27. AWWA Research Foundation and Compagnie Generale des Eaux, *Ozone in Water Treatment-Application and Engineering,* Lewis Publishers, Chelsea, MI, 1991.
28. Bernhardt, H., and Lusse, B., ''Elimination of Zooplankton by Flocculation and Filtration,'' *Aqua,* 38:1:23, 1989.
29. Richard, Y., et al., ''Influence of Preozonation on Clarification by Flotation for Drinking Water,'' *Ozone Sci. & Eng.,* Vol. 5, no. 3, 1983.
30. Richard, Y., ''Use of Ozone and Flotation for the Treatment of a Reservoir Water: The Dinan Case History,'' *Ozone Sci. & Eng.,* Vol. 15, pp. 465–480, 1993.
31. Richard, Y., and Dalga, N., ''Preozonation Related to Algae Removal—A Case History: The Plant of Mont-Valerien,'' *Ozone Sci. & Eng.,* Vol. 15, pp. 445–456, 1993.
32. Wilson, D., Lewis, J., Nogueria, F., Faivre, M., and Boisdon, V., ''The Use of Ozoflotation for the Removal of Algae and Pesticides from a Stored Lowland Water,'' *Ozone Sci. & Eng.,* Vol. 15, pp. 481–496, 1993.
33. Nieminski, E., and Evans, D., ''Pilot Testing of Trace Metals Removal with Ozone at Snowbird Ski Resort,'' *Ozone Sci. & Eng.,* Vol. 17, pp. 297–309, 1995.
34. Montgomery, J. M., Consulting Engineers, *Water Treatment: Principles and Design,* John Wiley & Sons, New York, 1985.
35. Monscvitz, J. T., and Ainsworth, L. D., ''Hydrogen Polysulfide in Water Systems,'' American Chemical Society Meeting, Division of Water, Air and Water Chemistry, Minneapolis, April, 1969.
36. Kawamura, S., *Integrated Design of Water Treatment Facilities,* John Wiley & Sons, New York, 1991.
37. Monscvitz, J. T., and Ainsworth, L. D., ''Treatment for Hydrogen Polysulfide,'' *JAWWA,* 66(9), 1974.
38. Palmer, C. M., *Algae in Water Supplies,* U.S. Dept. HEW, Water Supply, 1962.
39. Sigworth, E. A., ''Control of Odor and Taste in Water Supplies,'' *JAWWA,* Vol. 49(2), December, 1957.
40. Faust, S. D., and Aly, O. M., *Chemistry of Water Treatment,* 2nd ed., Ann Arbor Press, Chelsea, MI, 1998.
41. Harlock, R., and Dowlin, R., ''Use of Chlorine for Control of Odors Caused by Algae,'' *JAWWA,* 50(1), January 1958.
42. Riddick, T. M., ''Controlling Taste, Odor and Color with Free Residual Chlorination,'' *JAWWA,* 43(7), July 1951.
43. Lalezary, S., Pirbazari, M., and McGuire, M. J., ''Oxidation of Five Earthy-Musty Taste and Odor Compounds,'' *JAWWA* 78(3), March 1986.
44. Glaze, W. H., et al., ''Evaluating Oxidants for the Removal of Model Taste and Odor Compounds From a Municipal Water Supply,'' *JAWWA* 82(5), May 1990.
45. Bryan, P., Kuzcinski, L., Sawyer, M., and Feng, R., ''Taste Threshold of Halogens in Water,'' *JAWWA* 65:373, 1973.
46. Hoehn, R. C., Dietrich, A. M., Farmer, W. S., Orr, M. P., Ramon, R. G., Aieta, E. M., Wood, D. W., and Gordon, G., ''Household Odors Associated with the Use of Chlorine Dioxide,'' *JAWWA* 82(4), 1990.
47. Swanger, G., ''Use of Potassium Permanganate at Bowling Green, Ohio,'' *Water and Wastewater Engineering,* Vol. 46, October 1969.
48. Cherry, A. K., ''Use of Potassium Permanganate in Water Treatment,'' *JAWWA,* Vol. 54, p. 417, 1962.
49. Morgan, J. J., and Stumm, W., ''Colloid Chemical Properties of Manganese Dioxide,'' *Jour. Colloid. Sci.,* Vol. 4, 1964.
50. Terashima, K., ''Reduction of Musty Odor Substances in Drinking Water—A Pilot Plant Study,'' *Wtr. Sci. Tech.,* 20:8/9:275.

51. Daniel, P. A., and Meyerhoffer, P. F., ‘‘Ozonation of Taste and Odor Causing Compounds,’’ *Proc. 9th Ozone World Conf.* (L. J. Bollyky, ed.), pp. 239–242, IOA, New York, 1989.
52. Duguet, J. P., et al., ‘‘Geosmin and MIB Removal Using Ozone or Ozone/Hydrogen Peroxide Coupling,’’ *Proc. 9th Ozone World Conf.* (L. J. Bollyky, ed.), pp. 709–719, IOA, New York, 1989.
53. Anselme, C., et al., ‘‘Removal of Tastes and Odors by Ozonation,’’ *Proc. of the 8th Ozone World Congress,* pp. C102–C127, IOA, Zurich, 1987.
54. Gramith, J. T., Ferguson, D., McGuire, M. J., and Tate, C. H., ‘‘Overview of Metropolitan’s Ozone-Peroxide Demonstration Project,’’ AWWA Annual Conference, Cincinnati, Ohio, 1990.
55. Kato, M., et al., ‘‘The Further Development in Ozone Application Technology in Potable Water: Ultraviolet or Ultra-Sonic Assisted Ozonation,’’ *Wtr. Supply* 6:349, 1983.
56. Morico, K., Grasso, D., Hess, A., and Dunn, H., ‘‘Impact of Alternative Pre-Oxidation and Post-Disinfection Processes on Taste and Odor Characteristics,’’ ASCE Annual Conference on Environmental Engineering, 1990.
57. Tan, L., Amy, G., Rigby, M., Renna, J., and Kemp, K., ‘‘Ozonation of Colored Groundwater Pilot-Scale and Full-Scale Experiences,’’ *Ozone Sci. & Eng.,* Volume. 13, pp. 109–125, 1991.
58. Bellamy, W., Awad, J., Wei, J., and Gramith, J., ‘‘On-Line Ozone Dissolution Demonstration-Scale Evaluation,’’ *Proceedings IOA Pan American Committee Conference,* Shreveport, LA, 1990.
59. Dimitiou, M. A., and Ivanco, E., ‘‘The Myrtle Beach Ozone System and Water Treatment Plant,’’ in *Ozone in Water Treatment,* Vol. 1 (L. J. Bollyky, ed.), *Proceedings, Ninth Ozone World Congress,* New York, 1989. L. J. Bollyky (Ed.) Intl. Ozone Assoc., Zurich, 1989.
60. Prendiville, P. W., ‘‘Ozonation at the 900 cfs Los Angeles Water Purification Plant,’’ *Ozone Sci. & Eng.,* Vol. 8, p. 77, 1986.
61. Mastronardi, R. A., Fulton, G. P., Farrar, M., and Collins, A. G., ‘‘Preozonation to Improve and Optimize Diatomaceous Earth Filtration,’’ *Ozone Sci. & Eng.,* Vol. 15, pp. 131–147, 1993.
62. AWWA Research Foundation, *Ozone as an Aid to Coagulation and Filtration,* American Water Works Association, Denver, CO, 1993.
63. Jekel, M. R., ‘‘Flocculation Effects of Ozone,’’ *Ozone Sci. & Eng.,* Vol. 16, pp. 55–66, 1994.
64. Edwards, M., and Benjamin, M. M., ‘‘Transformation of NOM by Ozone and Its Effect on Iron and Aluminum Solubility,’’ *JAWWA,* 74(6), June 1992.
65. Edwards, M., and Benjamin, M. M., ‘‘Effect of Preozonation on Coagulant–NOM Interactions,’’ *JAWWA* 74(8), August 1992.
66. Reckhow, D. A., Singer, P. C., and Trussell, R. R., AWWA Seminar Proc.: *Ozonation, Recent Advances and Research Needs,* AWWA Seminar Proc., American Water Works Association, Denver, CO, 1986.
67. Hoigne, J., and Bader, H., ‘‘Rate Constants of Reactions of Ozone with Organic and Inorganic Compounds in Water—I. Non-Dissociating Organic Compounds,’’ *Wat. Res.* 17(2), p. 173, 1983.
68. Hoigne, J., and Bader, H., ‘‘Rate Constants of Reactions of Ozone with Organic and Inorganic Compounds in Water—II. Dissociating Organic Compounds,’’ *Wat. Res.* 17(2), p. 185, 1983.
69. Hoigne, J., Bader, H., Haag, W. R., and Staehelin, J., ‘‘Rate Constants of Reactions of Ozone with Organic and Inorganic Compounds in Water—III. Inorganic Compounds and Radicals,’’ *Wat. Res.,* 19(8), p. 993, 1985.
70. Beltran, F. J., Garcia-Araya, J. F., and Acedo, B., ‘‘Advanced Oxidation of Atrazine in Water—II. Ozonation Combined with Ultraviolet Radiation,’’ *Wat. Res.,* 28(10), pp. 2165–2174, 1994.
71. Buss, D. F., Winnen, S. A., and Ales, S. M., ‘‘Treatment of Herbicide Contaminated Groundwater to Non-Detectable Concentrations Applying U.V. Oxidation,’’ ASCE Environmental Engineering Conference, 1999.

72. Karimi, A. A., Redman, J. A., Glaze, W. H., and Stolarik, G. F., ‘‘Evaluating an AOP for TCE and PCE Removal,’’ *JAWWA* 89(8), August 1997.
73. Lambert, S. D., Graham, N. J. D., and Croll, B. T., ‘‘Degradation of Selected Herbicides in a Lowland Surface Water by Ozone and Ozone-Hydrogen Peroxide,’’ *Ozone Sci. & Eng.,* Vol. 18, pp. 251–269, 1996.
74. Laplanche, A., Orta De Velasquez, M. T., Boisdon, V., Martin, N., and Martin, G., ‘‘Modelisation of Micropollutant Removal in Drinking Water by Ozonation or Advanced Oxidation Processes (O3/H2O2),’’ *Ozone Sci. & Eng.,* Vol. 17, pp. 97–117, 1995.
75. Masten, S. J., and Hoigne, J., ‘‘Comparison of Ozone and Hydroxyl Radical-Induced Oxidation of Chlorinated Hydrocarbons in Water,’’ *Ozone Sci. & Eng.,* Vol. 14, pp., 197–214, 1992.
76. Prado, J, and Esplugas, S., ‘‘Comparison of Different Advanced Oxidation Processes Inv-Volving Ozone to Eliminate Atrazine,’’ *Ozone Sci. & Eng.,* Vol. 21, pp. 39–52, 1999.
77. Prados, M., Paillard, H., and Roche, P., ‘‘Hydroxyl Radical Processes for the Removal of Triazine from Natural Water,’’ *Ozone Sci. & Eng.,* Vol. 17, pp. 183–194, 1995.
78. Meijers, R. T., Oderwald-Muller, E. J., Nuhn, P. A. N. M., and Kruithof, J. C., ‘‘Degradation of Pesticides by Ozonation and Advanced Oxidation,’’ *Ozone Sci. & Eng.,* Vol. 17, pp. 673–686, 1995.
79. Roche, P., and Prados, M., ‘‘Removal of Pesticides by Use of Ozone or Hydrogen Peroxide/Ozone,’’ *Ozone Sci. & Eng.,* Vol. 17, pp. 657–672, 1995.
80. Liang, S., et al., ‘‘Oxidation of MTBE by Ozone and Peroxone Processes,’’ *JAWWA,* 91(6), June 1999.

CHAPTER 17

Activated Carbon Treatment

Activated carbon adsorption is the most effective and reliable water treatment process available for the removal of a broad spectrum of organic substances dissolved in water. For many refractory organic compounds (i.e., PCBs, insecticides, herbicides), it is the only effective treatment alternative. Activated carbon can be used to adsorb the specific organic compounds that contribute to taste and odor (i.e., Geosmin and MIB) as well as the broad spectrum of natural organic matter (NOM) that causes color and serves as precursor material for disinfection by-product formation.

PAST PRACTICES

In the past, the principal reason for the use of activated carbon treatment was to control taste- and odor-causing organics. This step was taken mainly for aesthetic rather than public health purposes. To control tastes and odors in water, the carbon dosage requirements are low and the necessary contact times short. This set of circumstances permitted the application of powdered activated carbon (PAC) rather than granular activated carbon (GAC). The use of PAC was advantageous because in most cases it could be used with no changes or additions to existing treatment facilities other than installation of the powdered carbon storage and feed equipment. Contact time was provided in existing settling basins, and the spent carbon was removed in existing rapid sand filters and disposed of along with the settling basin sludges.

RECENT DEVELOPMENTS

Since the late 1970s, there has been a tremendous proliferation in the number, variety, and quantity of complex organic chemicals used for agricultural, industrial, and domestic purposes. This trend is continuing, with no end in sight. Many of these substances eventually find their way into sources of public water supply. Even in very low concentrations, many of these compounds have toxic, carcinogenic, mutagenic, or teratogenic properties that may produce long-term insidious health effects in water consumers. As a result, when Congress passed the 1986 Safe Drinking Water Act Amendments, the legislation contained a provision requiring the U.S. Environmental Protection Agency (USEPA) to regulate 25 new compounds every 3 years—with no apparent end. Even though this requirement was removed in the 1996 Safe Drinking

Water Act reauthorization, it clearly signals the intent of Congress and USEPA to regulate the increasing number of organic compounds in public water supplies.

Development of this extensive, bewildering array of new synthetic organic chemicals has been paralleled by the development of sophisticated and extremely sensitive equipment that can detect and measure very minute concentrations of organics in water. Monitoring capability has progressed rapidly from parts per million to parts per billion, parts per trillion, and beyond. Some chemists predict that the day may not be far away when detecting individual molecules of substances in a water sample will be possible.

One surprising result of this newfound ability to detect trace quantities of organics in water is that examination of well water supply sources has revealed the universal presence of naturally occurring organics, such as aldehydes, ketones, terpenes, humic compounds, and other substances. Some of these materials, as well as some organic pollutants in well water, may react with chlorine and other disinfectants to form trihalomethanes and other undesirable organic by-products in drinking water. The disinfection by-products are the focus of the recent D/DBP Rule and the Information Collection Rule (ICR). The ultimate objective of these rules is to regulate the quantity of DBPs in public water supplies. The ICR identifies activated carbon and reverse osmosis as the two currently acceptable treatment technologies available to control DBPs and DBP precursors. (Chapter 3 discusses DBPs and other organic compounds.)

CURRENT TREATMENT PRACTICES

As already discussed, both natural and synthetic organics in water are for the most part adsorbable on activated carbon. However, dosage and contact times required to remove these organics to trace levels are much greater than for taste and odor removal. These differences in carbon treatment process requirements and removal efficiencies dictate the use of granular rather than powdered activated carbon. When granular activated carbon is used for taste and odor control, it is possible to add a shallow bed (1 ft [0.3 m] or less) of GAC on top of an existing sand filter, or to substitute a properly sized and graded bed (24 to 36 in. [0.6 to 1 m]) of GAC in lieu of the fine media in a rapid sand filter, with satisfactory results. However, experience has demonstrated that such shallow beds of GAC generally are not suitable for removal of natural or synthetic organics. Deeper beds and longer contact times (typically 10 to 45 minutes) are necessary for efficient removal of organic compounds.

The majority of new installations will use separate deep-bed GAC contactors. The contactors may be located in the treatment process train, either ahead of, in lieu of, or following plant filters. GAC contactors can be either downflow or upflow-downflow series configuration in order to avoid the leakage of carbon fines that is common to all upflow carbon beds. The selected flow direction and location of the GAC bed are determined by the objective for the process. When used before filtration, the GAC acts as a filter and will require more frequent backwashing to remove solids and prevent interference of solids with the adsorption process, as well as to prevent undesirable high headloss. When GAC is placed following filtration, the backwash frequency is reduced dramatically, and the adsorption efficiency is not impaired by solids coating the carbon particles and increasing the diffusion resistance to the carbon. (Treatment trends are discussed in references 1–4.)

Activated carbon must be reactivated once it reaches its adsorptive capacity. Very small plants using less than 200 lb/d (100 kg/d) of carbon could economically use carbon on a once-through, throwaway basis. Medium-size plants using between 200 and 1,500 lb/d (100 and 700 kg/d) of GAC might also consider central, off-site carbon reactivation or service contracts with carbon manufacturers to replace and reactivate spent carbon.[4] Large plants will typically run their own on-site regeneration facilities. In most instances, spent GAC will be classified as nonhazardous solid waste that can be taken to a Class D landfill for disposal.

Disinfection By-Product Control

GAC may be used in either of two ways for DBP control. It can be used directly to remove DBPs, or it can be used indirectly to remove the precursors that react with the disinfectant to produce DBPs. In either case, good to very good removal is technically feasible. When the GAC is fresh, removal is nearly complete, but toward exhaustion, breakthrough begins. Trihalomethanes (THMs) containing bromine are adsorbed better than chloroform. (Control of DBPs is described in Chapter 19 and in references 5–9.)

Volatile Organics Removal

Volatile organic chemicals occur in both untreated and treated drinking water. Significant concentrations are more likely to be found in well waters than in surface waters. The potential health effects and acceptable limits of these substances in drinking water are of concern. Volatile organics can be removed by aeration or adsorption on GAC or synthetic resins, or by combinations of these processes, as described in Chapter 9. Strong oxidants such as ozone and advanced oxidation (see Chapter 16) and reverse osmosis (see Chapter 15) can also remove volatile organics. Boiling tap water for 5 minutes can also be effective for removing most of these organic compounds.

PRINCIPLES OF CARBON ADSORPTION

Activated carbon removes organic contaminants from water by a process of adsorption that results from the attraction and accumulation of one substance on the surface of another. In general, the chemical nature of the carbon surface is of relatively minor significance in the adsorption of organics from water and is secondary to the magnitude of the surface area of carbon available. Thus, a high surface area is the prime consideration in adsorption. GACs typically have surface areas of 2.44 to 6.84 million ft^2/lb (500 to 1,400 m^2/g). Activated carbon has a preference to adsorb organic compounds and, because of this selectivity, is particularly effective in removing organic compounds that may cause taste and odor problems in water supplies. Because activated carbon can reduce a wide range of organic compounds to trace concentrations (low microgram-per-liter level), it is often used as a general method to protect the consumer against some unknown organic pollutant or to remove DBP precursors.

Activated carbon can be made from a variety of materials, such as coal, wood, coconut shells, and petroleum coke. Granular carbons made from coal are hard and dense and can be pumped in water slurry without appreciable deterioration. Hydraulic handling of coal-derived carbon allows dust-free loading and unloading of filters. These granular carbons are well suited to water treatment; the carbon wets rapidly and does not float, but does form a densely packed bed with acceptable pressure drop characteristics. Because the carbon is quite dense, it generally has a high adsorption capacity. Most waterworks use coal-derived carbons.

Wood-derived carbons are generally softer than their coal counterparts, and thus more friable. They are less resistant to breakage and may generate dust problems during storage or transfer.

Carbon has two dominant forms in nature: graphic, which is flat, and diamond, which is tetrahedral. Activated carbon consists mainly of randomly organized graphite platelets as shown in Figure 17–1(a). Dr. M. Greenbank[10] defined activated carbon as a ''crude form of graphite with random or amorphous structure, which is highly porous over a broad range of pore sizes, from visible cracks and crevices to cracks and crevices of molecular size.'' London dispersion forces (a form of Van der Waals force) is very strong over short distances and provides for the attraction between a molecule and the flat graphite surface or platelet. The strength of the adsorption force is determined by the distance between the molecule and graphite platelets In areas with a high density of graphite platelets, the adsorption forces will be very high as shown in Figure 17–1(b). The type of raw materials and production process will determine the activated carbon adsorption characteristics.

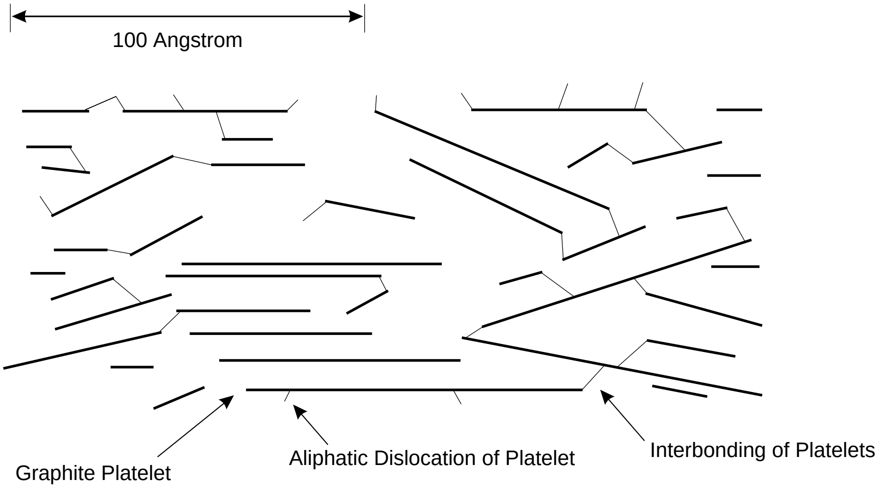
Adsorption is a dynamic process where molecules are continuously attached to and released from the surface similar to the ion-exchange process. Therefore, molecules compete for available surface area. The compounds with the highest surface attraction force and highest concentration will eventually dominate the surface coverage. This phenomenon of competitive adsorption also means that it is possible to displace a previously adsorbed compound with another compound that has a greater surface attraction. To prevent the undesirable, accidental bleed through a previously adsorbed compound, it is good practice to closely monitor the effluent quality of the contactor. In certain critical applications it may be necessary to design GAC systems with two beds in series, operated in lead and polish mode (as discussed in more detail later in this chapter).

ADSORPTIVE CHARACTERIZATION OF ACTIVATED CARBONS

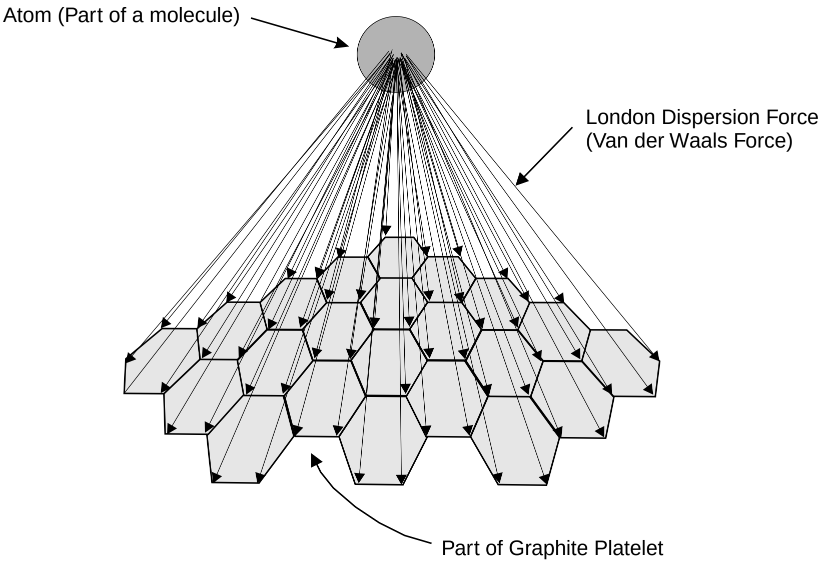
The most important characteristic of an adsorbent is the quantity of adsorbate it can accumulate. Simple capacity tests—such as the iodine number, molasses decolorizing index, threshold odor test, phenol value, tannin value, and others—may be used as an indirect measure of adsorptive capacity.[11–14] These capacity measures provide a general guide to the ability of an activated carbon to remove organics.

The iodine number is the number of milligrams of iodine adsorbed per gram of carbon when the carbon is in equilibrium, under specified conditions, with a solution of 0.02 N iodine concentration. It is an approximate measure of the adsorptive capacity of a carbon for small molecules such as iodine. The molasses decolorizing index is a measure of the adsorptive capacity of the carbon for color bodies in a specified molasses solution, compared to a standard carbon. Therefore, it is a measure of the adsorptive capacity for large molecules such as complex carbohydrates.

The adsorptive capacity of an activated carbon for a specific compound is best determined experimentally. Because competitive adsorption with other compounds in the water and the specific environmental conditions (temperature, pH, total dissolved solids [TDS], etc.) can significantly affect the adsorptive capacity, the experimental tests should be conducted under conditions closely matching the ultimate operating conditions.



(a) Platelet Graphite Structure of Activated Carbon



(b) Adsorption Force Between Molecule and Platelet

Fig. 17–1. Illustration of carbon adsorption principles (Courtesy of Calgon Carbon Corporation)

The adsorption isotherm describes the relationship, under given environmental conditions, between the amount of a specific compound adsorbed (concentration in grams compound per gram carbon) and the liquid-phase concentration at equilibrium. For example, Figure 17–2 shows a typical equilibrium relationship found for total organic carbon (TOC) removal in Colorado River water.[16]

For design purposes it can be very useful to mathematically describe the adsorbate/adsorbent interaction. In single-solute systems the Freundlich isotherm equation is commonly used and has been shown to accurately describe most adsorption data. The Freundlich equation is empirically derived and assumes a straight logarithmic isotherm. The mathematical formula for the Freundlich isotherm is as follows:

$$\frac{x}{m} = kC^{1/n} \tag{17–1}$$

where:

x = mass of compound adsorbed, g

m = mass of carbon, g

$\frac{x}{m}$ = amount of compound adsorbed per unit weight of carbon

C = residual concentration of compound left in solution, mg/L or μg/L

The terms k and n are isotherm constants determined experimentally for a specific compound of interest and the specific GAC product under specific test conditions.

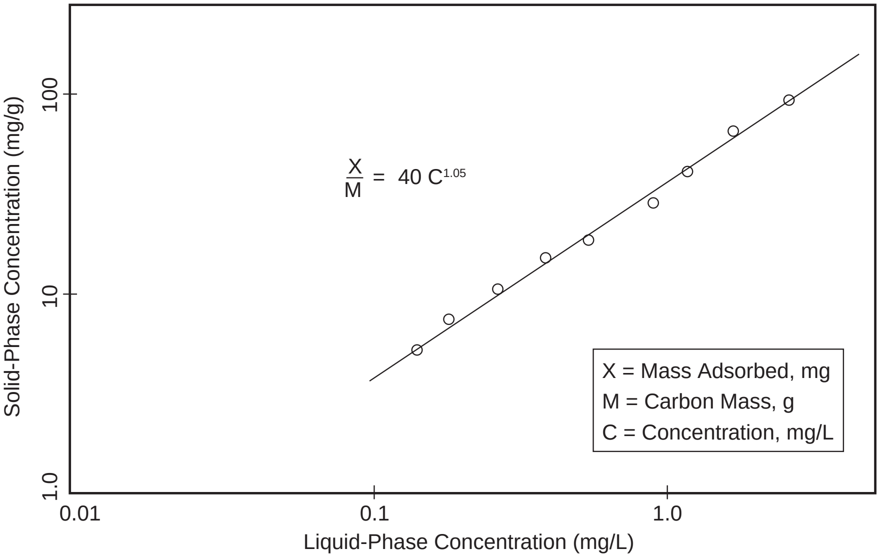


Fig. 17–2. Example isotherm for total organic carbon (TOC) adsorption in Colorado River water (*Source:* Reference 15. Adapted from *Optimization and Economic Evaluation of Granular Activated Carbon for Organic Removal.* Copyright © 1989, American Water Works Association and the American Water Works Research Foundation.)

For convenience, the isotherm equation above can be converted into logarithmic form:

$$\log \frac{x}{m} = \log k + \frac{1}{n} \log C \tag{17–2}$$

where $1/n$ represents the slope and k the intercept of the straight-line isotherm shown in Figure 17–2. Detailed procedures for establishing the experimental conditions and conducting and interpreting isotherm adsorption tests are presented elsewhere.

From an isotherm test, it can be determined whether a particular degree of treatment can be achieved. The test will also show the approximate capacity of the carbon for the application and provide a rough estimate of the carbon dosage required. Isotherm tests also afford a convenient means of studying the effects of pH and temperature on adsorption. Isotherms put a large amount of data into concise form for ready evaluation and interpretation. Isotherms obtained under identical conditions by using the same test solutions for two test carbons can quickly and conveniently be compared to reveal the relative merits of the different carbons.

PHYSICAL CHARACTERIZATION OF ACTIVATED CARBONS

The physical properties of granular carbons that are important to performance are resistance to breakage, particle size, and density. Resistance to breakage is measured by using empirical tests, such as the abrasion number and hardness number tests. Particle size is determined by a screen analysis, from which the mean particle diameter and effective size can be calculated. Density is simply the weight per unit volume of the carbon. Typical specifications of an activated carbon suitable for water treatment applications are given in Table 17–1.

Carbon particle size distribution is important in filter and contactor design for GAC. If the activated carbon is to replace anthracite coal in a dual-media filter, it should have similar filtration characteristics and similar backwashing characteristics to the coal. Fortunately, as shown in Table 17–2, commercial carbons are available with characteristics that are similar to anthracite coals used as filter media. The typical size distribution of a commercial carbon is determined by the manufacturing process. For

TABLE 17–1. Typical Activated Carbon Characteristics

Item	Units	Value for GAC	Value for PAC
Total surface area	m^2/g	500–1,500	500–1,500
Bulk density	lb/ft^3 (kg/m^3)	26 (420)	26 (420)
Apparent density	g/cm^3	0.25 minimum	0.20–0.75
Effective size	mm	0.3–2.0*	0.044
Uniformity coefficient	dimensionless	2.1	—†
Iodine number	mg/g	500 minimum	500 minimum
Abrasion number	%	70 minimum	—†
Ash	%	4 maximum	—†
Moisture	%	8 maximum	8 maximum

*Depends on design.

†Not applicable for PAC.

TABLE 17–2. Comparison of Coal and Granular Carbon as Filtration Media

	Values for Hard Coal Media	Values for a Coal-Based GAC	
		8 × 30 Mesh*	14 × 40 Mesh*
Real density, g/cm^3	1.5–1.6	2.1	2.1
Particle density in water, g/cm^3†	1.5–1.6	1.5–1.6	1.5–1.6
Effective size in:			
Single-medium filters	0.5 mm	—†	0.5 mm
Multimedia filters	0.8 mm	0.8 mm	—§
Uniformity coefficient	Less than 1.75	1.9 or less	1.7 or less

(Courtesy of Calgon Carbon Corporation)

*U.S. Sieve Series.

†The pores of the activated carbon filled with water.

‡To replace larger particles in multimedia filter.

§To replace smaller particles in single-medium filter.

example, an 8 × 30 mesh GAC will contain fewer than 3 percent particles larger than a No. 8 mesh and fewer than 1 percent smaller than the No. 30 mesh, with a mean size of 1.6 mm. Similarly, a 14 × 40 mesh GAC will have fewer than 3 percent particles exceeding a No. 14 mesh and less than 1 percent below a No. 40 mesh, with a mean size of 0.9 mm.

The headloss through granular carbon is a function of the carbon size, the depth of the carbon layer, the hydraulic throughput rate, and the water temperature. Figure 17–3 presents the headloss data for some commercial carbons in downflow service as filter media following backwashing. Bed expansion as a function of backwash of flow is shown in Figure 17–4.

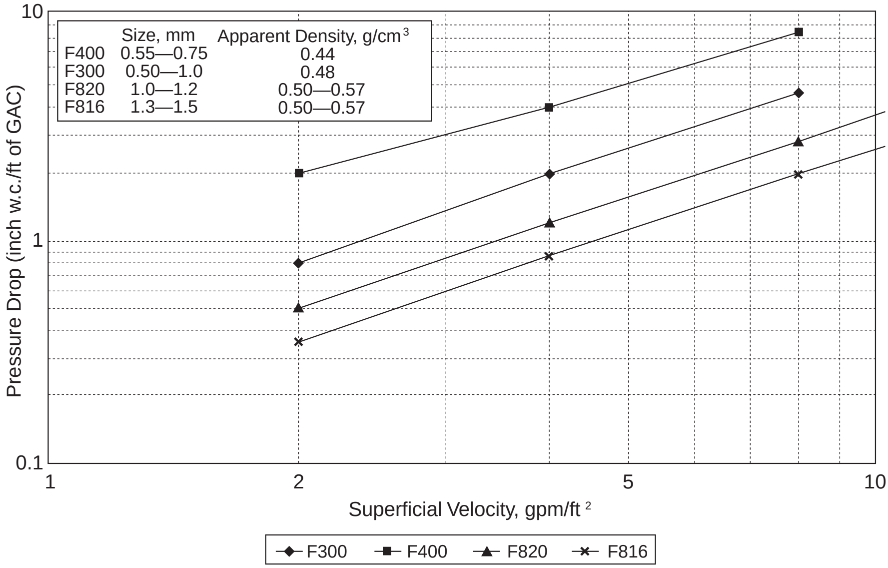


Fig. 17–3. Pressure drop versus hydraulic loading (Courtesy of Calgon Carbon Corporation)

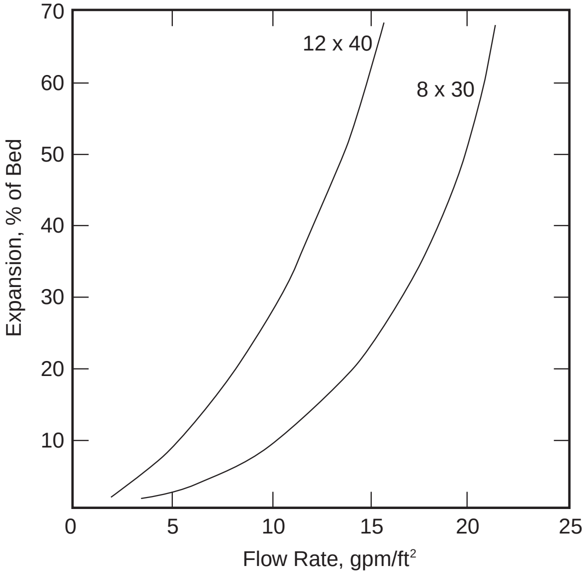


Fig. 17–4. Expansion of carbon bed at various flow rates. Carbon: 12 × 40, 8 × 30; liquid: water at 22°C (*Source:* Reference 10; courtesy of Calgon Carbon Corporation)

The primary characteristic that differentiates PAC from GAC is particle size. Adsorptive and physical characteristics are similar. Table 17–3 shows the typical particle-size distributions from commercially available PAC products.

DESIGN OF ACTIVATED CARBON FACILITIES FOR WATER TREATMENT

GAC and PAC have been used extensively in water treatment. PAC facilities are relatively simple and straightforward and can often be added to existing treatment plants with minimal disruption. GAC application requires extensive new treatment and support facilities.

Activated carbon has an affinity to adsorb many different compounds. However, for each application of activated carbon, the designer must determine the one or more key objectives for carbon usage—taste and odor control, TOC removal, DBP precursor removal, specific chemical removal, etc. In addition, the specific design conditions (temperature, background water matrix, TDS, pH, etc.) will affect the carbon perform-

TABLE 17–3. Typical Particle Sizes for Powdered Activated Carbon

Sieve Size	Percentage of Particles Passing Through the Mesh
100 mesh	99
200 mesh	95
325 mesh	90

ance. Because of the complex interaction of site-specific conditions, the performance of the activated carbon is generally best determined by bench and pilot studies. Extrapolation from other applications can also assist the designer in developing the proper design criteria.

Powdered Activated Carbon

PAC is a bituminous coal–based product activated at high temperature in a steam atmosphere and then pulverized to a powder form. PAC has been used successfully for more than 50 years to remove taste and odor from public drinking water supplies. Except for particle size, PAC is identical to the GAC used in water treatment. In this type of use, PAC dosages usually are in the range of 1 to 5 mg/L, although dosages as high as 20 to 30 mg/L have been used in some places for short periods of time when taste and odor problems were severe. PAC is commonly used on a one-time, throwaway basis, with no attempt at recovery or reuse. Used PAC containing adsorbed contaminants is removed with the sludge from the treatment basins.

The principal use of PAC in water treatment is to remove taste and odor, typically associated with changes in temperatures and early spring melting of snow. In some waters, PAC may also remove color or organics that otherwise would interfere with coagulation or filtration. During its widespread use by water utilities for more than 50 years, no harmful effects have been reported.

A key advantage of PAC is that it can easily be added to the existing conventional treatment plants. New PAC dosing and mixing facilities are required, while existing sedimentation basins and filters effectively remove the PAC from the water. PAC is particularly attractive to resolve seasonal taste and odor problems because the capital investment in equipment is low and the operation costs for purchasing PAC are limited to periods when taste and odor problems occur.

PAC is available in bags, drums, and bulk delivery. Bulk deliveries are available by truck up to 40,000 lb (18,000 kg) and by rail car up to 80,000 lb (36,000 kg). The choice of delivery is based on the size of the facility, availability, storage requirements, and operational philosophy.

There are two basic methods used for the storage and feeding of PAC. The first method is to store the PAC in the dry form in a silo and feed with a gravimetric or volumetric feeder. These feeders can meter the PAC into a water solution eductor that will create PAC slurry to the point of application. The second method is to unload the dry PAC into a slurry tank while mixing it with water to create slurry, typically composed of 1 lb PAC per gallon of water (0.12 kg PAC/L water). The PAC is kept in suspension in the slurry storage tank with mechanically driven mixers. The PAC slurry is pumped from the bulk storage to a day tank and from the day tank through metering pumps to each point of application. Use of a slurry tank type of bulk storage reduces the dust and housekeeping problems associated with a dry feeder system.

PAC is typically dosed to the water in a fashion similar to that used for chemicals. The PAC dose (in milligrams per liter) is determined to achieve the desired impact. The maximum and minimum dose requirements for different water qualities and conditions can typically be bracketed through bench-scale jar tests. Because the powdered activated carbon has very small particles, the intraparticle diffusional impact is greatly reduced, and the system will establish equilibrium in a relatively short time. The contact provided in sedimentation basins and ahead of filter units will typically provide sufficient time to establish equilibrium. In these cases, the dose can be established from the isotherm information and can be confirmed through bench tests. The ultimate dose will be established based on field results.

Granular Activated Carbon

An American Water Works Association Research Foundation (AWWARF) survey identified 66 surface water and 33 groundwater treatment plants using GAC.[17] In these plants, GAC was used to control taste and odor, as well as DBP precursors, in drinking water. (For specific cases of historical uses of GAC, see references 18–25.) In 1992, the city of Cincinnati, Ohio, brought on line for its 175-mgd water works the largest GAC adsorption and on-site regeneration facilities ever built. (See the special discussion that follows later in this chapter.)

GAC facilities require the following system components:

- Carbon contactors for the water to be treated for the length of time required to obtain the necessary removal of organics
- Reactivation or replacement of spent carbon
- Transport of makeup or reactivated carbon into the contactors
- Transport of spent carbon from the contactors to reactivation or hauling facilities
- Facilities to backwash the GAC beds

These facilities are discussed in more detail later in this section.

GAC Contactor: Process Design The process design of a GAC contactor determines the size and number of contactors. GAC contactor process design is complex because the kinetics of the contactor are controlled by the diffusional resistance of the target compound in the GAC bed. The design of the GAC bed is controlled by the particle size, porosity, pore size, temperature, adsorption isotherm, application velocity, and required contact time. The result is that a breakthrough curve develops in the GAC bed with a portion of the GAC in the contactor in equilibrium with the feedwater compound and the leading edge of the curve in a ‘‘polishing’’ mode. This breakthrough curve travels through the bed while establishing equilibrium in the bed itself (see Figs. 17–5 and 17–6).

Two key design parameters are commonly used to design the GAC contactor: the empty bed contact time (EBCT) and the approach velocity or hydraulic loading rate (represented by v_a). Expressions for these parameters are as follows:

$$\text{EBCT} = \frac{\text{contactor volume}}{\text{flow}} \tag{17–3}$$

$$v_a = \frac{\text{flow}}{\text{contactor area}} \tag{17–4}$$

In addition to the basic design parameter, the designer must determine the key operational parameters. The most important of these is the carbon usage to maintain the desired treatment objectives. During the operation of the contactor, the effluent concentration of the target compound is typically very low until the breakthrough curve approaches the end of the contactor. At that time, the compound will start leaking

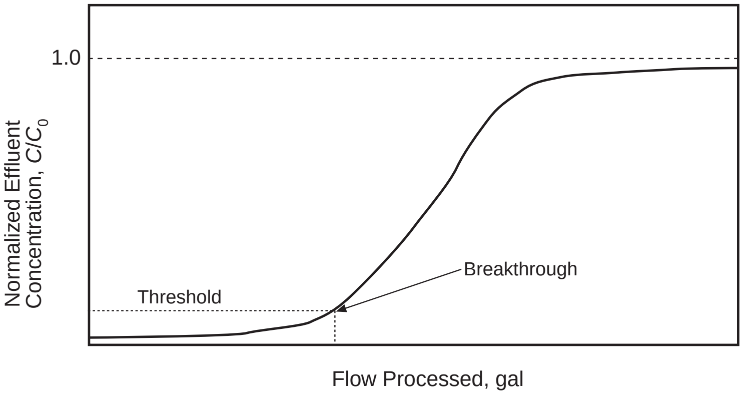


Fig. 17-5. Typical breakthrough curve

through, showing the breakthrough curve in effluent concentration (see Fig. 17–5). The bed is exhausted when the effluent concentration reaches the target treated water concentration.

For some limited applications, two GAC contactors may be designed to operate in series in order to use the full adsorptive capacity of the respective beds (see Fig. 17–6). Contactor 1 continues to serve as lead unit until it reaches capacity (complete breakthrough). At this point contactor 2, the lag contactor, serves as the polishing unit. Contactor 1 is taken off-line and regenerated while contactor 2 becomes the new lead unit. Once regenerated, contactor 1 becomes the new lag or polishing unit. By switching the lead and lag function, both GAC units can operate until reaching capacity.

The contactor depth is determined by the shape of the breakthrough curve, since it determines the ability to operate the bed until full capacity is reached (see Fig. 17–7). A sharp curve (for which breakthrough appears rapidly) means that a relatively

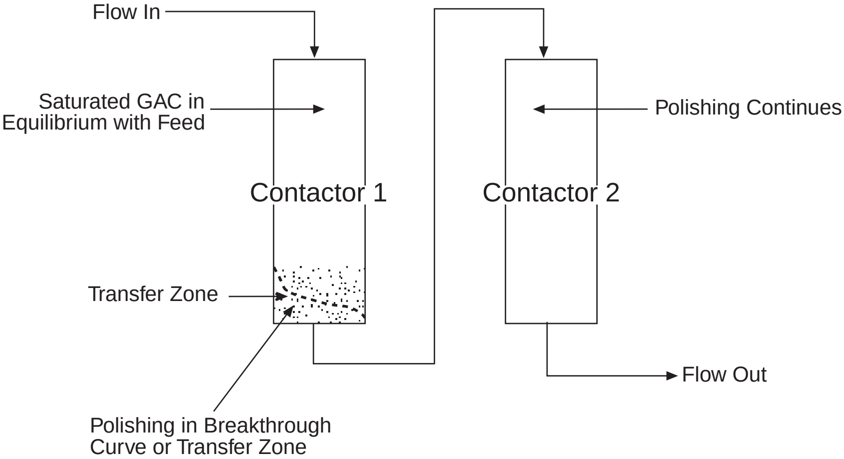


Fig. 17-6. Series operation of GAC contactors

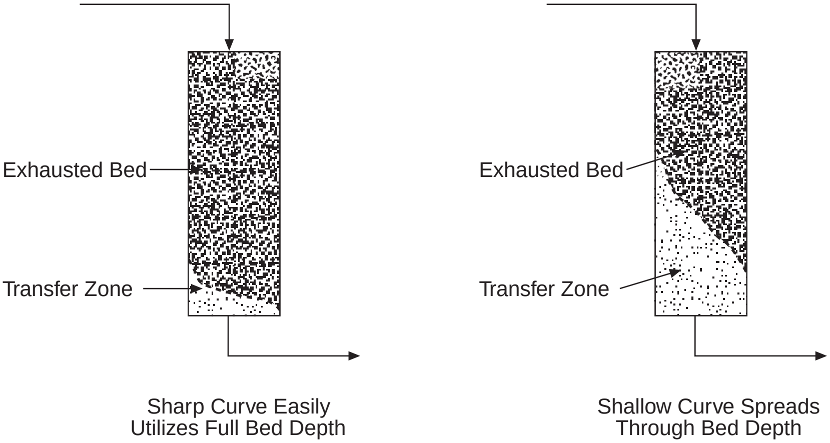


Fig. 17–7. Breakthrough curve shape determines effectiveness of adsorption

shallow bed can be used while protecting the effluent from degradation due to leakage. By contrast, the presence of an extended breakthrough curve means that early breakthrough will occur while the bed still has remaining adsorptive capacity that cannot be realized because the effluent quality is deteriorating too rapidly.

The designer determines the carbon usage based on the isotherm and kinetic characteristics of the bed design. The treatment capacity of a GAC bed is typically expressed as the “number of empty bed volumes treated until regeneration,” or the “number of EBCTs.” This parameter can be estimated based on isotherm results. On the basis of Equation 17–1, the carbon adsorption capacity of the GAC is calculated as follows:

$$\left(\frac{x}{m}\right)_{\text{design}} = \text{SF}kC_0^{-1/n} \tag{17–5}$$

where:

$(x/m)_{\text{design}}$ = the design capacity of the GAC (pound compound per pound GAC) to retain the compound when in equilibrium with maximum concentration
SF = safety factor
C_0 = influent concentration, mg/L

The designer must apply an appropriate safety factor to reduce the maximum design capacity (SF = 0.75–0.9). Once the maximum adsorptive capacity is set, the treatment capacity of the unit is calculated as:

$$\text{Capacity (number of empty bed volumes [EBVs])} = \left(\frac{x}{m}\right)_{\text{design}} \frac{\rho_b}{C_0} \times 16{,}000 \tag{17–6}$$

and

$$\text{Time to exhaustion (in unit days)} = \left(\frac{x}{m}\right)_{\text{design}} \frac{\rho_b}{C_0} \times 11.1 \times \text{EBCT} \qquad (17\text{–}7)$$

where ρ_b is the bulk GAC density (lb/ft^3), EBCT in minutes, and 11.1 is a conversion factor. The best method for determining the operational and design parameters is to conduct pilot, bench, or rapid small-scale column tests (RSSCTs). These latter tests are described next.

RSSCT Testing. The rapid small-scale column test method was developed by Crittenden and coworkers as a method to establish the operational parameters and evaluate treatment efficiency of GAC columns.[26–28] The key to the RSSCT test procedure is to set up the small-scale test columns to provide hydraulic simultude between the small-scale test unit and the ultimate full-scale unit to simulate the full-scale performance of the bed. The hydraulic loading and EBCT of the RSSCT are selected to keep the dimensionless groups that describe the adsorption process (i.e., the Reynolds numbers) constant as the full-scale unit is scaled down to the RSSCT.

The RSSCT tests can be conducted in a fraction of the time required for a full-blown pilot study. Because this approach requires only a small quantity of test water, RSSCT tests can also be conducted in a laboratory remotely located from the application with water shipped to the test laboratory. Full-scale design variables such as EBCT, use rate, and breakthrough curves can be determined with a high probability of success.

Crittenden et al.[28] describe the RSSCT test requirements under conditions of adsorbing specific organic compounds, as well as the removal of DOC. The study shows that the relationship between RSSCT performance of pilot units cannot be determined a priori because of the complex effect of the water matrix DOC on the adsorption capacity and kinetics of the carbon. However, for specific organic chemicals, the relationship between the small- and full-scale contactors can be described by the following dimensionless expressions:

$$\frac{\text{EBCT}_S}{\text{EBCT}_L} = \left|\frac{R_S}{R_L}\right|^2 = \frac{t_S}{t_L} \qquad (17\text{–}8)$$

$$\frac{v_S}{v_L} = \frac{R_L}{R_S} \qquad (17\text{–}9)$$

where:

S, L = indicators of the small and large contactors, respectively
R = the absorbent particle radius
t = the run time of the reactor
v = the hydraulic loading rate (approach velocity)

In order to determine the true relationship between the RSSCT test and full-scale implementation, Crittenden et al.[28] recommend that at least one pilot study be conducted to determine the scale-up dimensions for the particular application.

The RSSCT tests will simulate the performance of large-scale GAC contactors quite well. Figure 17–8 shows a typical comparison of RSSCT and pilot columns for re-

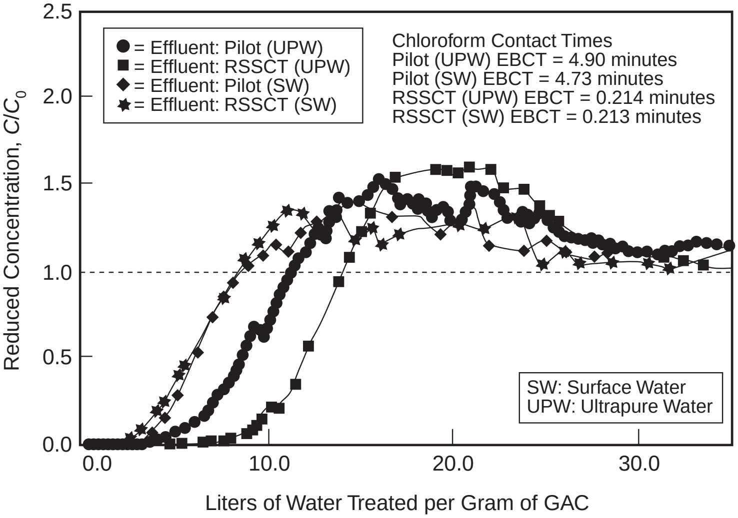


Fig. 17-8. Comparison of the GAC pilot column and RSSCT effluent concentration history profiles for chloroform in ultrapure water and surface water (*Source:* Reference 28. Reprinted from *Prediction of GAC Performance Using Rapid Small-Scale Column Test,* by permission. Copyright 1989, American Water Works Association and American Water Works Research Foundation.)

moval of chloroform from pure water and from surface water. The RSSCT tends to break through slightly before the pilot unit. Note that the RSSCT results are obtained in about 4 percent of the time required for the pilot study.

Pilot Plant Tests. Despite the usefulness of the RSSCT, larger-scale and pilot plant tests are needed to select the carbon and the most economical plant design for full-scale water and wastewater treatment designs. Pilot column tests make it possible to:

- Determine treatability
- Select the best carbon for the specific purpose based on performance
- Determine the required empty bed contact time
- Establish the required carbon dosage that, together with laboratory tests of reactivation, will determine the capacity of the carbon reactivation furnace or the necessary carbon replacement costs
- Determine the effects of influent water quality variations on plant operation

During pilot testing, the influence of longer carbon contact times on reactivation frequency can be measured. These measurements allow costs to be minimized through a proper balance of these two design factors.

Pilot-scale GAC columns are at least 4 in. (100 mm) in diameter to minimize the wall effect of a small contactor. Units are operated over a range of contact times and hydraulic loading rates to simulate full-scale performance. These studies can be very time-consuming, since the GAC contactor should be operated until the GAC is exhausted (i.e., has reached its adsorptive capacity).

GAC Contactors: Facilities Design Selection of the general type of carbon contactor to be used for a particular water treatment plant application may be based on several considerations, including economics and the judgment and experience of the engineering designer. The choice is generally made from three types of downflow vessels:

- Deep-bed, factory-fabricated, steel pressure vessels up to 17-ft (3.7-m) maximum diameter provide GAC bed volumes up to 700 ft^3 (20 m^3)—i.e., 20,000 lb (9,100 kg)—per single contactor. Multiple units are used to build large systems.
- Reinforced concrete, gravity-filter-type boxes are used for carbon volumes, typically above 1,000 ft^3 (30 m^3). Shallow beds could possibly be used only when short contact times are sufficient or when long service cycles between carbon regenerations can be expected from pilot plant test results. Shallow beds increase backwash facility needs. Deep beds are used for most applications.
- Deep-bed, site-fabricated, 20–30-ft (6–10-m)-diameter, open concrete or steel, gravity tanks may be used for midsize units.

These ranges overlap, and the designer may very well make the final selection based on local factors other than total capacity that affect efficiency and cost.

As previously mentioned, the current design trend in retrofitting existing water treatment plants with new GAC adsorption facilities is to provide separate, postfiltration, downflow contactors. Contactor flow rates are usually 2 to 10 gpm/ft^2 (5 to 25 m/h), and GAC bed depths are normally 2.5 to 15 ft (0.75 to 5 m). A direct linear relationship between contact time and carbon bed performance has been found in full-scale plant tests and concurrent small column tests. Carbon performance at a given contact time has been found to be unaffected by variations in hydraulic loading rates in the 2 to 10 gpm/ft^2 (5 to 25 m/h) range. Thus, in terms of adsorption only, contact time is the governing criterion. The surface loading rate is maintained within the ranges of practicality to limit hydraulic headloss and turbulence.

When the granular carbon bed is functioning as both a turbidity removal unit and an adsorption unit, there may be reasons to limit the bed depth and flow rate parameters to remove turbidity effectively and to backwash the filter properly. If GAC is to be effective in turbidity removal, the GAC particles must be hard enough to withstand vigorous backwash agitation. At the same time, it should be dense enough to expand during the backwash cycle and to settle quickly for immediate resumption of filtration. As discussed earlier, coal-based granular carbon possesses approximately the same density and filtration characteristics as anthracite and has found increasing use in the water field.

Particle size of the carbon, in addition to contact time, should be considered carefully as a design factor. Reduction of particle size for a given set of flow conditions is a means of increasing adsorption rates and thereby improving adsorption performance. Reduction in particle size to improve adsorption must be consistent with other significant factors such as headloss and backwash expansion. The length of the filter run in an adsorption-filtration bed would also be a problem if too small a particle size were chosen.

Where existing rapid sand filters are being converted to adsorption-filtration units, the permissible depth of the carbon layer will be limited by the freeboard available in the existing structure. Adequate space between the carbon surface and the backwash trough bottoms should be available to permit at least 30 percent expansion of the carbon layer.

The advanced wastewater treatment (AWT) experience with GAC contactors may be applied to water purification if some differences in requirements are taken into account. The required contact time must be determined from pilot plant test results. Contactors may be designed in a downflow or upflow mode of operation. Upflow packed beds provide maximum theoretical carbon adsorption efficiency through the use of countercurrent flow principles. However, because leakage of some carbon fines (1 to 5 mg/L) in upflow carbon column effluents can cause problems in downstream processes, downflow carbon beds are more commonly used in municipal water treatment applications. For example, at the Orange County Water Factory 21 (Fountain Valley, California), upflow beds were converted to downflow beds to correct a problem with escaping carbon fines. This full-scale plant operating experience indicates that leakage of carbon fines is not a problem in properly operated downflow GAC contactors.

Single beds or two beds in series may be used. Open gravity beds or closed pressure vessels are permissible. Structures may be properly protected steel or reinforced concrete. In general, small plants will use steel, and large plants may use steel or reinforced concrete.

Sand in rapid filters has, in some instances, been replaced with GAC. In situations where contact times are short and GAC regeneration or replacement cycles are exceptionally long (several months or years), as may be the case in taste and odor removal, this approach may be a solution. However, with the short cycles anticipated for most organics, conventional concrete-box-style filter beds may not be well suited to GAC contact; deeper beds may be more economical and provide more efficient use of GAC. Beds deeper than conventional filter boxes, or contactors with greater aspect ratios of depth to area, provide much greater economy in capital costs; the contactor cost for the needed volume of carbon is much less. In water slurry, carbon can be moved from contactors with conical bottoms easily and quickly and with virtually no labor. Flat-bottomed filters of a type that require labor to move the carbon unnecessarily add to carbon transport costs. The labor required to remove carbon from flat-bottomed beds varies considerably in existing installations from a little labor to a great deal, depending on the design of the excavation equipment.

For many GAC installations intended for precursor organic removal or synthetic organic removal, specially designed GAC contactors should be installed. Contactors should be equipped with flow-measuring devices. Separate GAC contactors are especially advantageous where GAC treatment is required only seasonally, because the contactors then can be bypassed when they are not needed, possibly saving unnecessary exhaustion and reactivation of GAC.

Carbon Contactor Design: Underdrains The design of carbon contactor underdrains requires experienced, expert attention. Although good proven filter underdrain systems—such as lateral collectors with coarse gravel, screens, and full-floor blocks—are available, they should be designed for GAC retention. Earlier underdrain designs have failed in many installations for conventional filter service, and they continue to be misapplied to GAC contactors as well as filters.

GAC Reactivation or Replacement One of the principal costs for GAC treatment is the reactivation frequency required. Organics of concern in water treatment may break through at carbon loadings as low as 0.001 lb organics/lb carbon and can exceed 0.25 lb organic/lb carbon. The actual carbon loading or carbon dosage for a given case must be determined from pilot plant tests. Generic cost curves, which are plots of flow (in million gallons per day) versus cost (capital or operation and maintenance costs), cannot be applied directly to water treatment. Allowance must be made in the capital costs for the different reactivation capacity needed, and in the operation and maintenance costs for the actual amount of carbon to be reactivated or replaced.

Carbon life varies greatly depending on the application. A short carbon life (6 months or less between regeneration cycles) will greatly increase the treatment cost. Long carbon life—exceeding 2 years—is desirable. The designer has little control over the carbon life, beyond selecting the carbon type and providing some pretreatment to reduce fouling. Pretreatment, such as enhanced coagulation to reduce the TOC loading to the GAC, can extend the carbon life. In addition, preoxidation such as ozonation can change the characteristics of the TOC in the raw water to promote biological activity and extend GAC life under certain circumstances.

Spent carbon may be removed from contactors and replaced with virgin carbon, or it may be reactivated either on-site or off-site. The most economical procedure depends on the quantities of GAC involved. As already discussed, on-site reactivation is more economical for larger volumes. For small quantities of carbon, replacement or off-site reactivation will probably be most economical. Several GAC suppliers will pick up spent carbon and replace it with new carbon on a contract basis.

Carbon may be reactivated thermally to very near virgin activity. However, carbon burning losses may be excessive under these conditions. Experience in industrial and wastewater treatment indicates that carbon losses can be maintained at 8 to 10 percent per cycle with the reactivated carbon capacity (as indicated by the iodine number) at about 90 percent of the original virgin capacity. For certain organics, there may be no decrease in actual organics removal despite a 10 percent drop in iodine number.

GAC may be reactivated in a multiple-hearth furnace, a fluidized-bed furnace, a rotary kiln, or an electric infrared furnace. However, multiple-hearth furnaces are presently dominating the market. Spent GAC is drained in a screen-equipped tank (40 percent moisture content) or in a dewatering screw (40 to 50 percent moisture) before being introduced to the reactivation furnace. Dewatered carbon is usually transported by a screw conveyor. Following thermal reactivation, the GAC is cooled in a quench tank. The water-carbon slurry may then be transported by means of diaphragm slurry pumps, eductors, or a blow tank. The activated carbon may contain fines produced during conveyance; these fines should be removed in a wash tank or in the contactor. Maximum furnace temperatures and retention times are determined by the amount (weight organics/weight carbon) and nature (molecular weight or volatility) of the organics adsorbed.

Off-gases from carbon reactivation present no air pollution problems, provided they are properly scrubbed. In some cases, an afterburner may also be required for odor control. Multiple-hearth furnaces are the simplest, most reliable, and easiest to operate for GAC reactivation. The infrared and fluidized-bed units have virtually disappeared from the market.

It is necessary with all four types of furnaces to specify top-quality materials to suit the conditions of service, as well as to see that these materials are properly installed. Corrosion resistance is important in the furnace itself and especially in all auxiliaries to the furnace.

Carbon Transport Facilities Once carbon is introduced into the adsorption-regeneration system, GAC is usually transported hydraulically in slurry form. Air or pneumatic transport of carbon is sometimes used for bulk handling of makeup carbon.

Handling characteristics have been reported for water slurries of 17 × 40 mesh granular carbon in a 2-in. (50-mm) pipeline. The data indicate that a maximum of 3 lb carbon/gal (0.36 kg carbon/L) of water could be transported hydraulically but that it is better to use a ratio of 1 lb carbon/gal (0.12 kg carbon/L). The velocity necessary to prevent settling of carbon is a function of pipe diameter, granule size, and liquid and particle density. The minimum linear velocity to prevent carbon settling was found to be 3.0 ft/sec (0.9 m/s). It is recommended that a velocity of 3.5 to 5.0 ft/sec (1.0 to 1.5 m/s) be used for design. Velocities of over 10 ft/sec (3 m/s) are undesirable because of carbon abrasion and pipe erosion. Carbon delivery rates are a function of pipe diameter, slurry concentration, and linear velocity. Data are shown in Figure 17–9 and 17–10 for a 2-in. (50-mm) pipe and a 1-in. (25-mm) pipe, respectively. The shaded area in Figures 17–9 and 17–10 show the operating region that meets the design criteria. One-inch (25-mm) pipes are able to provide GAC transfer rates of 5 to 16 lb/min. Two-inch (150-mm) pipes are used for GAC transfer rates above 30 lb/min. Pressure drop data for various slurry concentrations and velocities in 2-in. (50-mm) pipe are shown in Figure 17–11. This figure shows that the headloss increases rapidly as the slurry concentration increases. The designer must balance the headloss and

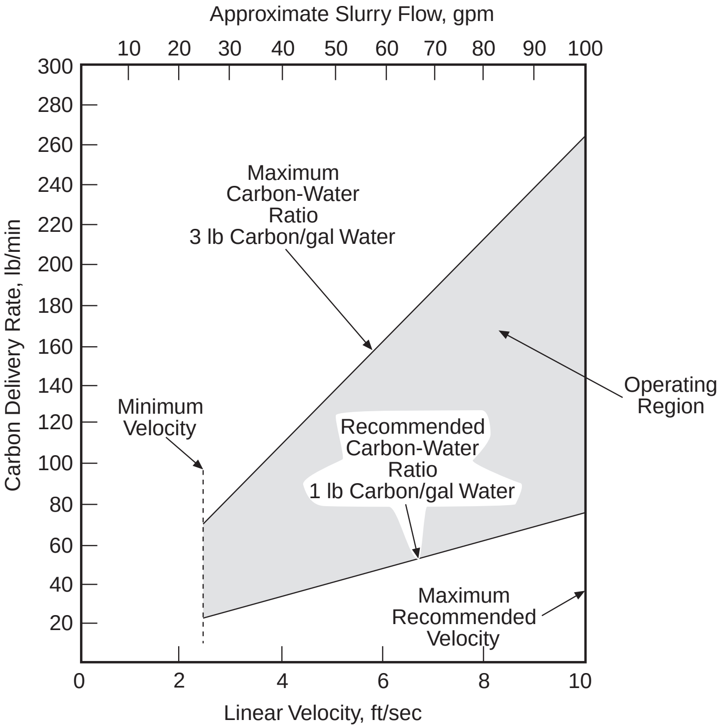


Fig. 17–9. Carbon delivery rate (2-inch pipe) (Courtesy of Calgon Carbon Corporation)

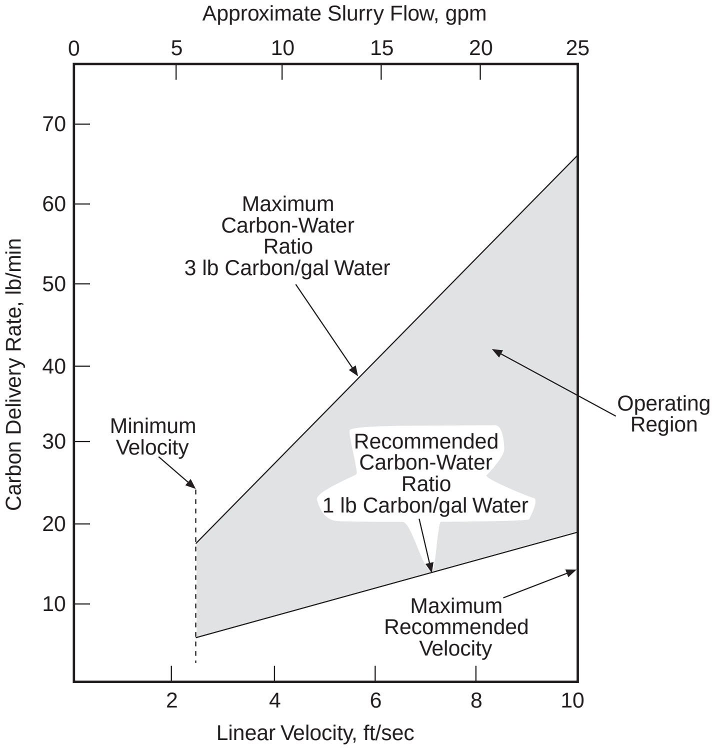


Fig. 17–10. Carbon delivery rate (1-inch pipe) (Courtesy of Calgon Carbon Corporation)

capacity relationship with the pump curves. See Krasner et al.[29] for a detailed discussion on carbon slurry hydraulics.

Pilot plant tests indicate that after an initial higher rate, the rate of attrition for activated carbon in moving water slurries is approximately constant for any given velocity, reaching an approximate value of 0.17 percent fines generated per exhaustion-regeneration cycle. This deterioration of the carbon with cyclic operation has been reported to be independent of the velocity of the slurry, within the recommended range of 3.5 to 5 ft/sec (1.0 to 1.5 m/s). Loss of carbon by attrition in hydraulic handling apparently is not related to the type of pump used (diaphragm or centrifugal).

Carbon slurries can be transported by using water- or air-pressure centrifugal pumps, eductors, or diaphragm pumps. The choice of motive power is a combination of owner preference, turndown capabilities, economics, and differential head requirements.

CARBON ADSORPTION ENHANCEMENT WITH OZONE

Depending on water quality, ozone pretreatment ahead of GAC adsorption may increase GAC service life by 5 to 100 percent.[30] Under most circumstances, a 10 to 20 percent extension of carbon service life can be expected. Preozonation tests should be conducted during GAC pilot plant operations.

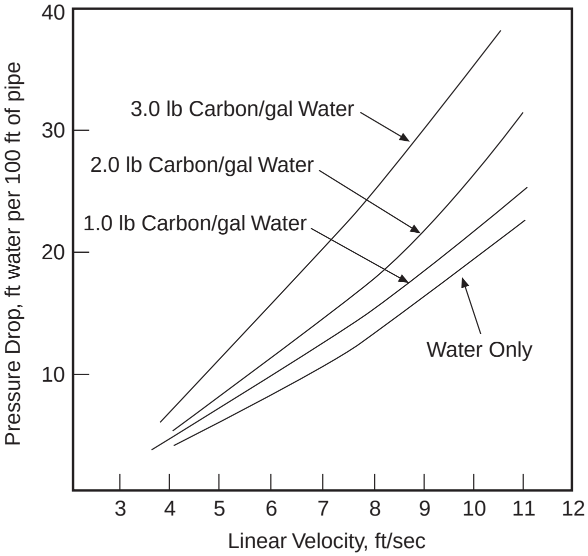


Fig. 17-11. Pressure drop of carbon-water slurries (2-inch pipe) (Courtesy of Calgon Carbon Corporation)

Ozonation does not oxidize natural organic matter (NOM) completely to carbon dioxide and water. Rather, the ozone oxidation results in a fragmentation of larger NOM molecules and generates small organics. Substitution reactions produce aldehydes, ketones, and other by-products.

Ozonation typically increases the biodegradability of NOM in water because large organic molecules are oxidized into smaller organic molecules that are readily biodegradable. This increase in assimilable organic carbon (AOC) can lead to accelerated bacterial growth and regrowth in the distribution system. When ozonation is placed upstream of filtration and environmental conditions such as dissolved oxygen, pH, and temperature are favorable, microbiological activity is increased and AOC removal is enhanced. Ozone addition not only increases the biodegradability of the dissolved organics but also introduces large amounts of oxygen to the water, thus creating an excellent environment for biological growth. The potential advantages of biological active filtration are:[31]

- Production of a biologically stable water that does not promote excessive bacterial growth and regrowth in the distribution system
- Removal of naturally occurring organic matter that can serve as precursors to by-product formation as a result of residual disinfection with free or combined chlorine
- Reduction of the residual disinfectant demand of the product water so that proposed regulations limiting the maximum disinfectant residual can be met
- Removal or control of ozonation by-products

GAC is an ideal medium to support biological growth following ozonation. For a complete discussion of biologically active filtration using GAC, the reader is referred to Chapter 12, ''Filtration.''

EXPERIENCES WITH ACTIVATED CARBON

Activated carbon has been used successfully at many treatment plants on a continuous and interim basis. Several applications are presented in this section. The reader should exercise caution when extrapolating experiences from one application to another, since specific objectives vary from plant to plant. The experiences reported here serve as examples only.

Real-World PAC Example: Omaha, Nebraska

Metropolitan Utilities District (MUD) of Omaha, Nebraska, operates a 158-mgd (598-ML/d) water plant that takes water from the Missouri River.[32] Figure 17–12 is a flow diagram of the Florence Water Treatment Plant. The water is split-treated with half lime softened and half alum coagulated. PAC can be added before the presettling basin or with alum before the settling basins.

MUD uses PAC mainly during the spring when snowmelts cause tastes and odors. In addition, PAC is used during a large rainfall event or when high runoff causes increased turbidities. Current operating practice is to maintain a polishing dose of 6

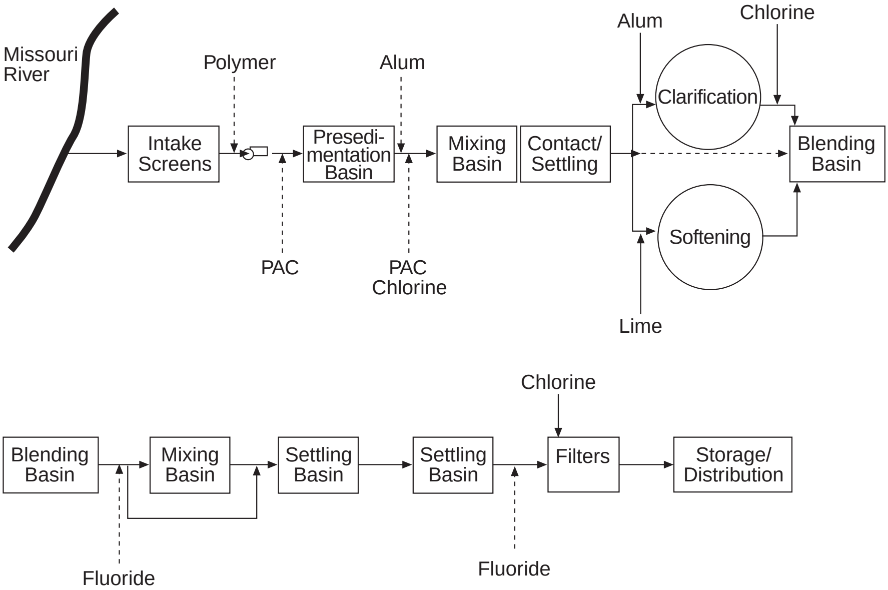


Fig. 17–12. Metropolitan Utilities District, Omaha, Nebraska, water treatment plant flow schematic

mg/L during the spring season. Higher doses of 25–30 mg/L are used during peak events.

PAC facilities include a storage bin receiving truckloads of PAC blown into the storage bin. A large day tank is prepared with a PAC suspension (typically 1.0-lb/gal [0.12 kg/L] concentration) and metered into the flow. The day tank is prepared and density measured each shift. Table 17–4 summarizes the operating criteria for the PAC facilities at this plant.

Real-World PAC Example: Council Bluffs, Iowa

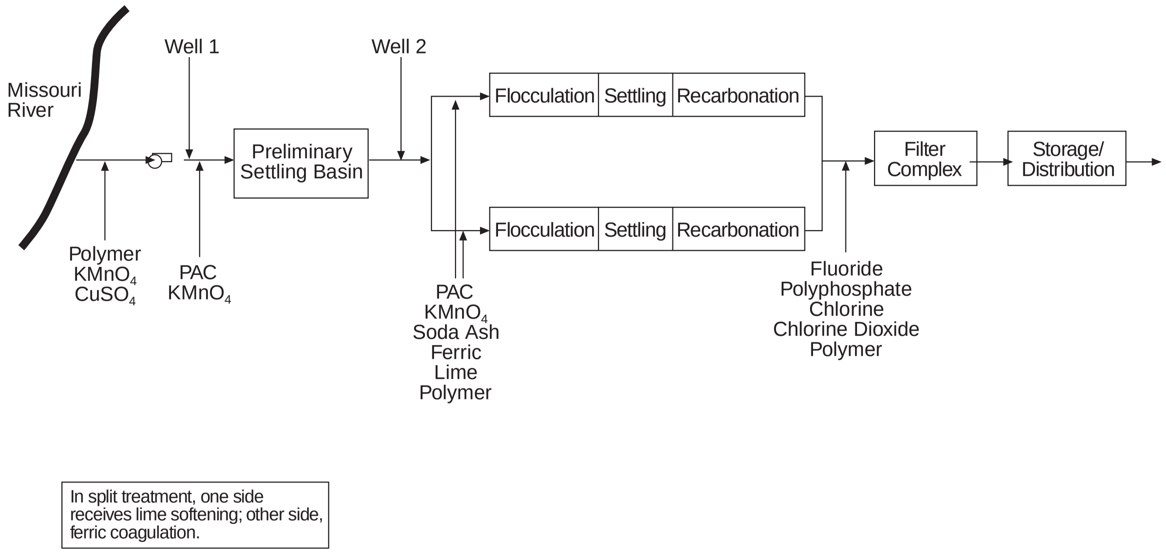
The Narrows Water Treatment Plant of the Council Bluffs Water Works in Council Bluffs, Iowa, operates a 16.9-mgd (64-ML/d) water plant.[33] Two water sources are available: the Missouri River or wells. Figure 17–13 shows a flow diagram of the plant. The water is split-treated with up to 11.2 mgd (42.4 ML/d) lime softened and the remainder alum coagulated to achieve a product water with approximately 160 mg/L hardness. The split depends on the water source (river or wells) and the hardness in the source waters, which varies during the year.

PAC can be added before the presettling basin or with other chemicals in the rapid mix before flocculation. The dose point is selected before the point of lime addition to take advantage of improved adsorption on the PAC at the low pH. Multiple feed points are used at various periods, depending on the softening needs. Generally speaking, the feed point is selected to avoid interference from the high pH during lime softening.

Table 17–5 shows the operating criteria for the PAC facilities at the Florence plant. The main reason for PAC application has historically been for taste and odor control. Recently, PAC has been found to be effective to control agricultural chemicals in runoff, specifically atrazine. PAC doses are increased when the potential for agricultural chemicals in the river increases, such as increases in turbidity due to increased

TABLE 17-4. Operating Criteria for Florence Water Treatment Plant, Omaha, Nebraska

Item	Value
Facilities	
Maximum production	158 mgd (598 ML/d)
Average flow	45 mgd (170 ML/d)
PAC dose	
Typical	6 mg/L
Maximum	30 mg/L
Annual usage	100–150 tons/year (91,000–136,000 kg/year)
PAC metering	
Day tank size	1,500 gal (5,700 L)
Day tank density	1.0 lb PAC/gal (0.12 kg PAC/L)
Metering pump capacity	9,600 gpd (36,000 L/d)
PAC storage	
Bin capacity (total)	8,000 ft^3 (227 m^3)
Typical refill interval	>1 year



In split treatment, one side receives lime softening; other side, ferric coagulation.

Fig. 17–13. Council Bluff Narrows water treatment plant process diagram

TABLE 17–5. Operating Criteria for the Council Bluffs Water Treatment Plant

Item	Parameter
Facilities	
Maximum production	16.9 mgd (64 ML/d)
Average flow	9.2 mgd (34.8 ML/d)
PAC dose	
Typical	25 mg/L
Maximum	40 mg/L
PAC metering	
Day tank size	35,000 gal (132,000 L)
PAC storage	
Bin capacity (total)	70,000 lb (32,000 kg)

runoff in the basin. PAC dose is typically 20 mg/L and is adjusted to maintain atrazine below 50 percent of the maximum contaminant level. Higher doses are used during peak events.

PAC facilities include a storage bin receiving 50-lb (23-kg) bags of PAC. An auger feeder feeds a large day tank to prepare a PAC suspension. The PAC suspension is dosed by using an eductor arrangement to control doseage.

Real-World PAC Example: Sioux Falls, South Dakota

Seasonal taste and odor problems occur in the Sioux Falls, South Dakota, water supply in spring when the snow melts or precipitation events occur. Water quality degradation is a result of high organics and humic materials. Taste and odor problems and organics are treated by adding PAC at various locations throughout the water treatment plant.

The City of Sioux Falls operates a softening/filtration plant rated for 54 mgd (200 ML/d) maximum-day capacity. Two water sources are available: the Big Sioux River or shallow wells in the Big Sioux Aquifer or Skunk Creek Aquifer. Figure 17–14 is a flow schematic of the water treatment plant, and Table 17–6 summarizes the operating criteria for the PAC facilities.

PAC can be added at a static mixer in the plant influent piping, in conjunction with the lime at the solids contact units, or in the recarbonation basins prior to filtration. The dose point in the influent before the point of lime addition could take advantage of improved adsorption in the PAC at the low pH; however, in this case the effectiveness of this particular PAC has not decreased substantially at the high pH experienced in the softening basins. Multiple feed points are used at various periods, depending on the treatment needs.

The reasons for PAC application are organics reduction and taste and odor control. The PAC is fed routinely throughout the year and has effectively reduced complaints. PAC dose is typically 5 mg/L and is adjusted to 20 mg/L or higher during some episodes. Since the city has an alternative groundwater source, they curtail surface water usage during the most severe taste and odor episodes. This step is dependent on the availability of groundwater supplies, surface water quality, and duration of the taste and odor episode.

A slurry tank is used for PAC storage. The slurry storage tankage consists of three 42,000-gal (160 m^3) compartments for a total storage volume of 126,000 gal (480 m^3).

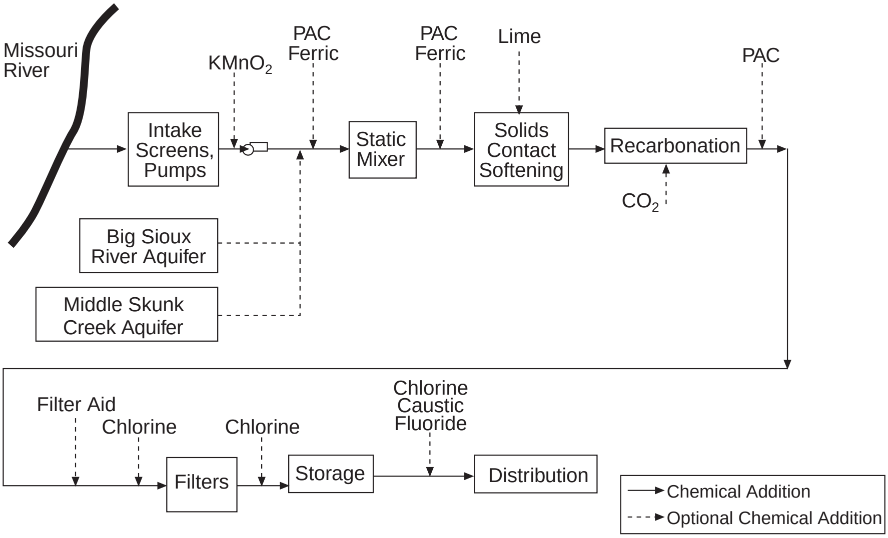


Fig. 17-14. Sioux Falls water treatment plant flow schematic

Each slurry tank has a single two-speed mixer to keep the PAC in suspension. The high speed is used primarily during the addition of dry PAC to the water to provide good wetting and minimize settlement. The slurry tanks have been sized for receiving a complete 40,000-lb (18,000-kg) truck delivery to provide a slurry made up of 1 lb PAC/gal water (0.12 kg PAC/L water). Silos for the storage of the dry PAC are planned to be added above the slurry tanks in the future when actual PAC demand is greater. The system includes PAC transfer pumps and a day tank for dilution to the desired concentration.

Seven metering pumps are available to provide flexibility in PAC feeding. These facilities allow PAC to be fed to several application points: plant influent lines (two),

TABLE 17-6. Operating Criteria for the Sioux Falls Water Treatment Plant

Item	Value
Facilities	
Maximum production	54 mgd (204 ML/d)
Average flow (1997)	19–20 mgd (72–76 ML/d/)
PAC dose	
Typical	5 mg/L
Maximum	20+ mg/L
PAC metering	
Day tank size	1,500 gal (5,700 L)
Metering pump capacity	2–200 gal/hr (7.6–760 L/h)
PAC storage	
Slurry storage capacity (total)	120,000 lb (54,000 kg)
Typical refill interval	30–45 days

the influent to each treatment basin (six), and the recarbonation basins. The metering pumps include flush water to automatically clean the line on pump shutdown.

Real-World GAC Example: Cincinnati, Ohio

The Cincinnati Water Works (CWW) operates the Richard Miller Treatment Plant for the City of Cincinnati, Ohio.[34] The plant treats water from the Ohio River in a 175-mgd (660-ML/d) (maximum-day) water treatment plant that includes coagulation, sedimentation, and filtration before GAC contacting (see Fig. 17–15). Concerns over possible industrial discharges to the river lead to the implementation of GAC as a general-purpose adsorbent at the plant. The GAC facilities were completed in October 1992 at a cost of approximately $60 million.

The treatment plant includes presettling with plate settlers and alum and polymer addition. Coagulation and flocculation with addition of fluoride, lime, ferric sulfate, and chlorine are provided before clarification. Additional chlorine can be added before rapid sand filtration.

The design criteria in Table 17–7 were established following extensive pilot testing to bracket the operational conditions. Since the plant has gone on-line in October 1994, performance has been excellent (see Table 17–8). The operational strategy is to target an effluent TOC of 1 mg/L out of any of the contactors. GAC is regenerated if this TOC level is exceeded.

Carbon usage has proven to be quite difficult to monitor in the various streams. The multiple-hearth furnace is typically operated 7 days a week, 24 hours a day at a feed rate of 40,000 lb/d (18,000 kg/d). Carbon recovery is generally good.

Real-World GAC Example: Manchester, New Hampshire

Lake Massabesic serves as the primary water source for the City of Manchester, New Hampshire, and nearby communities.[35] The city owns a part of the watershed and

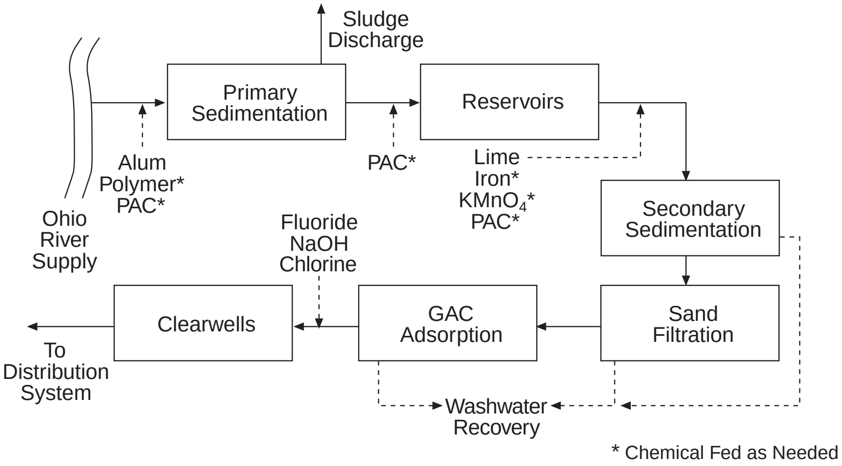


Fig. 17–15. Cincinnati Water Works Richard Miller Treatment Plant process flow schematic

TABLE 17–7. Design Criteria for Cincinnati Water Works GAC Treatment Facilities

Item	Value
Facilities	
Maximum hydraulic capacity	220 mgd (830 ML/d)
Maximum day flow	175 mgd (660 ML/d)
Average flow	124 mgd (470 ML/d)
GAC contactor	
Number	12
Size	30 ft × 65 ft (9.2 m × 19.8 m)
Bed depth	11.4 ft (3.5 m)
Area per contactor	1,950 ft^2 (181 m^2)
GAC volume per contactor	22,230 ft^3 (639 m^3)
Backwash rate	11 gpm/ft^2 (27 m/h)
Bed expansion during backwash	30%
EBCT	
Maximum hydraulic capacity	12.0 minutes
Maximum day flow	15.0 minutes
Average flow	21.2 minutes
Surface loading rate	
Maximum hydraulic capacity	6.9 gpm/ft^2 (17 m/h)
Maximum day flow	5.5 gpm/ft^2 (13 m/h)
Average flow	3.9 gpm/ft^2 (4.5 m/h)
Carbon Regeneration and Handling	
Annual usage	40,000 lb/d (18,000 kg/d)
Peak carbon usage	80,000 lb/d (36,000 kg/d)
Carbon storage vessels*	
Virgin carbon	34,000 ft^3 (960 m^3)
Spent carbon	34,000 ft^3 (960 m^3)
Regenerated carbon	34,000 ft^3 (960 m^3)
Regeneration facilities	
Number of systems	2
Type of regeneration system	Multiple-hearth furnace
Capacity of each system	40,000 lb/d (36,000 kg/d)

*Some storage vessels have duplicate functions.

monitors activities on the watershed to protect the water quality. It operates a 40-mgd (150-ML/d) water treatment plant serving approximately 118,000 customers. Figure 17–16 shows the process schematic of the plant, which was completed in 1974. The water treatment plant includes coagulation, sedimentation, and filtration before GAC contacting.

GAC is used at this plant to provide taste and odor control as well as organics (TOC) and THM control. Taste and odor problems are amplified only during the spring snowmelt, but the agency runs the GAC system year-round to provide high-quality water to its customers. In winter, fewer units are operated. A thermal fluidized bed is used for carbon regeneration. Table 17–9 shows the design criteria for the plant's GAC facilities.

Since the plant went on-line in 1974, performance has been excellent (see Table 17–10). The operational strategy is to target an effluent TOC of 2 mg/L or a THM

TABLE 17-8. Operating Experience at the Cincinnati Water Works GAC Treatment Facilities

Item	Value
Flows	
Maximum-day flow	200 mgd (760 ML/d)
Average flow	120 mgd (450 ML/d)
GAC contactor	
Number	12
EBCT	Varies
Carbon life*	200 days
Backwash interval	180 days (after a regeneration)
Backwash rate	11 gpm/ft^2 (27 m/h)
Backwash time	60 minutes
Carbon usage and regeneration	
Carbon regeneration	40,000 lb/d (18,000 kg/d)
Regeneration system operation	7 days/week 24 hours/day
Makeup carbon added†	7–8% per regeneration

* Estimate.

† Approximately 1 million lb/year (450,000 kg/year) in 1997.

of <40 g/L. The carbon life is typically between 20 and 30 months. The fluidized bed is typically operated continuously at a feed rate of 17,000 lb/d (7,700 kg/d) on an as-needed basis. At the current carbon life of 20–30 months, this will require 30–40 days per year continuous operation. Carbon use in the early 1980s was higher than it is at present. Carbon recovery is generally good; approximately 10 percent carbon supplement is required.

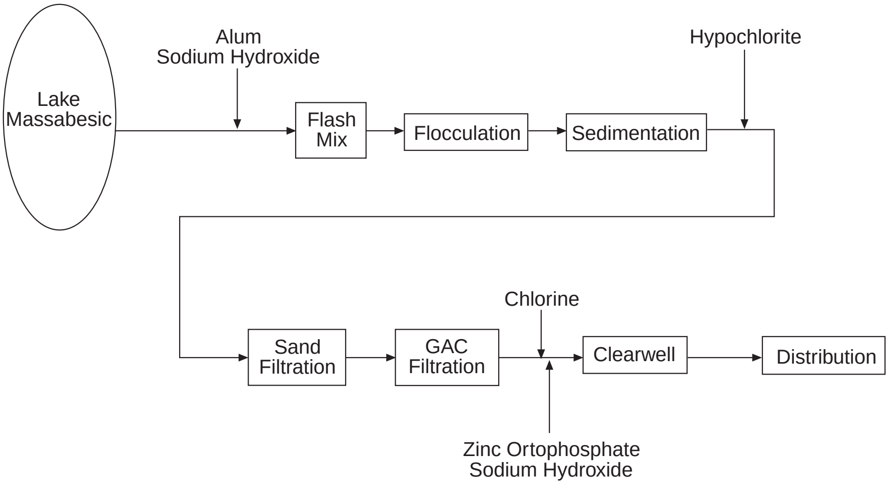


Fig. 17-16. Manchester Water Works treatment plant schematic

TABLE 17–9. Design Criteria for Manchester Water Treatment Facilities

Item	Value
Facilities	
Maximum hydraulic capacity	40 mgd (150 ML/d)
Maximum-day flow	30 mgd (110 ML/d)
Average flow	16 mgd (61 ML/d)

TABLE 17–10. Operating Experience for Manchester Water Treatment Plant GAC facilities

Item	Value
Flows	
Maximum-day flow	30.5 mgd (115 ML/d)
Average flow	16 mgd (61 ML/d)
GAC contactor	
Number	4
EBCT	15 minutes
Carbon life	20–30 months
Backwash interval	1 day
Backwash rate	About 20 gpm/ft^2 (50 m/h)
Backwash time	1 minute
Carbon usage and regeneration	
Carbon usage	250,000 lb/year (113,000 kg/year)
Regeneration system operation	7 days/week 24 hours/day
Makeup carbon added	About 30,000 lb/year (13,600 kg/year)

REFERENCES

1. Westerhoff, G. P., Arora, H., and Cline, G., ''The Roles of GAC in Drinking Water Treatment,'' in *Proceedings AWWA Seminar on Engineering,* American Water Works Association, Denver, Colo., 1990.
2. AWWA (American Water Works Association), *Activated Carbon in Drinking Water Technology,* AWWA, Denver, Colo., 1983.
3. Summers, R. S., and Sontheimer, H., *German Experiences with Granular Activated Carbon,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1989.
4. U.S. EPA (U.S. Environmental Protection Agency), *Process Design Manual for Carbon Adsorption, EPA Technology Transfer,* USEPA, Cincinnati, Ohio, 1973.
5. Wulfeck, W. M., Jr., and Summers, R. S., ''Control of DBP Formation Using Retrofitted GAC Filter-Adsorbers and Ozonation,'' in *Proceedings 1994 Water Quality Technology Conference,* American Water Works Association, Denver, Colo., 1994.
6. Weber, W. J., Jr., and Pirabazari, M., ''Adsorption of Toxic and Carcinogenic Compounds from Water,'' *JAWWA* 74(4):203–209, 1982.
7. Anderson, M. C., Butler, R. C., Holdren, F. J., and Kornegay, B. H., ''Controlling Trihalomethanes with PAC,'' *JAWWA* 73(8):432, 1981.

8. Quinn, J. E., and Snoeyink, V. L., ‘‘Removal of Total Organic Halogen by GAC Adsorbers,’’ *JAWWA* August:483, 1980.
9. Summers, R. S., Hooper, S. M., Solarik, G., Owen, D. M., and Hong. S., ‘‘Bench Scale Evaluation of GAC for NOM Control,’’ *JAWWA* 87(5):69–80, 1995.
10. Calgon Carbon Corporation, *Activated Carbon Principles,* Technical Information Bulletin, Calgon Carbon Corporation, Pittsburgh, PA, 2000.
11. Randtke, S. J., and Snoeynik, V. L., ‘‘Evaluating GAC Adsorptive Capacity,’’ *JAWWA* 75(8): 406–413, 1983.
12. AWWA (American Water Works Association), *AWWA Standard for Granular Activated Carbon,* ANSI/AWWA B604-90, AWWA, Denver, Colo., 1991.
13. AWWA (American Water Works Association), *AWWA Standard for Powdered Activated Carbon,* ANSI/AWWA B600-90, Denver, Colo., AWWA, 1991.
14. Qi, S., Snoeyink, V. L., Beck, E. A., Koffskey, W. E., and Lykins, B. W., Jr., ‘‘The Use of Isotherms to Predict GAC Performance for Synthetic Organic Chemical Removal at Jefferson Parish, LA,’’ in *Proceedings of AWWA Conference,* American Water Works Association, Denver, Colo., 1991.
15. McGuire, M. J., Davis, M. K., Liang, S., Crittenden, J. C., and Vaith. K., *Optimization and Economic Evaluation of Granular Activated Carbon for Organic Removal,* American Water Works Association Research Foundation and American Water Works Association, Denver, CO, 1989.
16. Faust, S. D., and Aly. O. M., *Adsorption Processes for Water Treatment,* Butterworths, Stoneham, Mass., 1987.
17. AWWA (American Water Works Association), ‘‘Water Stats: The Water Utility Database,’’ AWWA, Denver, Colo., 1998.
18. Koffskey, W. E., and Lykins, B. W., Jr., ‘‘Experiences with Granular Activated Carbon Filtration and On-Site Reactivation at Jefferson Parish, LA,’’ in *Proceedings of AWWA Conference,* American Water Works Association, Denver, Colo., 1987.
19. Culp, R. L., and Clark, R. M., ‘‘Granular Activated Carbon Installations,’’ *JAWWA* 75(8): 398–405, 1983.
20. Westerhoff, G. P., ‘‘GAC—The Cincinnati Experience: Facility Design Considerations and Project Costs,’’ in *AWWA Conference Proceedings,* American Water Works Association, Denver, Colo., 1994.
21. Culp, R. L., Faisst, J. A., and Smith, C. E., *Granular Activated Carbon Installations, Final Report,* Contract No. CI-76-0288, U.S. Environmental Protection Agency, Municipal Environmental Research Laboratory, Cincinnati, Ohio, 1982.
22. McCarty, P. L., Argo, D., and Reinhard, M., ‘‘Operational Experiences with Activated Carbon Adsorbers at Water Factory 21,’’ *JAWWA* 71(11):683–689, 1979.
23. Kornegay, B. H., ‘‘GAC Handling and Transport Systems,’’ in *Proceedings of AWWA Conference,* American Water Works Association, Denver, Colo., 1990.
24. Nickols, D., ‘‘Engineering Considerations for GAC Treatment Facilities: European Design Experience,’’ in *Proceedings of AWWA Conference,* American Water Works Association, Denver, Colo., 1990.
25. Neethling, J. B., and Culp. G. L., ‘‘Capital and Operating Costs of GAC Facilities,’’ in *Proceedings of AWWA Conference,* American Water Works Association, Denver, Colo., 1990.
26. Crittenden, J. C., Berrigan, J. K., Jr., and Hand, D. W., ‘‘Design of Rapid Small-Scale Adsorption Tests for a Constant Surface Diffusivity,’’ *J. WPCF* 58(4):317–319, 1986.
27. Crittenden, J. C., Berrigan, J. K., Hand, D. W., and Lykins, B., ‘‘Design of Rapid Fixed Bed Adsorption Tests for Non-Constant Diffusivities,’’ *J. Environ. Eng.* 113(2):243–259, 1987.
28. Crittenden, J. C., Reddy, P. S., and Hand. D. W., *Prediction of GAC Performance Using Rapid Small-Scale Column Tests,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1989.

29. Krasner, S. W., Sclimenti, M. J., and Coffey, B. M., ‘‘R&T Testing Biological Active Filters for Removing Aldehydes Formed during Ozonation,’’ *JAWWA* 85(5):62, 1993.
30. Culp, R. L., and Hansen. S. P., ‘‘Carbon Adsorption Enhancement with Ozone,’’ *J. WPCF* 52(2):270, 1980.
31. Price, M. L., *Ozone and Biological Treatment for DBP Control and Biological Stability,* American Water Works Association Research Foundation and American Water Works Association, Denver, Colo., 1994.
32. Shields, J., personal communication, Metropolitan Utilities District, Omaha, Neb., 1998.
33. Elliot, J., personal communication, Council Bluffs Water Treatment Plant, Council Bluffs, Ia., 1998.
34. Moster, L., personal communication, Cincinnati Water Works, Cincinnati, Ohio, 1998.
35. Paris, D., personal communication, Manchester Water Works, Manchester, N.H., 1998.

CHAPTER 18

Ion Exchange and Activated Alumina Sorption

INTRODUCTION

The sorption of soluble species at liquid-solid interfaces is of great importance in water treatment. For some inorganic species, the only practical means of removal is via an adsorptive/absorptive mechanism. This chapter deals with sorptive processes driven by electrochemical charge interaction, including ion exchange and sorption onto activated alumina. Examples of water constituents that can be removed via sorptive processes include hardness, nitrate, fluoride, and arsenic. We begin with a discussion of ion-exchange processes and mechanisms.

ION EXCHANGE

Fundamentals

Ion exchange is defined as a process whereby an insoluble substance removes ions of positive or negative charge from an electrolytic solution and releases other ions of like charge into solution in a chemically equivalent amount. The process occurs with no structural changes in the resin. The ions in solution rapidly diffuse into the molecular network of the resin, where exchange occurs. The exchanged ions proceed by the reverse path into solution. At some point during the ion-exchange process an ion-exchange equilibrium is established.

The equilibrium reaction for the exchange of ions A and B on a cation exchange resin is often represented as follows:

$$nR^-A^+ + B^{n+} \Leftrightarrow R_n^-B^{n+} + nA^+$$

where R^- is an anionic group attached to the ion exchange resin, and A^+ and B^{n+} are ions in solution.

In relatively dilute solutions of ions, the equilibrium constant expression for this reaction becomes:

$$K''^{B^{n+}}_{A+} = \frac{[R_n^-B^{n+}]_R[A^+]_S^n}{[R^-A^+]_R^n[B^{n+}]_S}$$

where

$[R_n^- B^{n+}]_R$ = concentration of B^{n+} in the resin
$[B^{n+}]_S$ = concentration of B^{n+} in the solution
$[R^- A^+]_R$ = concentration of A^+ in the resin
$[A^+]_S$ = concentration of A^+ in the solution

The term K_{A+}^{BN+} is not actually a constant, since it is dependent on experimental conditions. Rather, it is referred to as a selectivity coefficient and it is useful for determining which ions will exchange in reasonable amounts and for estimating the amount of resin required to remove some quantity of an ion from solution.

On the basis of selectivity coefficients, relative affinities of ions for an ion exchanger can be quantitatively evaluated (see Table 18–1). This suggests that an order can be established for ions of the same valence on the basis of their selectivity coefficient. The affinity of an ion for a resin can be generalized by the following rules:

1. In general, ions of high valence are preferred over ions of low valence; i.e., the extent of the exchange reaction increases with increasing ion valence, for example:

$$Fe^{3+} > Mg^{2+} > Na^{+} \quad \text{and} \quad PO_4^{3-} > SO_4^{2-} > NO_3^{-}$$

TABLE 18–1. Relative Affinity of Various Cations and Anions on Polystyrene-Based Exchange Resins

Cation	Affinity for Strong Acid Resin*	Anion	Affinity for Strong Base Resin*
H^+†	1.0	OH^- (reference)†	1.0
Li^+	0.81	$SO_4^=$	70
Na^+	1.7	Salicylate	65
NH_4^+	2.3	Citrate	23
K^+	3.0	ClO_3^-	12
Cs^+	3.2	NO_3^-	8
Cu^+	9.5	Br^-	6
Ag^+	12.0	NO_2^-	3
Mn^{++}	2.5	Cl^-	2.3
Mg^{++}	2.6	HCO_3^-	1.2
Fe^{++}	2.7	Formate	0.5
Zu^{++}	2.8	Acetate	0.5
Cu^{++}	3.1	F^-	0.3
Cd^{++}	3.3		
Ca^{++}	4.6		
Sr^{++}	6.2		
Hg^{++}	9.7		
Pb^{++}	10.1		
Ba^{++}	11.6		

Source: After Abrams and Benezra (see Reference 1).

* 12% cross-linked diviny/benzene.
† Affinities relative to the hydrogen ion

* Reactive group —$CH_2N^+(CH_3)_2C_2H_4C_2H_4OH$.
† Affinities relative to the hydroxyl ion

2. For ions of the same valence, the extent of the exchange reaction increases with decreasing hydrated radius and increasing atomic number, for example:

   $$Ca^{2+} > Mg^{2+} > Be^{2+} \quad \text{and} \quad K^{+} > Na^{+} > Li^{+}$$

   This type of response is a result of swelling pressure within the resin. Ions of larger hydrated radius increase the swelling pressure within the resin and decrease the affinity of the resin for such ion.
3. For a solution with a high total ionic concentration, the extent of the exchange reaction follows no general rule and is often reversed. This type of response is the basis for the reversibility of regeneration.
4. The relationship between the degree of cross-linking and the size of the hydrated ion may affect the extent of the exchange reaction. If the resin has a high degree of cross-linking, the ion may be too large to penetrate into the matrix of the resin.

In general, it is advantageous to use a resin with a high affinity for the ion to be exchanged. High affinities improve the kinetics of adsorption and allow higher hydraulic loading rates. Also, the greater the affinity, the sharper the breakthrough curve, and hence the shorter the ion-exchange column. The only disadvantage of a high affinity is that a higher regenerant concentration will be required.

Types of Ion Exchange Resins

The first ion-exchange materials were naturally occurring zeolite clays. Today, most ion-exchange resins are synthetic materials made up of a polymeric matrix (generally polystyrene chains held together by divinylbenzene cross-links), with soluble ionic functional groups attached to the polymer chain. The number and kind of functional groups determine the exchange capacity and ion selectivity, whereas the polymer matrix determines the durability and toughness of the resin. Resins are granular in nature and may have either a spherical or irregular shape. Although spherically shaped resins are generally used, the irregularly shaped form provides a larger surface, and often a lower void volume, which increases the exchange capacity per unit of volume.

Ion-exchange resins are usually classified in the following manner:

Cation-Exchange Resins (contain exchangeable cations):

- Strong-acid exchange resins
- Weak-acid exchange resins

Anion-Exchange resins (contain exchangeable anion):

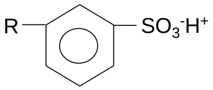
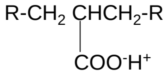
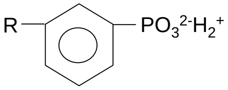
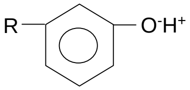
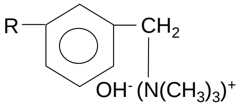
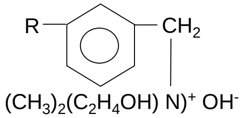
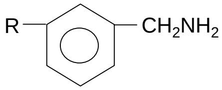
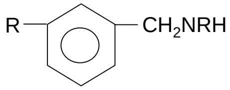
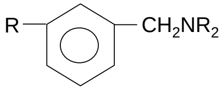
- Strong-base exchange resins
- Weak-base exchange resins

Strong-acid exchange resins contain functional groups derived from a strong acid (normally sulfuric acid). Their degree of ionization is analogous to that of a strong acid (low pKa), which permits the hydrogen to be dissociated and ready for exchange over a wide pH range. Weak-acid exchange resins, on the other hand, contain functional groups derived from a weak acid, commonly of the carboxylic or phenolic form. Such resins are useful only within a fairly narrow pH range.

Functional groups on strong-base exchange resins are usually quaternary ammonium groups, whereas weak-base exchange resins contain either primary, secondary, and/or tertiary amines as the functional group. The strong-base exchange resins are useful over a wide pH range, whereas the weak-base exchange resins are effective only within a fairly narrow pH range.

The active exchange groups associated with each of the different kinds of resins are listed in Table 18–2, along with their basic structure and dissociation constant. The

TABLE 18–2. Chemical Classification and Performance Parameters of Common Ion-Exchange Resins

Classification	Active Groups	Dissociation Constant pKa	Structure
Cation Exchange Resins Strong acid	Sulfonic	1	
Weak acid	Carboxylic	4–6	 $R\text{-}CH_2\,\underset{COO^-H^+}{CH}CH_2\text{-}R$
	Phosphonic	2–3	
	Phenolic	9–10	
Anion Exchange Resins Strong base	Type I—quartenary ammonium	13	
	Type II—quartenary ammonium	13	
Weak base	Primary amine	6–9	
	Secondary amine	7–9	
	Tertiary amine	9–11	

Source: See Reference 2.

dissociation constants (pKa) are important relative to the operable pH range of the resin.

Resin Characteristics

***Moisture-Retention Capacity* (*Moisture Content*)** When placed in water, ion-exchange resins swell, and ions are able to diffuse into and out of their structure. The extent of swelling is a function of the degree of cross-linking. A low degree of cross-linking is associated with high moisture content (high moisture-retention capacity), a high level of resin swelling in an aqueous solution, and a low total exchange capacity. Because of the high level of resin swelling, resins that have a low degree of cross-linking undergo large contractions and expansions during regeneration and service (as much as 25 percent). This must be considered when designing containing systems.

In contrast, resins that have higher degrees of cross-linking have higher exchange capacities per unit volume, lower moisture contents, and lower levels of swelling.

Density Ion-exchange resin density may be expressed as bulk density (weight per unit total volume—i.e., the sum of the particle volume and the void volume) or as specific gravity weight per unit particle volume). Bulk density values reported by resin manufacturers are usually for a fully hydrated resin after backwashing, settling, and draining. These values are used for shipping purposes, whereas specific gravity values are used for calculating acceptable resin backwash rates and resin settling rates.

Ion-Exchange Operation

Ion exchange can be operated in one of four modes: batch, fixed-bed, fluidized, and continuous. The fixed-bed system is by far the most common and is the one system considered in this text. A typical fixed-bed operating cycle consists of four steps:

- Service or operation cycle
- Backwash
- Regeneration
- Rinsing

Service Cycle The desired exchange reaction occurs during the service cycle. To illustrate the behavior of ion exchangers during the service cycle, consider the phase where a strong-acid cation-exchange resin in the hydrogen form is loaded in a fixed-bed reactor column and used to remove sodium ions from solution passing through it (see Fig. 18–1). What occurs in the column is a kind of multistage treatment where the solution passing down through the column is repeatedly brought into contact with fresh resin. After a short period of operation, the upper part of the resin bed becomes exhausted. The applied sodium chloride solution passes unchanged though this part of the bed, but farther into the bed it enters into the exchange zone, where the sodium ions displace the hydrogen ions from the resin. The displaced hydrogen ions exit the column at a concentration equivalent to the sodium concentration in the influent. As the column operation continues, the exchange zone moves down the column until it reaches the bottom. At this point, sodium ions show in up the effluent—that is, breakthrough occurs. The plots to the left and reight of center represent Na^+ and H^+ ion

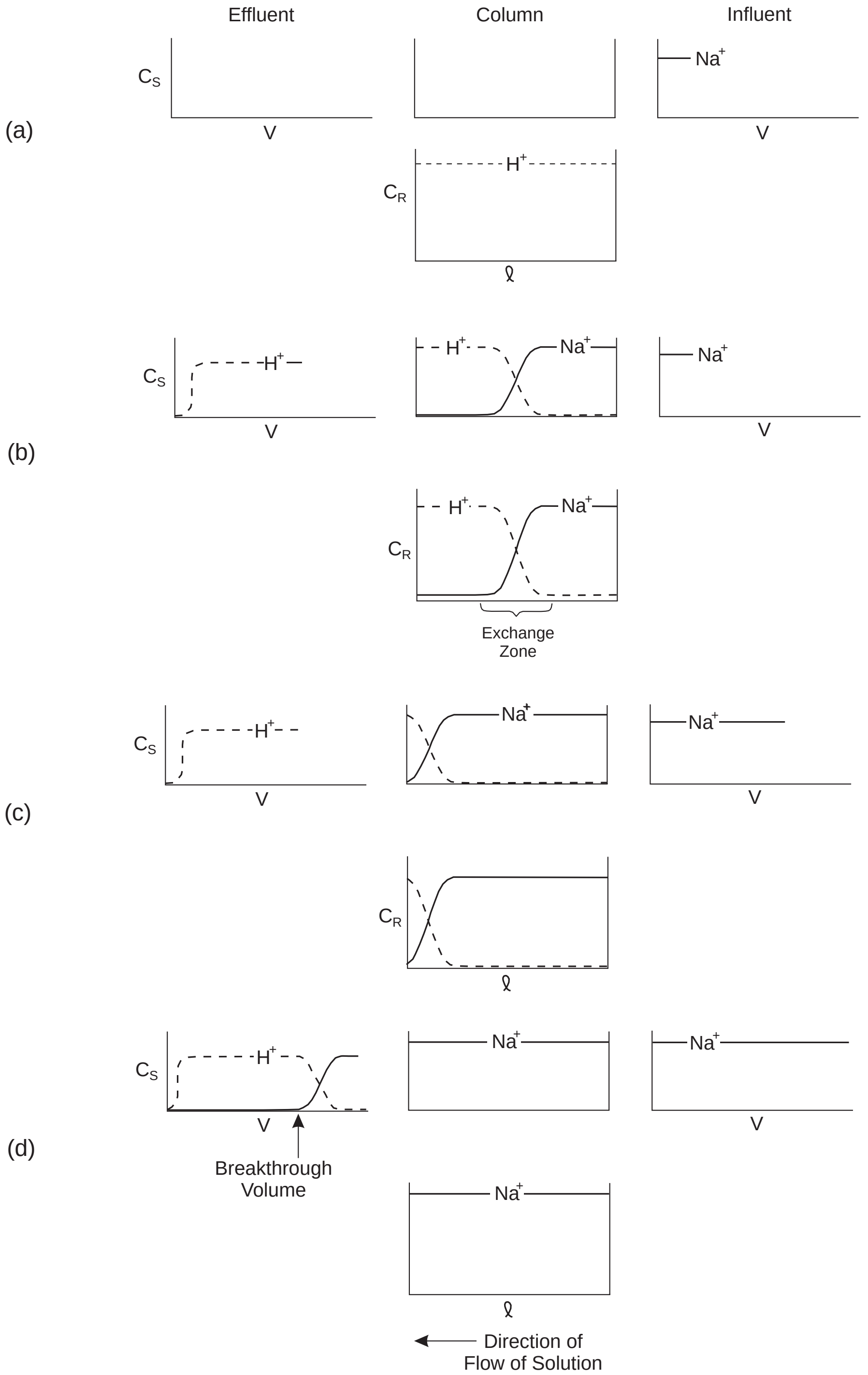


Fig. 18–1. Behavior of ion-exchange columns—displacement of H^+ ions by Na^+ ions in a cation-exchange column (*Source:* See Reference 3)

concentrations versus volume of solution passed through the column for the effluent and influent, respectively. The center plots illustrate the movement of the exchange zone through the depth of the column.

The shape of the front of the exchange zone as it moves through the column is very important in column operation, because it determines the utilization effieciency of the resin. The closer the shape of the front approaches that of a vertical line, the more efficient the resin utilization. The shape of the front is influenced by flow rate and concentration of the feed solution. If the ions initially attached to the resin are less strongly attracted to the resin than the ions in the feed solution, the front of the exchange zone will approach that of a vertical line. On the other hand, if the ions initially attached to the resin are more strongly attracted to the resin than the ions in the feed solution, the shape of the front of the exchange zone is diffused. In general, it can be said that the difference in total resin capacity and operating capacity increases as the value of the selectivity coefficient for the exchange reaction decreases (i.e., the breakthrough curve broadens as the selectivity coefficient for the exchange reaction decreases).

During the service cycle, certain ions are removed selectively over other ions. This ion selectivity can produce situations that are at first perplexing. Nitrate removal is a good example. Because of the selectivity of sulfate over nitrate, sulfate tends to be removed before nitrate, and in a downflow operating mode produces an upper ‘‘layer’’ of resin exhausted by sulfate and a lower ‘‘layer’’ of resin exhausted by nitrate. During the service cycle, each of these layers moves progressively downward through the resin bed. When the nitrate removal capacity of the resin is exhausted, nitrate appears in the effluent. If operation continues beyond this point, the nitrate in the exchanger product water will soon exceed the raw-water nitrate concentration. Thus, the nitrate removal of the resin is strongly influenced by the sulfate, nitrate, and total anion concentrations.

Backwashing Backwashing with product water is employed after the operating capacity of the ion exchanger has been reached. This is an upflow process used to prepare the resin for regeneration. Backwashing has several purposes: (1) break up resin clumps, (2) remove finely divided suspended material trapped in the resin by filtration, (3) eliminate gas pockets, and (4) restratify the resin bed to ensure a uniform distribution of flow during downflow operations.

Regeneration Regeneration displaces ions exchanged during the service run and returns the resin to its initial exchange capacity or to any other desired level, depending on the amount of regenerant used. In general, mineral acids are used to regenerate cation resins, and alkalies are used to regenerate anion resins. To minimize the regeneration time and the amount of regenerant used, the regenerant should provide a maximum peak elute concentration with minimum ‘‘tailing’’ of the elute. If the system is restored to its initial capacity, the number of equivalents of ions eluted from the resin during regeneration should equal the number of equivalents exchanged during the service cycle.

The regeneration efficiency is defined as the ratio of the total equivalents of ions removed from a resin to the total equivalents of ions present in the volume of regenerant used. Generally, the resin can be restored to full capacity by eluting all exchangeable ions. However, in many cases this may require that a large amount of regenerant be used, which can prove to be very costly. As a result, only a portion of the available exchange capacity is normally restored during the regeneration cycle. The extent of regeneration is referred to as the *regeneration level*.

The regeneration efficiency is higher for weak ionic resins than for strong ionic resins, because the weak ionic resins' affinity is higher for the H^+ and OH^- ions. This means that regeneration is more favorable for weak ionic resins, with the result that less regenerant is required to achieve the same degreee of exchange. This can be explained by considering that the value of the selectivity coefficient for the regeneration reaction is the reciprocal of the selectivity coefficient for the initial exchange reaction.

Rinsing After the regeneration step, the ion-exchange resin must be rinsed free of excess regenerant before being put back into operation. The rinsing procedure consists of using a slow and fast rinse with product water. The slow rinse of one bed volume displaces regenerant, and the waste from this rinse is combined with the regenerant brine for disposal. The fast rinse washes away excess ions, and the waste from that rinse is often collected and used for regenerant dilution water.

Brine Disposal

Potential techniques for brine disposal are discharge to sanitary sewers, evaporation ponds (lined or unlined), the ocean or an estuary, or disposal wells. In all cases, the disposal technique must be approved by regulatory agencies, and, in the case of sewer disposal, by the sewering agency. Particular attention must be given to degradation of groundwater and surface water quality. Chlorination may be necessary in some cases.

The cost of brine disposal is a key factor in the overall economic analysis that should be made to select the treatment technique.

Pilot Testing

Use of scale-model ion-exchange reactors is recommended to establish design and operational parameters for the particular water supply being evaluated. Pilot testing will help in optimization of design parameters, and in large systems may avoid costly errors in treatment process design.

In smaller systems where the cost of pilot testing may not be economically justified, the knowledge and experience of ion-exchange equipment manufacturers and resin suppliers is a valuable resource. Many manufacturers will analyze representative water samples and will recommend design criteria and resin type. The cost for such services is usually minimal. A prudent measure is to obtain recommendations from at least two manufacturers.

Softening Applications

Softening is the most commonly used ion-exchange application in water treatment. The resins of choice are strong-acid, cation-exchange resins in the sodium form. Typical resins are Amberlite IR-120 Plus, Duolite C-20, Dowex HCR, and Permutit Q-100.

The resin can be contained in a pressure steel vessel or an open concrete basin. Pressure vessels are most commonly used in small plants, whereas open concrete basins are usually in larger facilities.

Operating Conditions Suggested conditions for each of the operating phases are shown in Table 18–3 for three different resins. The four operations (treatment, backwash, regeneration, and rinsing) are discussed below.

Treatment. Treatment may be either upflow or downflow, although downflow treatment is the most common in water treatment applications. The service flow rate will vary between 2 and 5 gpm/cu ft, depending on the type of resin used (see Table 18–3) and the raw-water hardness. The length of the service cycle is a function of the hardness concentration in the raw water and the exchange capacity of the resin. Manufacturers typically express resin capacity in terms of kilograins of hardness (as $CaCO_3$) per cubic foot of resin. The conversion factors for hardness ions are listed in Table 18–4.

Amberlite IR-120 Plus has an exchange capacity between 17.5 and 34.5 kilograins of hardness (as $CaCO_3$) per cubic foot of resin. The actual capacity depends upon the regenerant level used. The more regenerant used per cubic foot of resin, the greater the exchange capacity of the resin. However, the overall regenerant cost per unit of hardness removed also increases. The relationship between regenerant level and exchange capacity is shown in Table 18–5. In water treatment applications, 10 lb NaCl/cu ft of resin represents a reasonable compromise between regenerant usage and regeneration efficiency.

Pretreatment is required in many situations. Suspended solids should be low (zero if possible), and turbidity should be less than 1 NTU, to prevent bed plugging. Ferrous iron should be removed prior to ion-exchange treatment, as it can oxidize to the ferric form within the bed if oxygen is present.

If water to be treated is a well water containing no dissolved oxygen, the resin can effectively remove ferrous and manganous ions. Conversely, if oxygen is present, or if iron or manganese is present in the oxidized form, it should be removed prior to ion-exchange treatment, to avoid resin fouling. Chlorine should not be present in the feedwater.

During conventional downflow treatment, there is a pressure drop across the resin, which is a function of the service flow rate, water temperature, and amount of suspended material in the raw water. When suspended matter is present, it is frequently filtered out by the resin, thereby reducing the void volume in the upper portion of the bed and causing increased resistance to flow. In extreme cases, resin particles can shatter, producing fine particles that are lost during backwash. Although some suspended material may be tolerated, it is wise to employ a resin bed as an ion-exchange medium and not as a filtering mechanism.

Organic material in the feedwater can also result in resin fouling. The fouling is caused by deposition within the resin, as well as by bacterial growth within the resin bed.

Backwash. Following completion of the treatment or operation cycle, the resin bed should be backwashed for approximately 10 minutes at a rate that will cause a bed expansion of between 50 and 75 percent. The goal of backwashing is to remove material that was filtered out by the bed in its upper layers. If this material is not removed during backwashing, it will lead to channeled water flow through the bed, premature leakage of hardness, and reduced exchange capacity.

Regeneration. Regenerant brine concentration has little effect on resin-exchange capacity. For example, Amberlite IR-120 resin shows only a 7 percent decrease in

TABLE 18–3. Suggested Operating Conditions for Amberlite IR-120 Plus, Duolite C-20, and Permutit Q-100 in the Sodium Cycle

Parameter	Units of Expression	Amberlite IR-120 Plus	Duolite C-20	Permutit Q-100
Service flow rate	gpm/cu ft	2	2–5	2–4
Exchange capacity	kilograins/cu ft of resin inches	17.5–34.5	18–37	41.4
Minimum bed depth	inches	24	24	24
Backwash flow rate*	gpm/sq ft	6 @ 72°F	6–8 @ 50–75°F	—
Backwash water requirement†	gal/cu ft of resin	30	10–50	—
Regenerant concentration	% NaCl	10	10–20	10–14
Regenerant flow rate	gpm/cu ft	0.5–1.0	0.2–1.0	0.5
Regenerant level	lb NaCl/cu ft of resin	5–30	5–20	5.5–16
Rinse flow rate	gpm/cu ft	1.0 initially, then 1.5	1.0 initially, then up to 5.0	—
Rinse water requirement	gal/cu ft of resin	25–75	10–50	25–50

Source: See Reference 4.

*50–75% bed expansion required.

†Based on 2-foot bed depth.

TABLE 18–4. Hardness Conversion Factors

Ion	Multiplier to Convert from mg/L as the Ion to grains/gal (as $CaCO_3$)
Ca^{+2}	0.146
Mg^{+2}	0.240
Fe^{+2}	0.105
Mn^{+2}	0.106

capacity when a 4 percent sodium chloride solution is used instead of a 10 percent solution. A much more critical factor is the regenerant flow rate, as lower flow rates give longer contact time between the brine and the resin. Regenerant flow rates in excess of 1 gpm/cu ft are not recommended, as appreciable capacity reduction is experienced at higher flow rates. Regeneration is typically in a downflow manner, with a minimum contact time of 30 minutes.

Rinsing. The initial rinse flow rate should be the same as that used during regeneration; rinse water is introduced while the regeneration process is concluding. After one to two bed volumes of rinse water have been added, the flow rate should be increased to 1.5 gpm/cu ft. This should continue until the sodium concentration in effluent from the bed is less than 5 mg/L above the rinse water sodium concentration. These two rinse operations are referred to as the *slow rinse* and the *fast rinse*.

Design Considerations

Exchange Vessels. Small plants generally use pressure units, as this allows pumping directly through the exchanger and into the distribution system. Larger plants typically use open, gravity-flow units, as they are less costly than pressure units per unit volume.

Pressure vessels (Fig. 18–2) may be either vertical or horizontal, with vertical vessels preferred. Disadvantages of horizontal vessels include poor hydraulic distribution and loss of exchange capacity because resin located in the ‘‘bulges’’ at the ends and side of the pressure vessels is not fully utilized. The vessel interior should be coated to provide protection against the corrosive sodium chloride regenerant. All piping should be corrosion-resistant, with plastic pipe most frequently utilized.

Resin Depth. The minimum resin depth is 2 feet, and the maximum resin depth 5 feet. The maximum resin depth is established by allowable headloss through the bed and by expanded bed depth during backwashing. Expansion of the resin bed by 50 to 75 percent during backwash should be allowed in the design, without resin reaching the bottom of the wash troughs. If sufficient contactor volume is not left for resin expansion, resin will be lost during backwash.

Sizing the Contactors. Contactor size is a function of several parameters:

- Resin volume
- Maximum raw-water hardness
- Time interval between regenerations
- Product water storage

TABLE 18–5. Relationship of Regenerant Level, Resin Exchange Capacity, and Regeneration Efficiency for Amberlite IR-120 Plus

Regenerant Level		Resin Exchange Capacity (as $CaCo_3$)		Regeneration Efficiencies	
lb NaCl per cu ft of Resin	kg NaCl per m^3 of Resin	Kilograins per cu ft	Kilograins per m^3	lb NaCl per Kilograins Removed	kg NaCl per Kilograins Removed
5	80.2	17.5	618	0.28	0.127
10	161	24.5	866	0.41	0.186
15	241	29.0	1,025	0.51	0.232
20	321	32.5	1,148	0.62	0.282
25	401	34.0	1,202	0.74	0.336
30	481	34.5	1,219	0.87	0.395

Source: See Reference 5.

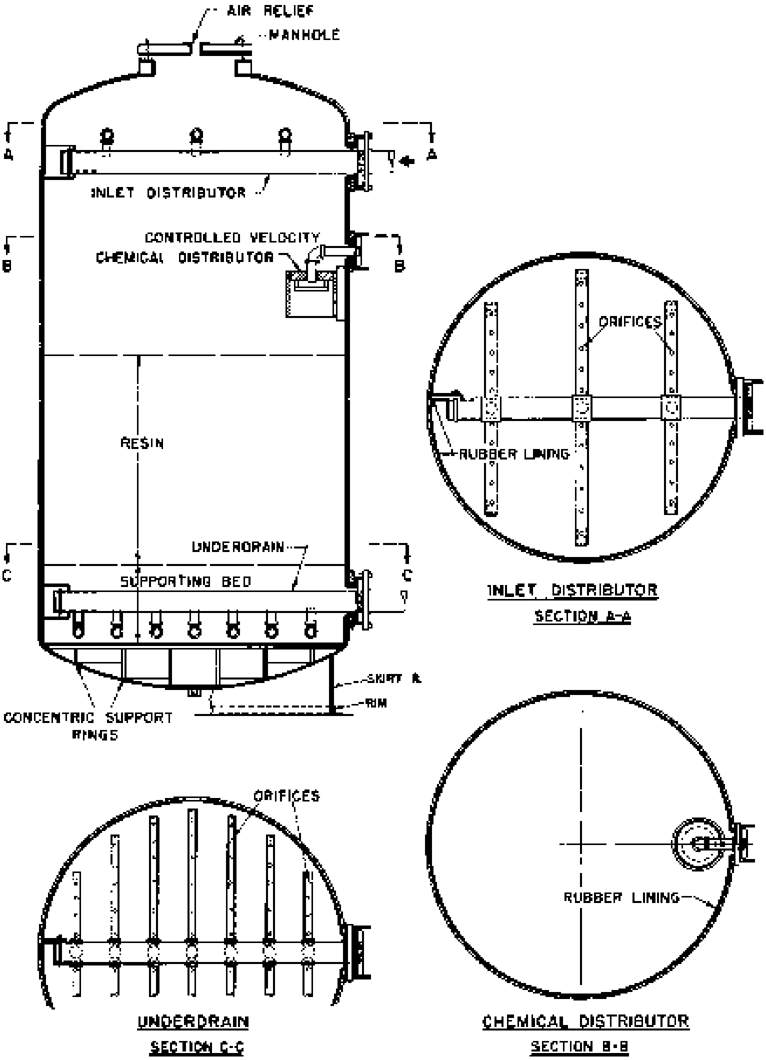


Fig. 18–2. Cut-away of typical ion-exchange pressure vessel (Courtesy of Infilco Degremont, Inc.)

Resin volume can be varied by changing the bed depth and the number of contactors. Because maximum raw-water hardness is not within the control of the plant designer, the only remaining variable is regeneration frequency. If the plant is automatically operated, a minimum regeneration frequency of 8 to 12 hours is suggested. However, in manually operated plants that have one shift per day, the frequency should be every 24 hours. Sufficient product water storage to “average out” the reduction in product water flow during contactor regeneration is desirable. Without storage, the treatment system must be designed to continuously meet distribution system demands even with one contactor out of service for regeneration.

The required resin volume is calculated according to the formula:

$$V = \frac{QHT100}{E24\%OP}$$

$$V = \frac{QHT}{240E\%OP} \quad \text{(metric)}$$

where

V = required resin volume, cu ft (m^3)
Q = flow, gal/day (L/d)
H = raw-water hardness, kilograins/gal as $CaCO_3$ (mg/L)
T = regeneration frequency, hours (h)
E = resin exchange capacity, kilograins as $CaCO_3$/cu ft (g/m^3)
$\%OP$ = average percent of time that contactors are operating, as %

Underdrains. Similar to a sand filter, the underdrain collects water during operation, distributes backwash water, and collects brine and rinse water during and following regeneration. Good flow distribution is essential if each of the four phases of the cation exchange process is to be performed efficiently. The total area of the holes in the underdrain should be 0.16 to 0.18 percent of the surface area of the exchanger. Porous plates can be used only with relatively clear water, as suspended material will clog the plates. A number of proprietary underdrains are available.

Gravel Layer. To prevent resin from being washed out of the underdrain system, as well as to achieve good flow distribution during backwash, a layer of graded gravel is used. Porous plate underdrains do not require a gravel layer. A total gravel depth of 15 to 18 inches using three or more layers of gravel graded from ⅛ inch to 1 inch is typical.

Wash Troughs. Wash troughs are located at an elevation above the highest level reached by the resin during backwash. Thus, wash trough elevation must account for gravel depth and media depth during backwash.

Brine and Rinse Water Distribution. Assuming downflow regeneration, the brine distribution manifold is placed immediately above the softener bed. Prior to brine introduction, water in the contactor is drawn down to slightly above the top of the resin. This allows brine to be introduced with minimal disruption of the resin. Rinse water, assuming downflow rinse, is introduced either through the brine distribution system or by flooding the bed, if this can be done without disrupting the resin surface.

Brine and Salt Storage. Salt storage and brine production are usually accomplished in the same basin. Salt is added to the basin in excess of the quantity that can dissolve. Thus, as concentrated brine (approximately 26 percent at saturation) is withdrawn from the bottom of the basin, fresh water introduced at the top of the basin dissolves additional salt. Undissolved salt should always be present.

Tank volume is established by the quantity of salt that must be stored, which is a function of the proximity of the plant to a reliable source of salt and the method of salt delivery. Large plants can take advantage of the lower cost of bulk delivery by either truck or rail; smaller plants can use salt delivered in bags. Some large plants use unrefined rock salt, at a significant cost savings. However, unrefined rock salt contains sand and silt, which require removal. Sand precipitates during brine production, whereas silt removal usually is accomplished in a separate holding basin. Periodic basin cleaning is necessary to remove sand and silt.

Saturated brine is removed from the basin by either pumping or a hydraulic eductor. Maintaining the brine at a consistent concentration is essential to assure that the proper salt dose is used during each regeneration. A sampling tap or a meter should be provided.

Water addition to the brine tank should be done with an air gap to prevent any possible cross-connection. Also, an overflow from the brine tank to a sewer should be provided. The density of salt is approximately 70 lb/cu ft.

Product Water Blending Product water from the exchanger will have slight hardness (about 1 percent of influent hardness), due to bed ‘‘leakage.’’ Because a product water hardness concentration of about 100 mg/L as $CaCO_3$ is usually desirable, blending should be done to reduce operating costs. This may be accomplished by blending low-hardness product water with unsoftened water in the correct proportion to achieve the desired final water hardness, or by running an exchanger past the point where the normal background leakage concentration is exceeded. The latter technique requires sufficient storage capacity for softened water.

Nitrate Removal Applications

Nitrate ions can be removed from water supplies using strong-base ion-exchange resins operating in the chloride cycle. Resins commonly used are Duolite A-101D and A-104, Dowex SAR, Ionac A-550, and Amberlite IRA-900 and IRA-910. These resin types can be regenerated using sodium chloride brine.

These resins will remove a number of anions in addition to nitrate, including sulfate, nitrate, chloride, and bibcarbonate. The order of affinity for these anions is:

Increasing affinity ↑

- Sulfate
- Nitrate
- Nitrite
- Chloride
- Bicarbonate

Because sulfate ions are preferentially removed over nitrate ions, and nitrate ions are preferentially removed over nitrate, chloride, and bicarbonate ions, as the resin becomes exhausted, bicarbonate breaks through first, followed by chloride, then nitrate, then nitrite. The sulfate ion is the most strongly sorbed, and hence will displace previously removed nitrate ions at the end of the service cycle. This is an undesirable situation, and can result in a higher nitrate concentration in the exchanger product water than in the exchanger feedwater. The service cycle must be terminated before this point.

Design and Operating Parameters Equipment virtually identical to that described for the softening exchange can be used for anion exchange. Similarly also, sodium chloride is used for resin regeneration, and thus the salt storage and brine feed facilities are identical to cation-exchange applications.

Operation is usually downflow, followed by upflow backwash, and downflow regeneration and rinsing. Suggested design parameters for Duolite A-104 resin are presented in Table 18–6.

Pretreatment. Pretreatment requirements are basically the same as for cation-exchange resins, and include upper limits for iron, suspended solids, and organics.

Resin Capacity Determination. To size the exchangers adequately, the capacity of the resin must be determined for the particular water being treated. Resin capacity is determined primarily by the ratio of nitrate to total anions, and the ratio of sulfate to total anions.

The first step in the capacity determination is to conduct a complete water analysis. Using this analysis, all free anions should be converted to mg/L as calcium carbonate, using the multipliers in Table 18–7.

Next, the ratio of nitrate (as $CaCO_3$) to total anions (as $CaCO_3$) is calculated, and entered into Figure 18–3 to determine nitrate removal capacity in kilograins per cubic foot of resin. Then this resin capacity must be adjusted to account for the presence of sufate ions. This is accomplished by calculating the ratio of sulfate (as $CaCO_3$) to total anions (as $CaCO_3$), and entering the figure into Figure 18–4 to determine the percent resin efficiency. This resin efficiency must be multiplied by the nitrate removal capacity from Figure 18–3.

For illustrative purposes, assume a water supply with 118 mg/L of sulfate, 105 mg/L of nitrate, 55 mg/L of chloride, and 206 mg/L of bicarbonate. Proceed by calculating nitrate/total anion and sulfate/total anion ratios.

Ion	Concentration, mg/L	Concentration as mg/L $CaCO_3$
Sulfate	118	122.7
Nitrate	105	85.1
Chloride	55	77.6
Bicarbonate	206	168.9
		Total anions = 454.3

$$\frac{\text{Nitrate}}{\text{Total anions}} = \frac{85.1}{454.3} = 0.181$$

$$\frac{\text{Sulfate}}{\text{Total anions}} = \frac{122.7}{454.3} = 0.27$$

Using Figure 18–3, the uncorrected exchange capacity is 7.75 kilograins/cu ft. The correction factor for the sulfate ion concentration is 71 percent, as shown in Figure 18–4. Thus, the resin capacity corrected for sulfate concentration is:

TABLE 18–6. Suggested Design Parameters for Duolite A-104 Resin

Parameter	Design/Operating Value
Minimum bed depth	30 inches
Service flow rate	Up to 5 gpm/cu ft of resin
Backwash flow rate	2 to 3 gpm/sq ft
Regenerant concentration (NaCl)	10–12% by weight
Regenerant dosage	10–18 lb NaCl/cu ft of resin
Regenerant flow rate	0.5 gpm/cu ft of resin
Regenerant contact time	50–80 minutes
Rinse flow rate	0.5 gpm/cu ft per bed volume plus 2 gpm/cu ft of resin
Rinse volume	50–70 gal/cu ft of resin

Source: See reference 6.

$$7.75\ (0.71) = 5.50 \text{ kilograins/cu ft } (194.4 \text{ kilograins/m}^3)$$

Required Resin Volume. The volume of resin required is a function of three factors:

- Resin capacity
- Required run time between regenerations
- Resin bed depth and flow per cubic foot as recommended by the resin manufacturer

Using the previously calculated resin capacity, a flow rate of 0.25 mgd (0.95 ML/d), an operating time of 12 hours between regeneration, a bed depth of 3 feet (0.91 m), and a maximum service flow rate of 5 gpm/cu ft (0.67 $m^3/m/m^3$), the required resin volume is calculated as follows:

$$\begin{aligned} &\text{Nitrate feed per 12 hours} = \\ &= (0.25 \text{ mgd})\ (8.34)\ (105 \text{ mg/L}) \left(\frac{12 \text{ hr}}{24 \text{ hr/day}}\right) (7 \text{ kilograins/lb}) \\ &= 766 \text{ kilograins} \end{aligned}$$

$$\text{Required resin volume} = \frac{766 \text{ kilograins}}{5 \text{ kilograins/cu ft}} = 153 \text{ cu ft}$$

TABLE 18–7. Hardness Conversion Factors to mg/L as $CaCO_3$

Ion	Multiplier to Convert from mg/L as the Ion to mg/L as $CaCO_3$
SO_4^{-2}	1.04
NO_3^-	0.81
Cl^-	1.41
HCO_3^-	0.82

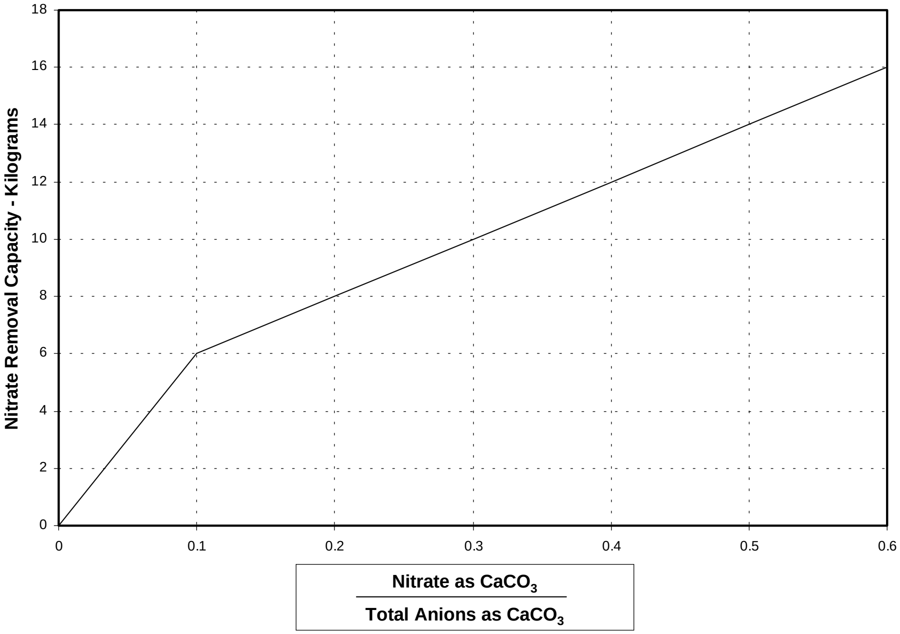


Fig. 18–3. Typical plot showing nitrate removal capacity as a function of the nitrate/total anion ratio (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

A check of the service flow rate per cubic foot of resin shows that it is 173.6 gpm/153 cu ft, or 1.13 gpm/cu ft. This is less than the maximum rate recommended by the manufacturer and is therefore acceptable. At a 3-foot bed depth, the required contactor area is:

$$\frac{153 \text{ cu ft}}{3 \text{ ft bed depth}} = 51 \text{ sq ft}$$

This is equivalent to a loading rate of 3.4 gpm/sq ft. Consideration should be given to a deeper resin depth, which would give a higher loading. Loading rates up to 20 gpm/sq ft are reported to provide satisfactory nitrate removal.

Bed Expansion During Backwash. Contactor design must include allowance for bed expansion during backwash. As with cation-exchange resins, the percentage bed expansion is a function of water temperature and backwash flow rate. Figure 18–5 presents backwash expansion curves for Amberlite IRA-900 operating in the chloride form.

Resin Regeneration. Resin regeneration can be either complete of partial. If resin is completely regenerated, the salt requirement is high, but there is little leakage of nitrate ions during operation. Conversely, if a low amount of salt is used, regeneration is incomplete, and nitrate leakage occurs during normal operation. Run time between

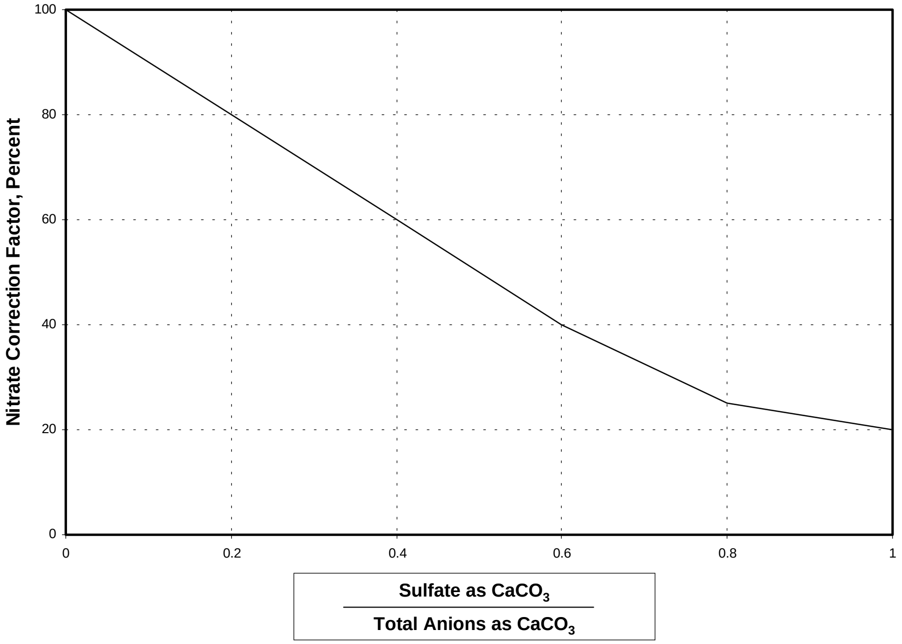


Fig. 18–4. Typical plot showing efficiency of nitrate removal as a function of the sulfate/total anion ratio (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

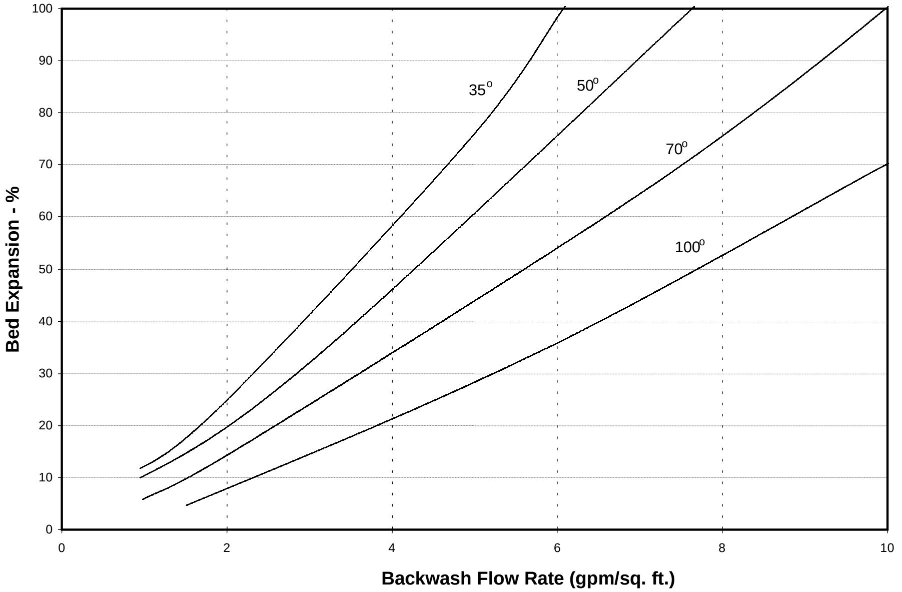


Fig. 18–5. Bed expansion versus backwash flow rate for Duolite A-104 resin

regenerations is lessened as the salt requirement is decreased. The overall advantage of incomplete regeneration is substantial decrease in salt requirements. The optimum regenerant dosage should be determined during pilot testing.

SORPTION ON ACTIVATED ALUMINA

Introduction

Historically, activated alumina has seen limited use in water treatment, primarily for defluoridation of drinking water supplies. Recent changes in the Safe Drinking Water Act place emphasis on arsenic removal, a task for which activated alumina is particularly well suited. Although at this writing there are no major plants using activated alumina for arsenic treatment, it is anticipated that activated alumina will be the technology of choice for many small utilities confronted with a new arsenic MCL in the 5–10 ppb range.

Activated alumina is a granular, highly porous material consisting of a mixture of amorphous and crystalline phases of aluminum oxide. Its chemical composition is approximated as aluminum trihydrate, Al_2O_3. It is frequently prepared as a by-product of aluminum production in which an aluminum hydroxide slurry is heated to remove waters of hydration, then activated with steam or acid to increase its surface area and adsorptive properties. It is widely used as a commercial desiccant and in many gas-drying processes. Typical properties of commercially available alumina are as shown in Table 18–8.

Because alumina has a higher pH zero-point-of-charge (pHzpc $\sim$ 8.2) than most oxide minerals, it has an adsorptive affinity for many negatively charged constituents. And because of its crystalline structure, it is highly selective for specific anions. The removal mechanism involves the exchange of hydroxyl ions (OH^-) for the dissolved species. It was originally thought that anion removal by activated alumina was purely an ion-exchange process. We now know that the process is more complicated, and for some anions is at least partially an adsorptive process that can be modeled on a Langmuir isotherm.[7] The selectivity of most activated aluminas is as follows:

TABLE 18–8. Properties of and Specifications for Activated Alumina, F-1 Type

Constituents and Properties	Content
Al_2O_3	92.00%
Na_2O	0.90%
Fe_2O_3	0.08%
SiO_2	0.09%
Loss on ignition (1,100°C)	6.50%
Form	Granular
Surface area (sq. m/g)	210
Size	¼″ 14 mesh
Bulk density, loose (g/cu cm)	0.83
Bulk density, packed (g/cu cm)	0.88
Specific gravity	3.3

Courtesy of Alcoa.

$$OH^- > H_2AsO_4^- > Si(OH)_3^- > F^- > SO_4^{2-} > HCO_3^- > Cl^-$$

Activated alumina can be regenerated with HCl, H_2SO_4, alum, or NaOH. The use of NaOH, followed by a neutralization step to remove residual NaOH from the bed, appears to be the most practical approach to regeneration.

Defluoridation of Water Supplies

The beneficial effect of fluoride is its ability to prevent dental cavities when an optimum amount is present in drinking water. However, on the negative side, long-term consumption of water containing excessive amounts of fluoride can lead to fluorosis of the teeth and bones. Fluoride commonly occurs in the earth's crust as fluorspace (CaF_2), cryolite (Na_3AlF_6), and fluorapatite ($(Ca_{10}F_2PO_4)_6$) and ranks 13th among the elements in order of abundance. Fluoride is present in seawater at a concentration of approximately 1.4 mg/L, and concentrations as high as 9–10 mg/L are not uncommon in some groundwaters.

According to the Safe Drinking Water Act (SDWA), all waters that contain more than the allowable maximum fluoride concentration must be defluorinated prior to use as public water supplies. At least 125 U.S. communities reportedly use groundwater supplies with 4.0 mg/L or more of fluorides.[8] Most of these communities are located in Arizona, Colorado, Illinois, Iowa, New Mexico, Ohio, Oklahoma, California, South Dakota, and Texas.

If the provisions of the SDWA are strictly enforced, many municipalities must either install defluoridation facilities or switch to low-fluoride sources of supply. Since so few treatment facilities have been built, no single technique has emerged as the best approach for defluoridation. The most promising approach to defluoridation of municipal water supplies appears to be removal with packed beds of granular activated alumina. Defluoridation with an activated system involves four separate modes: treatment, back-wash, regeneration, and neutralization. To describe system operation, it is convenient to start by assuming that the bed is in an exhausted state. Sodium hygroxide has been widely used in practice, and this appears to be the best choice of regenerant. The following sequence of operations describes the procedure to regenerate an exhausted bed of activated alumina.[9]

1. An upflow backwash at 8–9 gpm/ft^2 with raw water. This backwash expands the bed and removes any suspended solids that might have been trapped in the bed. Normal backwash time is approximately 10 min.
2. An upflow regeneration step employing a 1 percent (by weight) NaOH at 2.5 gpm/ft^2 for approximately 35 min.
3. An upflow rinse at 5.0 gpm/ft^2 for approximately 30 min.
4. A final regeneration step in the downflow direction, using 1 percent NaOH at 2.5 gpm/ft^2 for approximately 35 min.

This series of operations strips fluoride from the bed and restores the removal capacity of the activated alumina. However, following these operations the entire bed is in the pH range of 12.5 to 13 as a result of the caustic solution used for regeneration. Fluoride removal by activated alumina is strongly pH dependent with optimum removal at pH

5.0 (see Fig. 18–5). Consequently, the regenerated bed will not be effective in removing fluoride unless its pH is adjusted.

Bed neutralization is accomplished by adjusting the raw-water pH with sulfuric acid. The procedure used at the Gila Bend, Arizona, defluoridation plant has been described as follows:[9] After regeneration, the raw-water pH is initially adjusted to 2.5, and the water is fed to the bed at the normal treatment rate of 5–6 gpm/ft^2. Neutralization of the bed is evidenced by a drop in the pH of effluent water, as shown in Figure 18–6. Fluoride will not be removed from the raw water during the early stages of neutralization because of the caustic condition of the bed. The first water through the regeneration bed must be discharged to waste.

When the pH of the effluent water drops to 9.0–9.5, the bed will begin to remove some fluoride. At this point, the raw-water pH is adjusted to 4.0 and the neutralization process continues. When the effluent stream shows a pH of 8.5, the raw water is adjusted to pH 5.5, and it is maintained at that pH throughout the remainder of the run.

Fluoride removal will be nearly 100 percent during the early part of a run, but will decrease toward the end of the run as the bed becomes exhausted. Low-fluoride water (0.1 mg/L) produced during the early part of the run can be blended with high-fluoride water (3.0 mg/L) produced during the latter stages of the run to yield a water with an acceptable level of fluoride. If a suitable reservoir is not available for blending, staggered regeneration of the treatment beds may be employed to maximize system performance. During staggered regeneration, low-fluoride water from the fresh bed is blended with water containing a higher concentration of fluoride from the nearly ex-

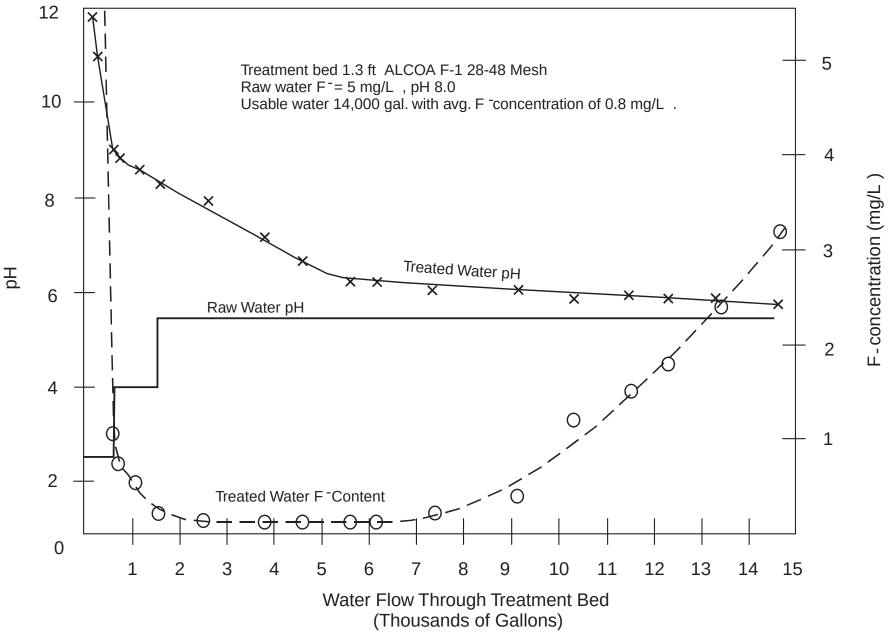


Fig. 18–6. Typical fluoride removal performance of activated alumina pilot unit at Gila Bend, Arizona

hausted second bed, to produce a finished water with an acceptable fluoride concentration. The maximum fluoride concentration at which a run must be terminated will depend on the fluoride concentration achievable by blending. Blending also serves to smooth out pH variations; however, supplemental pH control is usually required to produce stabilized water.

Waste, amounting to about 4 percent of the total plant throughout, is produced during backwashing, regeneration, and the early part of neutralization. The backwash water is composed of raw water only and may be discharged to surface waters or storm sewers. The neutralization waste has a high pH, and the regeneration waste has a high pH and a high concentration of fluoride ions. Disposal techniques must be chosen to conform to local water pollution control guidelines.

The ability of activated alumina to remove fluoride depends on the chemistry of the water being treated. Factors such as hardness, silica, and boron, if present in the water, will interface with fluoride removal and reduce the efficiency of the system. Information necessary for the design of activated alumina systems must be collected through laboratory and pilot plant tests of the water to be treated.

Most activated alumina defluoridation systems employ beds that contain a 5-ft depth of media, with at least two beds per installation. Design flows are in the range 5.0–7.5 gpm/ft^2 (1.0–1.5 gpm/ft^3 of media). A 5-minute empty bed contact time (EBCT) appears to be the minimum time needed to achieve maximum removal efficiency. Rubel and Woosley (1979) indicated removal capacities in the range of 2,000–4,000 grains/ft^3 when the raw-water pH is approximately 5.5. Capacities are reported to drop to 500 grains/ft^3 at a raw-water pH of 7.0.

Arsenic Removal

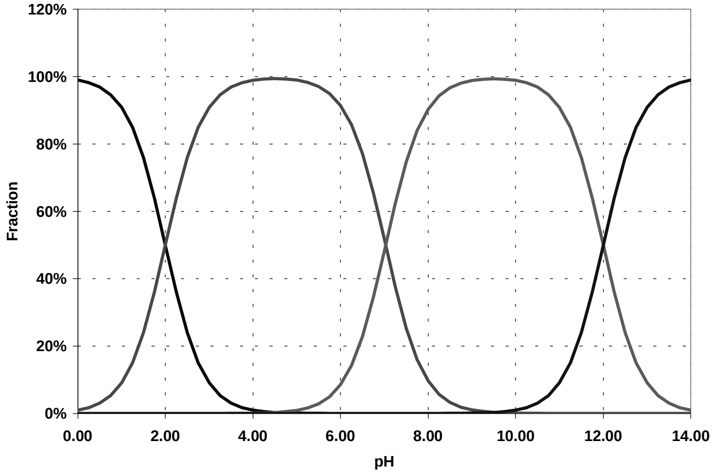
Soluble arsenic is commonly present in two oxidation states: As(V) and As(III). Hydrated As(V) species are negatively charged above a pH of 2.2, whereas negatively charged As(III) species do not predominate until a pH of 9.2 (see Fig. 18–7). Since the pHzpc of activated alumina is approximately 8.2, As(V) is readily sorbed at near-neutral pH, whereas As(III) species are not. However, As(III) is readily oxidized by common oxidizing agents, including chlorine, to the As(V) state.

Design Criteria Based on investigations for the City of Albuquerque, several variables have been identified that must be considered in designing an activated alumina As treatment facility, including:

- pH
- Hydraulic contact time (3- to 15-min EBCT)
- Effect of sulfate concentration on As removal
- Effect of silica concentration on As removal
- Process run time
- Regeneration methods
- Activated alumina consumption as a result of regeneration

The effect of each of these variables on the treatment process is beyond the scope of this text. However, the most important of these, which include pH, empty bed contact time, and process run time are presented in the following sections.

As (V) Species



As (III) Species

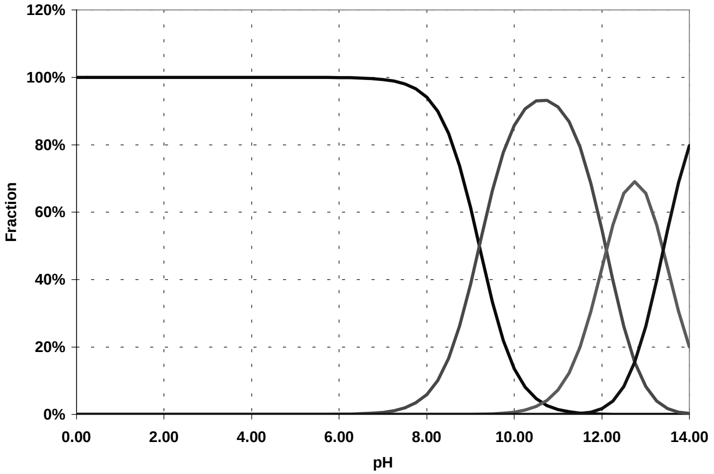


Fig. 18–7. Distribution of AS(III) and As(V) species as a function of pH

pH. The optimum pH for As removal by activated alumina was reported by Clifford and coworkers[10] to be in the range of 5.5 to 6.0. However, that study also noted that reducing the feedwater pH to this value consumes nearly all of the alkalinity, increases the TDS through addition of acid, and requires subsequent neutralization of the treated water. The benefit of lowering the pH is that the column runs are 5 to 20 times longer than those at natural pH. The amount of acid required to lower the pH of natural water depends on the initial pH and alkalinity of the water.

Because the feedwater must have reduced pH for the activated alumina system to operate, it will be necessary to adjust the product water pH prior to placing in the distribution system. This would likely be accomplished with caustic soda.

Empty Bed Contact Time. Clifford et al.[11] investigated EBCT values ranging from 1.5 to 10 minutes at pH 6; Amy et al.[12] (1998) investigated EBCT values ranging from 2.5 to 15 minutes at pH 7. Both investigations found that EBCT had a significant effect on run length.

Selection of the operating EBCT represents a compromise between the improved As removal versus the added cost of extra alumina and a bigger reactor vessel for a long EBCT. While neither investigator makes an explicit recommendation regarding the EBCT, the Clifford et al. report[11] implies that EBCT of 5.0 minutes is appropriate for this system. Alcan, a manufacturer of activated alumina, has also confirmed this recommendation of EBCT.

A second factor to consider in the hydraulic design of the sorption column is its aspect ratio (ratio of length to diameter). The Clifford et al. study recommends bed depth of at least 2.5 feet. For an EBCT of 5.0 minutes, this correlates to a surface loading of 3.7 gal/min-ft^2, which is comparable to ion-exchange loading rates.

Process Run Time. A common method of tracking the performance of any sorption process is to report the number of bed volumes (BVs) of water that have been treated to reach either breakthrough or exhaustion. Breakthrough is typically identified as when the effluent concentration increases to within 10 percent of the influent concentration or when it reaches a target concentration. Exhaustion is achieved when the total sorption capacity of the media has been consumed. Since the effluent concentration

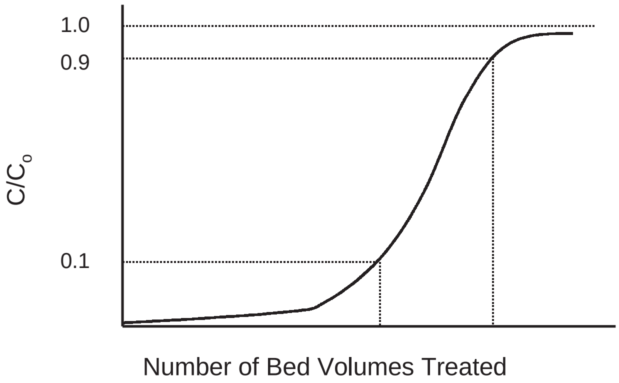


Fig. 18–8. Example of typical breakthrough curve for activated alumina

TABLE 18–9. Summary of Activated Alumina Design Criteria

Parameter	Design Criteria
Empty bed contact time, minutes	5
Activated alumina bed depth, feet	5
Activated alumina mesh size	28 × 40
NaOH regeneration strength, normal	1.0
Volume of NaOH for regeneration, BV	4.0
Sulfuric acid rinse strength, normal	0.4
Sulfuric acid rinse volume, BV	4.0
Activated alumina loss per regeneration, %	2

Source: Reference 13.

(C) approached the initial concentration (C_0) asymptotically, it is common to assume that exhaustion has been reached when $C/C_0 = 0.9$. These points are illustrated in Figure 18–8.

Clifford et al.[11] report that the number of BVs to reach breakthrough for a 5-minute EBCT ranges from 11,000 to 15,600 at pH 6, corresponding to run lengths ranging from 38 to 54 days. Amy et al.[12] report that treating approximately 1,200 to 3,000 BVs to breakthrough at pH 7.0. Most remarkably, Simms and Azizian give results of a treatability study in Severn Trent, United Kingdom, that treated 110,500 BVs to breakthrough at pH 6, which corresponded to 230 days of operation.[13] It is not clear why the three studies have such dramatically different run lengths. The columns run by Amy et al. used a different activated alumina and were operated at a higher pH than those in the study by Clifford et al.; thus it is possible that the capacity of the alumina was less and there was competition from other ions.

The use of activated alumina columns can be implemented as either series or parallel systems. In the parallel method, each vessel would be operated independently and the total product water flow would be combined. In the series method of operation, two vessels would be operated together, with one vessel being the lead and the second vessel acting as a polishing step. The advantage of this approach is that the first column can be run to exhaustion to take advantage of its full sorption capacity. The polishing column is then moved into the lead position, while the former lead is regenerated and placed in the polishing column position.

It has been noted that if very long runs (i.e., >20,000 BVs) are possible, it may be economically feasible to utilize direct disposal of the loaded activated alumina media rather than regenerate it. In addition to the potential cost savings, eliminating regeneration from the treatment system would limit handling of hazardous chemicals (caustic using in regeneration) and would simplify disposal of treatment residuals.

Summary of Activated Alumina Design Criteria Based on the above discussion, reasonable design criteria for activated alumina as applied to arsenic removal are summarized in Table 18–9.

REFERENCES

1. Abrams, I. M., and Benezra, L., "Ion Exchange Polymers," in *Encyclopedia of Polymer Science,* John Wiley & Sons, New York, 1967.

2. Panswad, T., ‘‘Ion Exchange Removal of Inorganic and Organic Wastewater Contaminants,’’ Ph.D. dissertation, University of Colorado, 1975
3. Salmon, J. E., and Hale, D. K., *Ion Exchange*, Academic Press, New York, 1959.
4. Amberlite Fact Sheet, Technical Bulletin, IE-169-70/76/78, Rohm and Haas Company, Philadelphia, 1980
5. Amberlite IR-120 Plus Technical Bulletin No. IE-123-67/78, Rohm and Haas Company, Philadelphia, Pa., May 1980.
6. Duolite A-104 Data Sheet, Rohm and Haas Company, Philadelphia, Pa., 1980.
7. Wu, Y. C., and Nitya, A., ‘‘Water Defluoridation with Activated Alumina,’’ Journal of Environmental Engineering, Div. ASCE, 105:4, 1979.
8. Water Industry Technical Action Fund (WITAF) Database, American Water Works Association, Denver, CO, 1995.
9. Rubal, F., and Woosley, R. D., ‘‘Removal of Excess Fluoride from Drinking Water,’’ EPA Technical Report, EPA 570/9-78-001, 1978.
10. Clifford, D., Linn, C. C., ‘‘Ion Exchange, Activated Alumina and Membrane Processes for Arsenic Removal,’’ *Proceedings of Annual Environmental Engineering Conference*, University of Kansas, 1995
11. Clifford, D., et. al., ‘‘Field Study of Arsenic Removal in Albuquerque, New Mexico,’’ City of Albuquerque N.M., 1998
12. Amy, G., et. al., ‘‘Arsenic Treatability Issues and Evaluation of Residuals,’’ AWWA Research Foundation, 1998
13. Simms, J., and Azizian, F., ‘‘Pilot Plant Trials on Removal of Arsenic from Potable Water Using Activated Alumina,’’ report by Alcan Chemicals, Buckinghamshire, England, 1998.

CHAPTER 19

Disinfection

The term ‘‘disinfection’’ refers to the inactivation of pathogenic organisms, making them incapable of transmitting disease. Inactivation does not mean killing. It may only be necessary to prevent an organism from reproducing to prevent the spread of disease. Chemical disinfectants inactivate pathogens through one of three main mechanisms:

- Destroy or impair cellular structure by attacking major cell constituents. This involves destroying the cell wall or impairing the functions of semipermeable membranes.
- Interfere with energy-yielding metabolism by rendering the enzymes nonfunctional
- Interfere with biosynthesis and growth by preventing synthesis of normal proteins, nucleic acids, coenzymes, or the cell wall

Depending on the disinfectant and microorganism type, combinations of these mechanisms can also be responsible for pathogen inactivation. In water treatment, it is believed that the primary factors controlling disinfection efficiency are the ability of the disinfectant to oxidize or rupture the cell wall,and the ability of the disinfectant to diffuse into the cell and interfere with cellular activity.[1]

In water treatment, disinfection effectiveness depends on several factors:

- *Disinfectant type.* Each type of disinfectant inactivates an organism at a different rate.
- *Disinfectant dose.* Higher dose increases the inactivation rate.
- *Type of organism and its physiological condition.* In general, protozoa are more resistant to disinfectants than bacteria or viruses. Some organisms form resistant forms (for example, cysts).
- *Contact time.* Increasing the contact time will increase pathogen inactivation.
- *pH.* The effectiveness of certain disinfectants (such as chlorine) is highly pH dependent. Others (such as monochloramine or ozone) are not pH dependent.
- *Temperature.* Typically, increasing the temperature will increase the rate of disinfection.
- *Turbidity.* Particles causing turbidity can shield pathogenic microorganisms from disinfectants. Particles also increase the disinfectant demand and reduce the concentration.

- *Dissolved organics.* Dissolved organics can interfere with disinfection by consuming the disinfectants to produce compounds with little or no microbiocidal activity. These reactions also produce undesirable disinfection by-products (DBPs).

Because of the difficulties inherent in timely measurements of pathogens, regulatory compliance for disinfection requirements is specified as a *CT* value. *CT* is defined as the product of the residual disinfectant, *C*, in mg/L, and the contact time, *T*, in minutes. Because disinfectant concentrations decrease as the liquid passes through the contact basin, the disinfection residual is typically the average or effluent disinfectant residual. In order to address nonperfect plug flow conditions in full-scale contact basins, the contact time used for this calculation is specified as the T_{10} value that represents the time after which 10 percent of the water has passed through the contact basin—in other words, the time that 90 percent of the liquid is retained in the contact basin.

The EPA developed *CT* values for the inactivation of *Giardia* and viruses for various disinfectants. Table 19–1 presents illustrative *CT* values for virus inactivation using chlorine, chlorine dioxide, ozone, chloramine, and ultraviolet light disinfection. Table 19–2 shows illustrative *CT* values for inactivation of *Giardia* cyst using chlorine, chloramine, chlorine dioxide, and ozone.

REGULATORY REQUIREMENTS

Disinfection practice must balance the need to inactivate pathogens and eliminate the formation of undesirable disinfection by-products (DBPs). Disinfection must achieve the following:

- Inactivate the target organisms
- Meet DBP requirements
- Meet disinfectant residual requirements
- Maintain the required disinfection residual in the distribution system

TABLE 19–1. *CT* Values for Inactivation of Viruses

		Inactivation		
Disinfectant	Units	2-log	3-log	4-log
Chlorine*	mg · min/L	3	4	6
Chloramine†	mg · min/L	643	1,067	1,491
Chlorine dioxide‡	mg · min/L	4.2	12.8	25.1
Ozone	mg · min/L	0.5	0.8	1.0
UV	mW · s/cm^2	21	36	Not available

Source: See Reference 2. Adapted from *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Works Systems Using Surface Water Systems,* by permission. Copyright © 1991, American Water Works Assocation.

*Values are based on a temperature of 10°C, pH range of 6 to 9, and a free chlorine residual of 0.2 to 0.5 mg/L.

†Values are based on a temperature of 10°C and a pH of 8.

‡Values are based on a temperature of 10°C and a pH range of 6 to 9.

TABLE 19–2. *CT* Values (mg · min/L) for Inactivation of *Giardia* Cysts

Disinfectant	Inactivation 0.5-log	1-log	1.5-log	2-log	2.5-log	3-log
Chlorine*	17	35	52	69	87	104
Chloramine†	310	615	930	1,230	1,540	1,850
Chlorine dioxide‡	4	7.7	12	15	19	23
Ozone‡	0.23	0.48	0.72	0.95	1.2	1.43

Source: See Reference 2.

* Values are based on a free chlorine residual less than or equal to 0.4 mg/L, temperature of 10°C, and a pH of 7.

† Values are based on a temperature of 10°C and a pH in the range of 6 to 9.

‡ Values are based on a temperature of 10°C and a pH of 7.

There are two disinfection objectives: primary and secondary disinfection. The goal in primary disinfection is to meet pathogen inactivation requirements, while secondary disinfection is used to maintain the residual in the distribution system.

Primary Disinfection

The Surface Water Treatment Rule (SWTR) requirement for *Giardia* cysts and viruses removal can be achieved through a combination of filtration and disinfection. Depending on the filtration process implemented, the disinfection inactivation requirements can range from 0.5 to 3 log and 2 to 4 log for *Giardia* cysts and viruses, respectively. Table 19–3 shows the disinfection requirements for various filtration options.

Secondary Disinfection

A disinfectant residual is required in the distribution system to protect the treated water from becoming contaminated. A utility can demonstrate the biological stability of the

TABLE 19–3. Log Removal/Inactivation Through Filtration and Disinfection Required under SWTR

Process	*Giardia* cysts	Virus
Total log removal/inactivation required	*3.0*	*4.0*
Conventional sedimentation/filtration credit	2.5	2.0
Disinfection inactivation required	0.5	2.0
Direct filtration credit	2.0	1.0
Disinfection inactivation required	1.0	3.0
Slow sand filtration credit	2.0	2.0
Disinfection inactivation required	1.0	2.0
Diatomaceous earth credit	2.0	1.0
Disinfection inactivation required	1.0	3.0
No filtration	0.0	0.0
Disinfection inactivation required	3.0	4.0

Source: See Reference 2.

water by carrying a residual disinfectant in the distribution system, or by meeting regulatory limits for heterotrophic plate counts (currently 500 organisms/mL).

Disinfectant and Disinfection By-Products

The regulations specify limits on the DBPs as well as residual disinfectants in finished water. It is important to note the that disinfectants themselves are also controlled compounds. This is important because it limits the maximum dose that can be applied to water to achieve the pathogen inactivation requirements. The relevant limits are presented in Chapter 1, Criteria and Standards for Potable Water Quality.

GENERAL DESIGN ISSUES

The two main components of a disinfection system are the chemical dose system and the contact basin.

The disinfectant dose is very site specific and often determined by field studies. The dose can be calculated by:

$$\text{Disinfectant dose} = \text{Residual required} + \text{Immediate demand} + \text{Disinfectant decay}$$

The required residual is that needed to meet the *CT* requirement. Many chemical reactions in the water will consume disinfectant. It is convenient to distinguish between immediate reactions that proceed very rapidly and then stop, and slow-decay reactions that continue for a long time. The slow-decay reactions are due to reactions with other pollutants in the water as well as spontaneous decay of the disinfectant itself.

The chemical feed system designed must provide the following:

- *Correctly sized units to meet the anticipated dose requirements.* The dose is determined from field tests and prior experience and selected to meet the *CT* requirements for the specific disinfectant. Because disinfection is a critical component of drinking water supply, redundant disinfectant feed equipment is typically provided.
- *Dose monitoring equipment that will control the disinfectant dose.* Online disinfectant residual analyzers can be used.
- *Chemical storage requirements to meet delivery schedules and handling objectives.* These requirements are site specific and should address typical chemical feed system design considerations (see Chapter 24, ‘‘Chemical Storage and Feeding Systems’’).
- *Disinfectant piping systems.* Avoid long diluted solution pipelines that give slow response times in the dose control system.
- *Worker safety requirements.* Many disinfectants are dangerous and require special handling features for containment, control, treatment, and monitoring.

A plug flow contact basin is preferred for disinfection systems. Plug flow eliminates short circuiting and the potential for pathogens escaping the contact basin with minimal contact time. Plug flow conditions are typically achieved by following the following guidelines:

- Design flow through contact basins with high length to width ratios—typically 40:1 length:width.
- Provide multiple reactors in series.
- Baffle basins to achieve a longer length:width ratio.
- Use pipe reactors.

Special attention should be give to the chemical dose/injection point. Good mixing must be provided to assure dispersion of the disinfectant through the entire liquid stream. The high reactivity of disinfectants also causes undesirable reactions in poorly mixed systems.

DISINFECTION ALTERNATIVES

Chlorine

Chlorine is widely used in the drinking water industry as both a disinfectant and an oxidant. Features that contribute to chlorine's wide use include:

- Effectively inactivates a wide range of waterborne pathogens
- Maintains a residual level in the water that can be monitored and controlled
- Lowest relative cost
- Successful long-term use in improving water treatment operations

Two of the most prominent concerns regarding chlorine usage are:

- Formation of disinfection by-products by oxidizing naturally occurring organic and inorganic compounds in water
- Safety considerations and regulations associated with using chlorine gas

In addition to disinfection, chlorine is used as an oxidant for:

- Prevention of algal growths
- Maintenance of clean filter media
- Removal of iron and manganese
- Destruction of hydrogen sulfide
- Bleaching of certain organic colors
- Control of biological growth in the distribution system
- Restoration and preservation of pipeline capacity
- Restoration of well capacity
- Improved coagulation by activated silica

Several of the above applications are discussed in Chapter 16, ‘‘Oxidation.’’

Forms of Chlorine

Chlorine Gas. Upon addition to water, chlorine gas reacts rapidly with water to form hypochlorous acid (HOCl) per the following reaction:

$$Cl_{2(g)} + H_2O \Rightarrow HOCl + H^+ + Cl^- \qquad (19\text{–}1)$$

Chlorine gas addition to water produces a hydrogen ion that reduces the pH of the water.

Hypochlorous acid is a weak acid that dissociates slightly into hydrogen and hypochlorite (OCl^-) ions:

$$HOCl \Leftrightarrow H^+ + OCl^- \qquad (19\text{–}2)$$

This dissociation is incomplete between a pH of 6.5 and 8.5, resulting in the presence of both HOCl and OCl^- species. Very little dissociation of HOCl occurs below a pH of 6.5, while complete dissociation to OCl^- occurs above a pH of 8.5. Because the disinfection efficacy of HOCl is significantly higher than that of OCl^-, chlorination at lower pH is preferred for disinfection.

Sodium Hypochlorite. Bulk commercial sodium hypochlorite solution typically contains 12.5 percent available chlorine. One gallon of 12.5 percent sodium hypochlorite solution typically contains the equivalent of one pound of chlorine.

Sodium hypochlorite is formed by dissolving chlorine gas in a sodium hydroxide solution. The addition of sodium hypochlorite solution to water results in the formation of HOCl similar to chlorine. The reaction is shown in the following expression:

$$NaOCl + H_2O \Rightarrow HOCl + Na^+ + OH^- \qquad (19\text{–}3)$$

Note that the addition of sodium hypochlorite to water yields a hydroxyl ion (OH^-) that increases the pH of the water, as opposed to chlorine addition, which decreases pH. In addition, the pH of the water is further increased due to excess sodium hydroxide used to manufacture sodium hypochlorite.

Sodium hypochlorite degrades over time. The sodium hypochlorite solution stability depends on the hypochlorite concentration, the storage temperature, and the length of storage time before use. The degradation of sodium hypochlorite is catalyzed by the impurities of the solution and exposure to sunlight. Sodium hypochlorite decomposition affects the feed rate and dosage over time.

Dilute sodium hypochlorite solutions (0.8 percent) can be generated electrochemically on-site from salt brine solution (see Chapter 24, “Chemical Storage and Feeding Systems,” for details). At this dilute concentration, the sodium hypochlorite is more stable than the bulk 12.5 percent solution.

Calcium Hypochlorite. Calcium hypochlorite is supplied in solid form and typically contains 65 to 70 percent available chlorine. The addition of calcium hypochlorite to water also results in the formation of HOCl. The reaction is shown in the following expression:

$$Ca(OCl)_2 + 2H_2O \Rightarrow 2HOCl + Ca^{++} + 2OH^- \qquad (19\text{–}4)$$

Note that the addition of calcium hypochlorite to water yields hydroxyl ions that increase the pH of the water, similar to sodium hypochlorite solution.

Application of Chlorine At a conventional surface water treatment plant, chlorine is typically added at the raw-water intake or flash mixer, ahead of the filters, at the filter clearwell, or in the distribution system. Table 19–4 summarizes the typical uses for each point of application.

Typical Chlorine Doses. Table 19–5 shows the typical range of dosages for the three forms of chlorine. The wide range of chlorine gas dosages most likely represents its use as both an oxidant and a disinfectant due to its lower cost than other forms of chlorine.

Disinfection CT Values. Chlorine is an effective disinfectant for bacteria, viruses, and *Giardia* cysts. Chlorine is not effective for inactivating *Cryptosporidium* oocysts at typical chlorine dosages. For example, Figure 19–1 shows the *CT* values required for virus and *Giardia* cyst inactivation at a pH of 7.0 and a temperature of 10°C. *CT* values for *Giardia* cyst inactivation vary with pH and temperature. The level of *CT* required increases as the pH increases and as the temperature decreases (see reference 2 for *CT* tables).

Chlorine Feed Facilities

Chlorine Gas. Gaseous chlorine is delivered in containers ranging in size from 150-lb cylinders to 90-ton rail cars. Chlorine gas is typically evaporated from liquid to gaseous chlorine prior to metering. A liquid chlorine evaporator or ambient heat provides the heat for evaporation. Once the compressed liquid chlorine is evaporated, chlorine gas is typically fed under vacuum until dissolved in water. Either an injector or a vacuum induction mixer can be used to create the required vacuum. The injector uses water to create a vacuum and drain chlorine into a solution containing about 3,500 mg/L of chlorine. The vacuum induction mixer uses the motive forces of the

TABLE 19–4. Typical Chlorine Points of Application and Uses

Point of Application	Typical Uses
Raw water intake	Zebra mussel and Asiatic clam control, control biological growth
Flash mixer (prior to sedimentation)	Disinfection, iron and manganese oxidation, taste and odor control, oxidation of hydrogen sulfide
Filter influent	Disinfection, control biological growth in filter, iron and manganese oxidation, taste and odor control, algae control, color removal
Filter clearwell	Disinfection
Distribution system	Maintain disinfectant residual throughout the system

Sources: See References 3–5.

TABLE 19–5. Typical Chlorine Dosages at Water Treatment Plants

Chlorine Compound	Range of Doses
Calcium hypochlorite	0.5–5 mg/L
Sodium hypochlorite	0.2–2 mg/L
Chlorine gas	1–16 mg/L

Source: See Reference 6.

mixer to create a vacuum and draw the chlorine gas directly into the process water at the mixer. A schematic of gaseous chlorine feed system is shown in Figure 19–2.

Sodium Hypochlorite. Sodium hypochlorite can be purchased in bulk in quantities ranging from 55-gal drums to 4,500-gal truckloads. Bulk loads can be stored in fiberglass-reinforced plastic or polyethylene tanks. Sodium hypochlorite solutions degrade over time and produce an off-gas that tends to accumulate in the piping and pumps. Design of the feed system should include methods for venting any high points in the delivery system.

Metering pumps are typically used to feed sodium hypochlorite solution directly into the process water. The sodium hypochlorite is mixed with the process water with either a mechanical mixer or induction mixer. Sodium hypochlorite solution is typically not diluted prior to mixing to reduce scaling problems. If dilution water is required for mixing, softened water can be used to minimize scaling. Distribution piping can be acid washed if needed to remove scaling due to sodium hypochlorite dilution. Figure 19–3 is a schematic of a typical hypochlorite feed system.

Calcium Hypochlorite. Under normal storage conditions, calcium hypochlorite loses 3 to 5 percent of its available chlorine in a year.[7] Calcium hypochlorite solution is

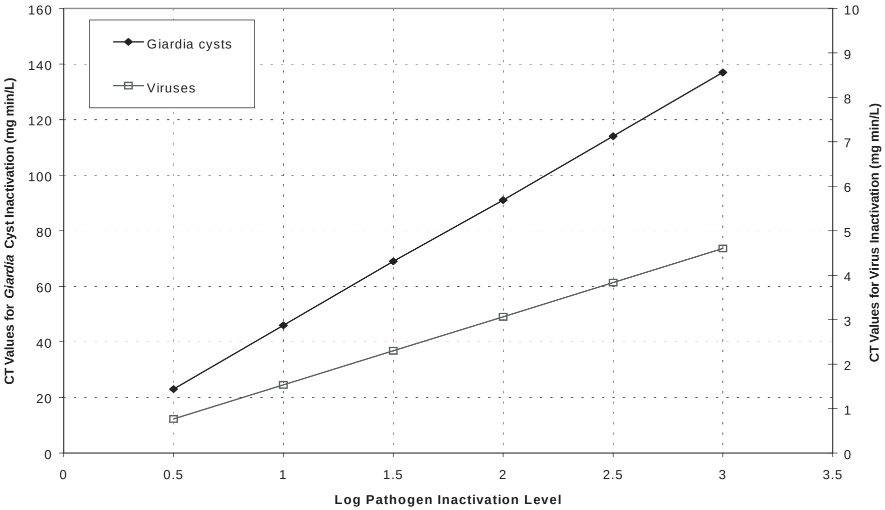


Fig. 19–1. Free chlorine *Giardia* and virus *CT* requirements (*Source:* Reference 2)

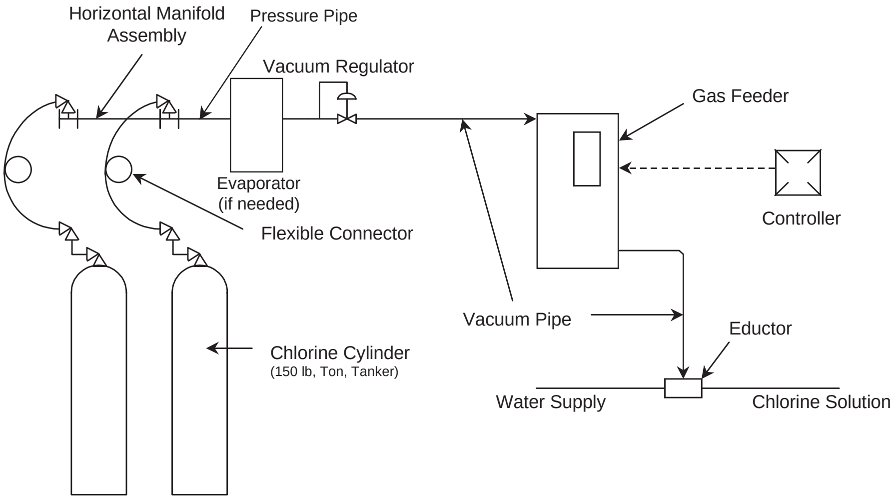


Fig. 19–2. Gaseous chlorine feed system

prepared by dissolving solid calcium hypochlorite in a sidestream of process water. The concentrated hypochlorite solution is flow paced into process water flow.

Considerations for Safety and Handling The U.S. Department of Transportation classifies chlorine as a poisonous gas. Fire codes typically regulate the storage and use of all forms of chlorine. Storage regulations typically require spill containment, separation from incompatible materials, and leak neutralization. In the case of chlorine gas, a scrubber may be required.

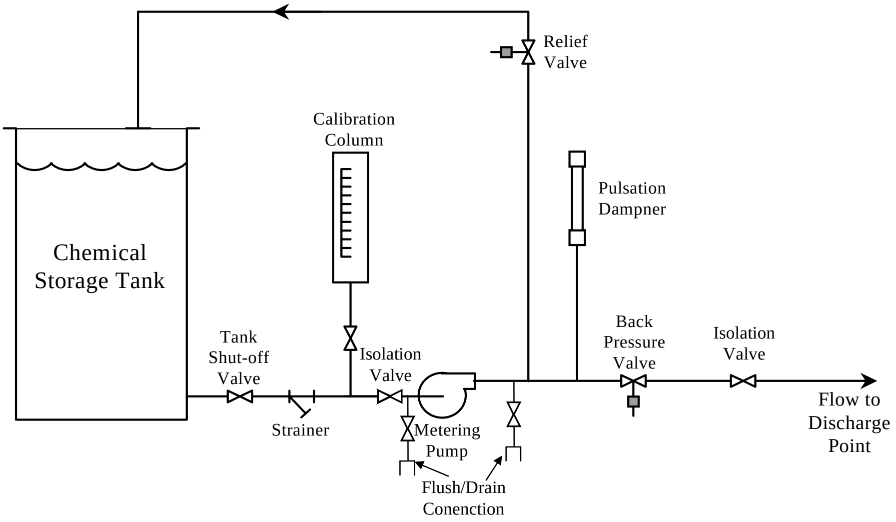


Fig. 19–3. Sodium hypochlorite feed system

In addition to these regulations, facilities storing more than 2,500 pounds of chlorine are subject to the following two safety programs:

- Process Safety Management standards regulated by the Occupational Safety and Health Administration under 29 CFR 1910.
- The Risk Management Program Rule administered by EPA under Section 112(r) of the Clean Air Act.

All of these regulations (as well as local and state codes and regulations) must be considered during the design and operation of chlorination facilities at a water treatment plant.

Monochloramine

Chloramines are formed by the reaction between ammonia and hypochlorous acid. Initially, chloramines were used in drinking water treatment for taste and odor control. However, upon use chloramines were found to be more stable than free chlorine in the distribution system. Hence, chloramines were found to be effective for controlling bacterial regrowth in the distribution system. Monochloramine has also gained increased interest because they form fewer DBPs than free chlorine

The mixture of chloramines that results from the reaction of chlorine and ammonia may contain monochloramine (NH_2Cl), dichloramine ($NHCl_2$), and/or nitrogen trichloride (NCl_3). Monochloramine is the preferred chloramine species for use in disinfecting drinking water because of taste and odor problems associated with dichloramine and nitrogen trichloride.

When chlorine is added to water, a rapid reaction occurs to form hypochlorous acid (see Equation 19–1). The hypochlorous acid (HOCl) formed is a weak acid that dissociates to hypochlorite (OCl^-) depending on the pH of the solution (Equation 19–2). HOCl reacts rapidly with ammonia in solution to form chloramines in a series of competing reactions.[4] The simplified chloramine reactions are as follows:

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \qquad \text{(monochloramine)}$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \qquad \text{(dichloramine)}$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \qquad \text{(nitrogen trichloride)}$$

These competing reactions are primarily dependent on pH, chlorine:ammonia nitrogen (Cl_2:N) ratio, temperature, and contact time. Figure 19–4 shows the typical relationships between the chloramine species at various Cl_2:N ratios for pHs ranging from 6.5 to 8.5. This figure shows that monochloramine is the predominant species formed when the Cl_2:N ratio is less than 5:1. The chlorine dose required to form different chlorine nitrogen reaction end products is shown in Table 19–6. To control chloramine reactions so that monochloramine is formed, the Cl_2:N ratio should be maintained between 3 and 5 by weight. Figure 19–5 shows the relationship between chloramine species as the pH changes.[8] Dichloramine becomes a dominant species at low pH.

The reaction rate to produce monochloramine is strongly pH-dependant (see Table 19–7). At pH levels encountered in normal water treatment plants, the reaction is practically instantaneous. The fast reaction requires that the dose point be very well mixed.

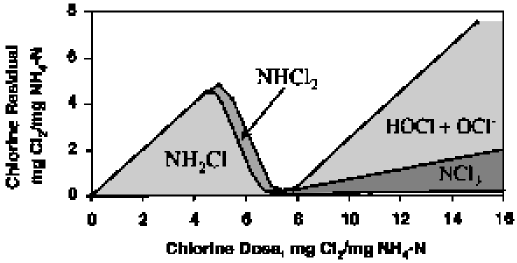


Fig. 19–4. Theoretical breakpoint curve (chlorine and ammonia)

Ammonia Feed Facilities The location of ammonia feed facilities can be on-site at the water treatment plant or in the distribution system. There are two forms of ammonia feed facilities: gaseous (anhydrous) ammonia or liquid (aqueous) ammonia.

Anhydrous Ammonia. Anhydrous ammonia is a gas at ambient temperature and pressure. Under pressure, anhydrous ammonia can be stored and transported in cylinders or tanks. Portable cylinders are similar to chlorine cylinders and are available in 100-, 150-, and 800-lb sizes. Stationary tanks are typically 1,000 gallon or greater and are refilled by tanker trailers. An outdoor tank requires protection from extreme temperatures (greater than 125°F and less than 28°F) to avoid over- or underpressurization. In colder climates, the tank should be insulated and heat traced to ensure the flow of ammonia gas from the tank.

Ammoniators are used to feed hydrous ammonia. An ammoniator is a modular unit that includes pressure-reducing valve, gas flow meter, and feed rate control valve to control the flow of ammonia. Automatically controlled ammoniators are available. An evaporator may be required when large quantities of ammonia gas are needed.

Anhydrous ammonia can be applied by either direct or solution feed. The typical piping materials for both direct and solution feed systems are stainless steel, PVC, and black iron. Stainless-steel or black iron pipe is used in the pressurized portions of the feed system. PVC pipe is used only in the vacuum portion of the feed system, after the ammonia vacuum regulators. The direct feed method is used when the application point has a low pressure and the maximum ammonia feed rate is less than 1,000 lb per day. Ammonia is regulated from the high-pressure storage tank and applied directly

TABLE 19–6. Chlorine Dose Required for NH_3/Cl_2 Reaction

Reaction	mg Cl_2/mg NH_3
Monochloramine (NH_2Cl)	4.2
Dichloramine ($NHCl_2$)	8.4
Nitrogen trichloride (NCl_3)	12.5
Nitrogen (N_2)	6.3
Nitrate (NO_3)	16.7
Free residual reaction	9

Source: Reference 7. (Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association.)

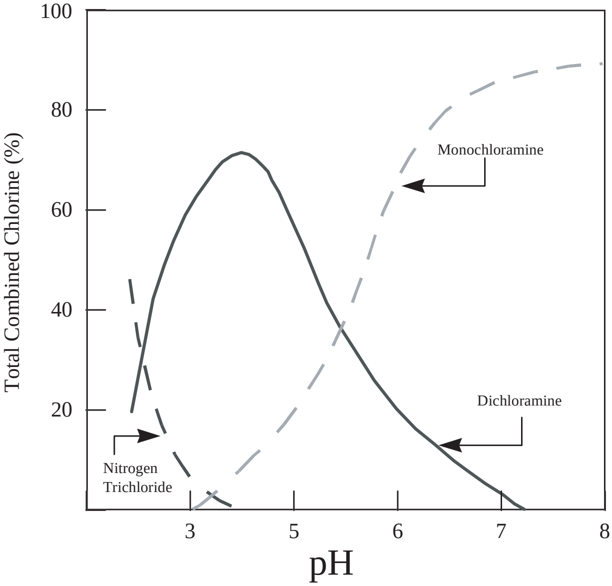


Fig. 19–5. Distribution diagram for chloramine species with pH (*Source:* Reference 8)

at a pressure of 15 psi. The storage tank pressure is first reduced to approximately 40 psi using a pressure reducing valve, and then to 15 psi by another pressure-reducing valve in the direct feed ammoniator. Typical low-pressure application points are open channels and clearwell facilities. Figure 19–6 is a schematic of a direct anhydrous ammonia feed system.

The solution feed method is used where the maximum ammonia feed rate is greater than 1,000 lb/day or where the application point pressure is higher than 10 psi. The solution feed method is similar to a chlorine vacuum feed system. A water supplied eductor is used to withdraw ammonia from the ammoniator. The ammonia is dissolved into the eductor water stream and applied to the process stream under pressure. Solution feed ammoniators are available up to 4,000 lb/day capacities and operate at application point pressures up to 100 psi. Softened water with a hardness less than 29 mg/L as $CaCO_3$ is required for the carrier stream to reduce scaling. One concern of

TABLE 19–7. Time to 99 Percent Conversion of Chlorine to Monochloramine

pH	Time (seconds)
2	421
4	147
7	0.2
8.3	0.069
12	33.2

Source: Reference 4.

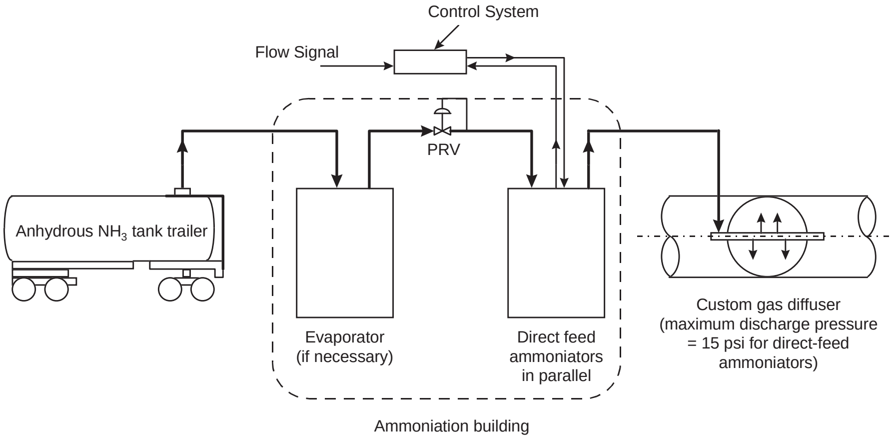


Fig. 19–6. Anhydrous ammonia direct feed system (*Source:* Reference 1)

solution feed systems is the formation of precipitate scale as the pH is elevated due to ammonia addition. The scale may plug the eductor and application point diffuser. Figure 19–7 shows a schematic of a solution feed system.

Aqueous Ammonia. Aqueous ammonia, otherwise known as ammonium hydroxide, is manufactured by dissolving anhydrous ammonia into softened water. Aqueous ammonia is shipped in bulk tanker trucks or 55-gallon drums. Aqueous ammonia is stored in low-pressure tanks, typically steel, polyethylene, or fiberglass-reinforced plastic. Piping materials are typically PVC. High temperatures will cause ammonia gas to

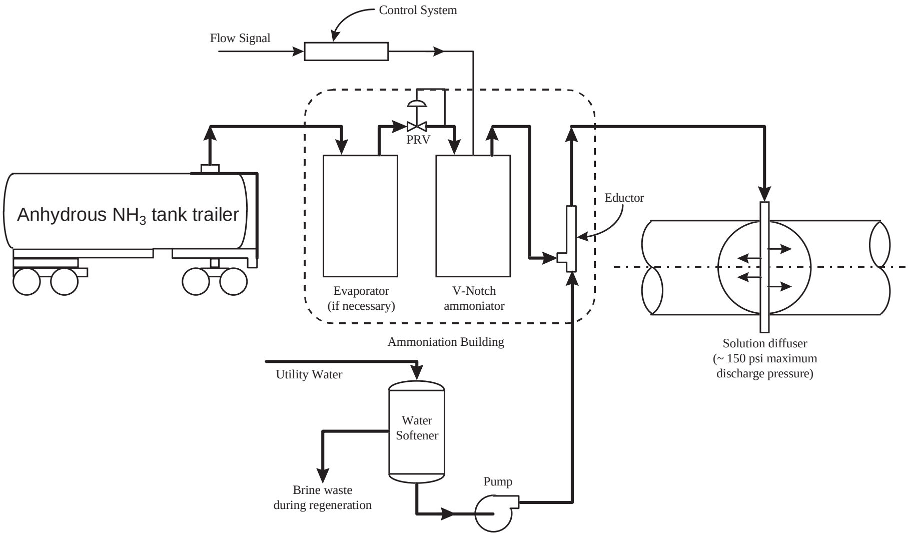


Fig. 19–7. Anhydrous ammonia solution feed system (*Source:* Reference 1)

vaporize. Storage tanks should be sealed with a water trap and an ammonia scrubber to keep vapors from escaping to the atmosphere.

Aqueous ammonia feed systems are similar to other liquid chemical feed systems. They require a storage tank, chemical metering pump (positive-displacement type), and relief valve. Other accessories may include a pulsation dampener, flow meter, calibration column, and backpressure valve. The chemical feed pumps should be placed near the aqueous ammonia storage tank to reduce ammonia vaporization in the piping. When aqueous ammonia solution is added to the process stream, complete mixing is required to ensure formation of monochloramine and to reduce the formation of dichloramine and nitrogen trichloride. Figure 19–8 shows a schematic of an aqueous ammonia feed system.

Safety Provisions for Chloramination Facilities A chloramination facility should include safety provisions to prevent the mixture of chlorine and ammonia gases. Chlorine gas and ammonia gas should never be stored in the same room. The reaction between the two gases forms nitrogen trichloride. Gaseous ammonia is lighter than air, so any leaking vapor will rise quickly. Indoor installation of storage tanks and/or chemical feed equipment require ventilation and leak detection devices to be located at high points in the room. Ventilation rates for the room will vary, depending on the appropriate regulatory agency's requirements.

Application of Monochloramine The primary use of monochloramine in water systems is as a secondary disinfectant in the distribution system because:

- Monochloramine has relatively weak disinfecting properties and is not practical for primary disinfection. The germicidal effectiveness of monochloramine is a factor of 200 less than for free chlorine, necessitating long contact times for monochloramine to meet EPA disinfection *CT* requirements.[2]
- Monochloramine is not as reactive with organic compounds as free chlorine and forms fewer DBPs than free chlorine.
- Monochloramine residual is the most stable and longest lasting of the disinfectant chemicals discussed, providing protection against bacterial regrowth in large distribution systems.

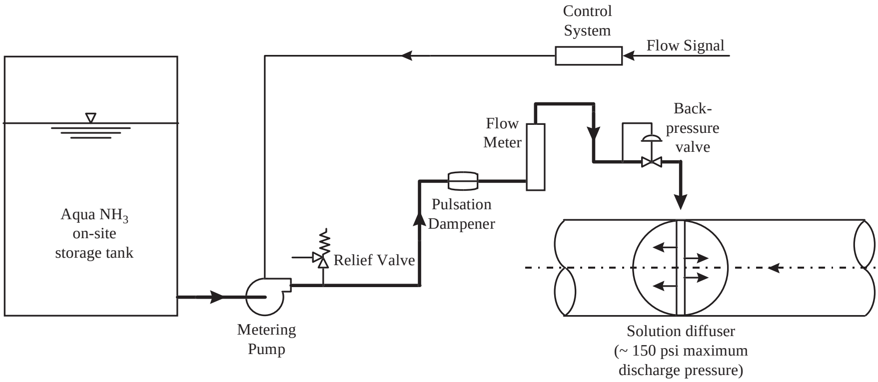


Fig. 19–8. Aqua ammonia feed system (From Montgomery, J. M., *Water Treatment Principles and Design.* Copyright © 1995 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

- Monochloramine residual has a superior ability to penetrate and control the biofilm in the distribution system.

The normal dosage range for monochloramine is in the range of 1.0 to 4.0 mg/L. The minimum residual of monochloramine in the distribution system is typically regulated at 0.5 mg/L.

Sequence of Chlorine and Ammonia Application. The formation of monochloramine in the treated water can be accomplished by first adding ammonia and then chlorine, or vice versa. Ammonia is added first where reactions between chlorine and organic compounds form DBPs and/or taste and odor compounds. Most drinking water systems add chlorine first in the treatment plant to meet EPA's SWTR disinfection requirements. Typically, the point of ammonia addition is selected to ''quench'' the free chlorine residual after disinfection requirements have been met to minimize DBP formation.

Operational Considerations for Monochloramine

Conversion to Chloramination from Chlorination. A bench-scale study should be performed to identify the water characteristics and to determine if chloramination is suitable. The amount of ammonia required for chloramine residual disinfection depends on several factors including:

- Organic nitrogen level in the water. If organic nitrogen is present in the untreated water, the amount of supplemental ammonia required should be carefully determined by subtracting the background ammonia present from the desired dose.
- Ammonia residual desired in the distribution system. For residual disinfection, approximately 1 to 2 mg/L of ammonia is required.
- Monochloramine residual concentration required in the distribution system. For each specific water, a breakpoint curve should be developed to determine the chlorine: ammonia ratio required.

Chloramination Operational Considerations

Nitrification. Ammonia in excess of the required chlorine:ammonia ratios can promote the growth of nitrifying bacteria in filter beds and in the distribution system.[4] Nitrification can have adverse effects on distribution system water quality, including a loss of total chlorine residual, an increase in heterotrophic plate count (HPC) bacteria levels, and the formation of nitrite during an intermediate step of nitrification. The occurrence and control of nitrification is discussed in detail in Chapter 22, ''Water Quality Control in Distribution Systems.''

Organic Nitrogen. Concentrations of organic nitrogen as low as 0.3 mg/L may interfere with the monochloramine residual levels in the distribution system. Monochloramine hydrolyzes the organic nitrogen present to form nongermicidal organochloramines. This reaction takes about 30 to 40 minutes. After the monochloramine is consumed by the organic nitrogen, free ammonia is released to instigate biological instability in the distribution system.

Mixing. Mixing at the point of application greatly affects the efficiency with which monochloramine is formed. The reaction time between ammonia and chlorine is nearly instantaneous at between 7 and 8.5. If chlorine is mixed slowly into ammoniated water, organic matter may react with the chlorine and consume chlorine prior to monochloramine formation. If ammonia is mixed slowly into chlorinated water, an excess of chlorine may be present in localized areas, causing breakpoint chlorination and/or the formation of dichloramine and nitrogen trichloride.

Blending Waters. When chlorinated water is blended with chloraminated water, the chloride:ammonia ratio will increase, causing breakpoint chlorination and formation of dichloramine and nitrogen trichloride. The entire monochloramine residual can be depleted. Therefore, it is important to determine how much chlorinated and chloraminated water can be blended without significantly affecting the monochloramine residual. Blended residual curves should be developed to determine the impact for each specific blend.

Human Health and the Environment. Kidney dialysis equipment users are the most impacted by monochloramine use. If chloramines are not removed from the dialysate water, chloramines can cause methemoglobinemia and adversely affect the health of kidney dialysis patients. Chloramines can also be deadly to fish in aquariums. Chloramine residuals should be removed from the water prior to addition to the aquarium.

Monochloramine DBP Formation. The effectiveness of monochloramine in controlling DBP production depends upon a variety of factors, including the point of addition of ammonia relative to that of chlorine, the level of mixing, and pH of the water.

Monochloramine (NH_2Cl) does not produce DBPs to any significant degree, although some dichloroacetic acid can be formed. Also, cyanogen chloride formation is greater with monochloramine than with free chlorine during secondary disinfection of the distribution system.

Chlorine Dioxide

The major uses of chlorine dioxide for potable water treatment are:

- Disinfection
- Taste and odor control
- Iron and manganese control

Chlorine dioxide (ClO_2) is a neutral compound with chlorine in the +IV oxidation state. It disinfects by oxidation of the pathogens without chlorinating. It is a volatile free radical. Even while in dilute aqueous solutions, chlorine dioxide forms a free radical, making a highly energetic molecule. Chlorine dioxide is reduced to chlorite (ClO_2^-) via a single electron transfer mechanism (Equation 19–5). The oxidation-reduction reaction for chlorine dioxide is:

$$ClO_{2(aq)} + e^- = ClO_2^- \qquad E° = 0.954V \qquad (19\text{–}5)$$

Other important half-reactions for the by-products of chlorine (ClO_2^-) and chlorate (ClO_3^-) are:

$$ClO_2^- + 2H_2O + 4e^- = Cl^- + 4OH^- \qquad E° = 0.76V \qquad (19\text{–}6)$$

$$ClO_3^- + H_2O + 2e^- = ClO_2^- + 2OH^- \qquad E° = 0.33V \qquad (19\text{–}7)$$

$$ClO_3^- + 2H^+ + e^- = ClO_2 + H_2O \qquad E° = 1.152V \qquad (19\text{–}8)$$

In drinking water (ClO_2^-) is the predominant reaction end product, with up to 70 percent of the chlorine dioxide converted to chlorite. The remaining end products are mainly ClO_3^- and chloride (Cl^-).[9]

Chlorine Dioxide Generation Chlorine dioxide gas cannot be compressed or stored commercially. It is explosive under pressure. Therefore, it is always generated at the point of use. Chlorine dioxide is also considered explosive at concentrations that exceed 10 percent by volume in air.

Most commercial generators use sodium chlorite ($NaClO_2$) as the base chemical from which chlorine dioxide is generated for drinking water application. Alternatively, chlorine dioxide can be produced from sodium chlorate ($NaClO_3$) with a mixture of concentrated hydrogen peroxide (H_2O_2) and concentrated sulfuric acid (H_2SO_4). Chlorate-based systems have traditionally been used in pulp and paper applications, but are considered an emerging technology in the drinking water field. Other emergent technologies include electrochemical systems and a solid chlorite inert matrix (flow-through gaseous chlorine).

Chlorine dioxide can be formed by reacting sodium chlorite with any of the following: chlorine ($Cl_{2(g)}$), hypochlorous acid (HOCl), or hydrochloric acid (HCl). Chlorine dioxide is formed from these combinations by one of the following equations:

$$2NaClO_2 + Cl_{2(g)} = 2ClO_{2(g)} + 2NaCl \qquad (19\text{–}9)$$

$$2NaClO_2 + HOCl = 2ClO_{2(g)} + NaCl + NaOH \qquad (19\text{–}10)$$

$$5NaClO_2 + 4HCl = 4ClO_{2(g)} + 5NaCl + 2H_2O \qquad (19\text{–}11)$$

Table 19–8 provides information on some types of available commercial generators.

Chlorate By-Product Formation The most undesirable by-product in chlorine dioxide generation is the chlorate ion (ClO_3^-). Chlorate production is representative of inefficient chlorine dioxide production. The overall reactions that describe chlorate ion formation during chlorine dioxide generation are:

TABLE 19–8. Commercial Chlorine Dioxide Generators

Generator Type	Main Reactions (Reactants, by-products, key reactions, and chemistry notes)	Special Attributes
ACID–CHLORITE: (Direct Acid System)	$4HCl + 5NaClO_2 \rightarrow 4ClO_{2(aq)} + ClO_3^-$ • Low pH • ClO_3^- possible • Slow reaction rates	Chemical feed pump interlocks required. Production limit ~25–30 lb/day. Maximum yield at ~80% efficiency.
AQUEOUS CHLORINE–CHLORITE (Cl_2 gas ejectors with chemical pumps for liquids or booster pump for ejector water)	$Cl_2 + H_2O \rightarrow [HOCl/HCl]$ $[HOCl/HCl] + NaClO_2 \rightarrow ClO_{2(g)} + H/OCl^- + NaOH + ClO_3^-$ • Low pH • ClO_3^- possible • Relatively slow reaction rates	Excess Cl_2 or acid to neutralize NaOH. Production rates limited to ~1000 lb/day. High conversion but yield only 80–92% More corrosive effluent due to low pH (~2.8–3.5). Three chemical systems pump HCl, hypochlorite, chlorite, and dilution water to reaction chamber.
RECYCLED AQUEOUS CHLORINE, OR "FRENCH LOOP"™ (Saturated Cl_2 solution via a recycling loop prior to mixing with chlorite solution)	$2HOCl + 2NaClO_2 \rightarrow 2ClO_2 + Cl_2 + 2NaOH$ • Excess Cl_2 or HCl needed due to NaOH formed.	Concentration of ~3 g/L required for maximum efficiency. Production rate limited to ~ 1000 lb/day. Yield of 92–98% with ~10% excess Cl_2 reported. Highly corrosive to pumps; draw-down calibration needed. Maturation tank required after mixing.

TABLE 19–8. (*Continued*)

GASEOUS CHLORINE–CHLORITE (Gaseous Cl_2 and 25% solution of sodium chlorite; pulled by ejector into the reaction column)	$Cl_{2(g)} + NaClO_{2(aq)} \rightarrow ClO_{2(aq)}$ • Neutral pH • Rapid reaction • Potential scaling in reactor under vacuum due to hardness of feedstock.	Production rates 5-120,000 lb/day. Ejector-based, with no pumps. Motive water is dilution water. Near neutral pH effluent. No excess Cl_2. Turndown rated at 5–10X with yield of 95–99%. Less than 2% excess Cl_2. Highly calibrated flow meters with min. line pressure ~40 psig needed.
GASEOUS CHLORINE–SOLIDS CHLORITE MATRIX (Humidified Cl_2 gas is pulled or pumped through a stable matrix containing solid sodium chlorite.)	$Cl_{2(g)} + NaClO_{2(s)} \rightarrow ClO_{2(g)} + NaCl$ • Rapid reaction rate • New technology	Cl_2 gas diluted with N_2 or filtered air to produce ~8% gaseous ClO_2 stream. Infinite turndown is possible with >99% yield. Maximum rate to ~1200 lb/day per column; ganged to >10,000 lb/day.
ELECTROCHEMICAL (Continuous generation of ClO_2 from 25% chlorite solution recycled through electrolyte cell)	$NaClO_{2(aq)} \rightarrow ClO_{2(aq)} + e^-$ • New technology	Countercurrent chilled water stream accepts gaseous ClO_2 from production cell after it diffuses across the gas-permeable membrane. Small one-pass system requires precise flow for power requirements (Coulombs law).
ACID/PEROXIDE/CHLORIDE	$2NaClO_3 + H_2O_2 + H_2SO_4 \rightarrow 2ClO_2 + O_2 + NaSO_4 + H_2O$	Uses concentrated H_2O_2 and H_2SO_4. Downscaled version; foam binding; low pH.

Source: Adapted from Reference 10.

$$ClO_2^- + HOCl = ClO_3^- + Cl^- + H^+ \tag{19–12}$$

and

$$ClO_2^- + Cl_2 + H_2O = ClO_3^- + 2Cl^- + H^+ \tag{19–13}$$

Conditions that contribute to the production of chlorate ion include:

- High ratios of Cl_2 gas to ClO_2^-
- High ratios of HOC1 to ClO_2
- Generation at high pH values (pH >11)
- Reaction mixtures that are highly acidic (pH <3)

Formation of Chlorite and Chlorate. Chlorite and chlorate are produced in varying ratios as by-products during chlorine dioxide disinfection and subsequent degradation. The primary factors affecting the relative concentrations of chlorine dioxide, chlorite, and chlorate in finished drinking water involve:

- Ratio of dosage applied to oxidant
- Exposure of water containing chlorine dioxide to sunlight
- Reactions between chlorine and chlorite (if free chlorine is used for distribution system residual maintenance)

Incomplete reaction or nonstoichiometric addition of the sodium chlorite and chlorine reactants during generation can result in unreacted chlorite in the chlorine dioxide feed stream. Dilute chlorine dioxide solutions are stable under low or zero oxidant-demand conditions. However, in the presence of organic material, chlorine dioxide will continue to react and degrade. The application of 2 mg/L chlorine dioxide is expected to produce 1 to 1.4 mg/L of chlorite.[11]

Sunlight may increase chlorate concentrations in uncovered storage basins containing water with chlorine dioxide residuals. Exposure to ultraviolet light from sunlight will also create potential reactions between chlorine dioxide and the bromide ion in uncovered basins.

In addition, chlorine dioxide degrades at greater than pH 9 to chlorite and chlorate according to the following reaction:

$$2ClO_2 + 2OH^- = ClO_2^- + ClO_3^- + H_2O \tag{19–14}$$

Chlorine dioxide should be added after the pH has been lowered when water treatment processes require a high pH, such as for softening.[12]

Organic DBPs Formed by Chlorine Dioxide. Chlorine dioxide generally produces few organic DBPs. The application of chlorine dioxide does not produce THMs and produces only a small amount of total organic halide (TOX).[9]

Chlorine Dioxide DBP Control Strategies. Once formed, chlorate is stable in finished drinking water. No known treatment exists for removing chlorate once it is formed. However, three strategies that have been proven effective for chlorite removal are:[13]

- Adding reduced-sulfur compounds, such as sulfur dioxide and sodium sulfite (not recommended), converting chlorite to chlorate.
- Applying either granular activated carbon (GAC) or powdered activated carbon (PAC).
- Adding reduced iron salts, such as ferrous chloride and ferrous sulfate, converting chlorite to chloride.

Disinfection with Chlorine Dioxide Chlorine dioxide is a strong primary disinfectant. Chlorine dioxide also maintains a residual level in low-oxidant-demand water and can be considered as a secondary disinfectant. *CT* values for *Giardia* and virus inactivation for primary disinfection are shown in Figure 19–9 and Figure 19–10, respectively.[2]

Storage and Feed Equipment Depending upon system size, sodium chlorite can be purchased in 55-gallon drums, 275-gallon nonreturnable totes, or in bulk quantities. A 30-day storage supply of sodium chlorite can easily be met for most small systems by using 55-gallon drums. Gaseous chlorine and hypochlorite solution are discussed in the chlorine section of this chapter.

Sodium chlorite solution feed pumps are commonly diaphragm-metering pumps for liquid feed rate control. If centrifugal pumps are used, the only acceptable packing material is Teflon. Flows from the metering pumps are frequently monitored with magnetic flow meters, mass flow meters, or rotameters for control. Back-flow prevention should be provided to avoid backfeeding through the metering pump to the storage tank. Sodium chlorite is extremely reactive, especially in the dry form, and care should be taken to protect against potentially explosive conditions.

Outside storage of 25 percent sodium chlorite solutions (or greater) is not recommended where temperatures drop below 4°C (40°F); stratification of the solution may

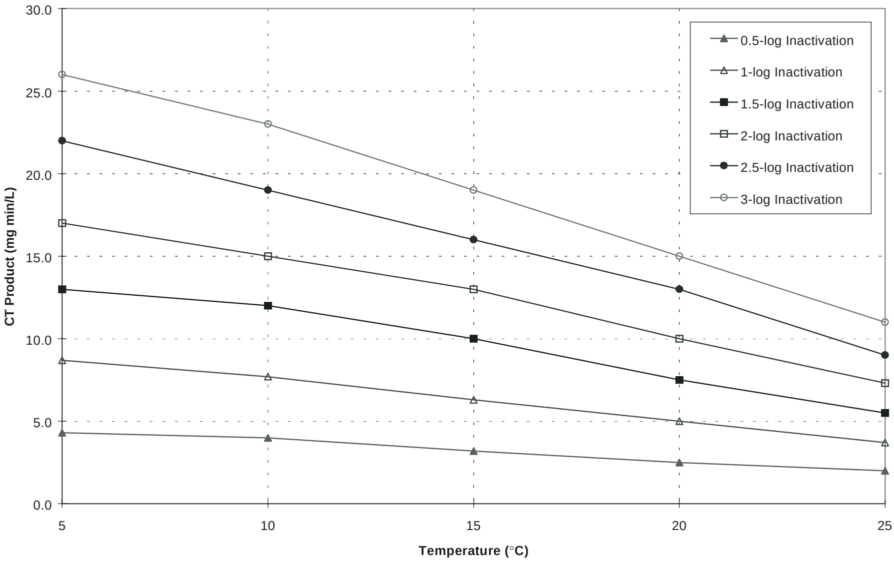


Fig. 19–9. *CT* values for inactivation of *Giardia* cysts by chlorine dioxide (*Source:* Reference 2)

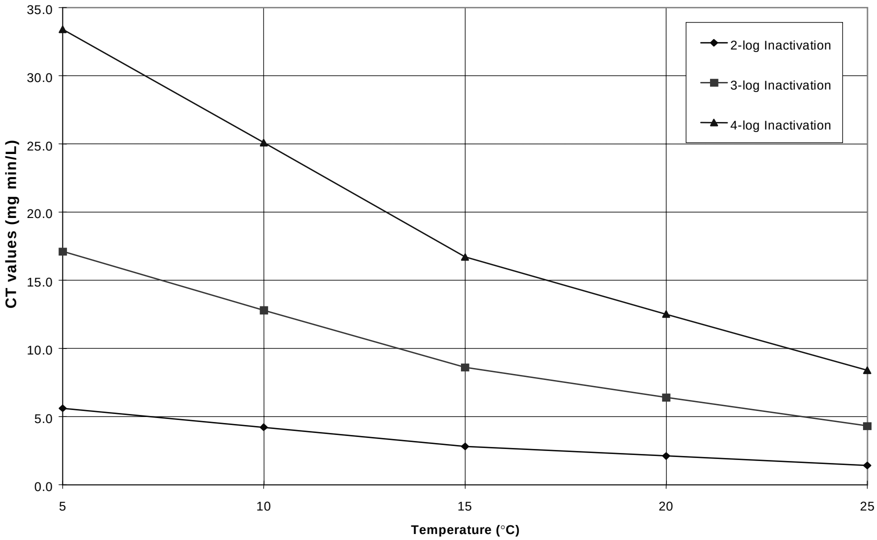


Fig. 19–10. *CT* values for inactivation of viruses by chlorine dioxide (*Source:* Reference 2)

occur. Also, any ice formation may, at this temperature, damage the storage tanks. Storage of sodium chlorite solution for long periods in hot climates should be avoided, since sodium chlorite decomposition will occur more rapidly at higher temperatures.

Chlorine dioxide solution concentrations below about 10 g/L do not produce vapors in high enough concentrations to present an explosion hazard under most ambient conditions of temperature and pressure. In conventional sodium chlorite generator systems, chlorine dioxide solution concentrations rarely exceed 4 g/L for temperatures less than 40°C.

Operational Considerations The requirements of the treatment facility's primacy agency should be consulted when selecting disinfectants. For the application of chlorine dioxide, certain states have their own additional operational, maintenance, and monitoring requirements. State requirements for chlorine dioxide must be reviewed to determine the overall cost-effectiveness of its use. Analytical testing and reporting requirements can have significant labor and cost impacts.

A typical chlorine dioxide generation system includes the following components:

- Chlorine storage and feed system (either gaseous chlorine or hypochlorite solution)
- Sodium chlorite storage and feed system
- Chlorine dioxide generator
- Chlorine dioxide solution, distribution piping, and application equipment

Generator Operation. The ideal production of 1.0 pound of chlorine dioxide requires 0.5 pounds of chlorine and 1.34 pounds of pure sodium chlorite. Manual chlorine dioxide feed system can be used where chlorine dioxide usage remains constant. In this case, the reagent chemical feed rates are manually set for the desired chlorine dioxide output to obtain the maximum chlorine dioxide yield. Some generating systems can produce dilute chlorine dioxide solutions with 95 percent purity at full design capacity. However, as the chlorine dioxide usage varies, the purity of the dilute solution may be reduced. Chlorine dioxide generators can also be provided with automated control to match chlorine dioxide feed rates to changes in flow (flow paced) and chlorine dioxide demand (residual control). The method of generator modulation used to meet a demand setpoint varies with the manufacturer. For automatic control of chlorine dioxide output, the turndown capacity is limited to typically 20 percent of rate capacity. Again, chlorine dioxide solution purity can vary when the usage rate is changed significantly. Supply water alkalinity, operating temperature, and pH also can affect the purity of the chlorine dioxide solution. The ratio of reagent chemicals should be routinely adjusted for optimum operation.

Feed Chemicals. Typically, solutions of 25 percent active sodium chlorite or less are used by chlorine dioxide generators. The major safety concern for solutions of sodium chlorite is the unintentional and uncontrollable release of high levels of chlorine dioxide. Explosive levels may be reached by accidental acidification of the sodium chlorite. The feedstock acid used by some of the generators is only one source of accidental release of chlorine dioxide gas. Mixing sodium chlorite with large amounts of any reducing agent (such as sodium bisulfite) or oxidizable material (such as powdered activated carbon or organic solvents) also represents a significant hazard.

Crystallization is another concern when handling and storing sodium chlorite solutions. Crystallization occurs as a result of reduced temperature and/or increased concentration. The crystals formed will plug pipelines, valves, pumps, and other equipment. Sodium chlorite solution leaks from tanks, piping, or pumps should not be allowed to evaporate to a powder. When dried, sodium chlorite becomes a fire hazard and can ignite on contact with combustible materials. Fires caused by sodium chlorite result in the release of molecular oxygen, and appropriate techniques are required to extinguish closed containers or large amounts of dry sodium chlorite.

Stratification of sodium chlorite in holding tanks can also occur, and this influences the purity of the chlorine dioxide solution. Stratification causes sodium chlorite concentrations to change from high to low density as it is fed. This change in sodium chlorite concentration affects the reagent chemical ratios, which impact the resulting purity.

Ozone

Ozone is a powerful oxidant, second only to the hydroxyl free radical among chemicals typically found in water treatment. Therefore, it is capable of oxidizing many organic and inorganic compounds in water. These reactions with organic and inorganic compounds cause an ozone demand in the water treated, which must be satisfied during water ozonation.

Ozone decomposes spontaneously during water treatment by a complex mechanism that involves the generation of hydroxyl radicals. As shown in Figure 19–11, ozone reacts in two modes in aqueous solution:[14]

- Direct oxidation of compounds by aqueous ozone ($O_{3(aq)}$)
- Oxidation of compounds by hydroxyl radicals produced during the spontaneous decomposition of ozone

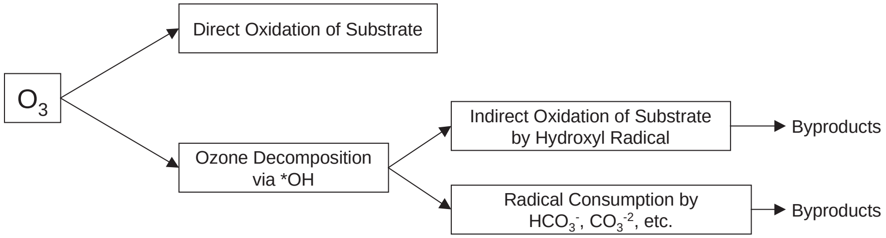


Fig. 19–11. Oxidation reactions during ozonation of water (*Source:* Reference 14)

Ozone and hydroxyl radicals compete for compounds to oxidize. The direct oxidation with aqueous ozone is relatively slow (compared to hydroxyl oxidation), but the concentration of aqueous ozone is relatively high. On the other hand, the hydroxyl radical reaction is fast, but the concentration of hydroxyl radicals under normal ozonation conditions is relatively small. By increasing the hydroxyl radical concentration, the overall oxidation rate can be increased. This technique is utilized in the advanced oxidation processes to provide more effective oxidation of certain compounds.

Ozone and hydroxyl radicals decompose rapidly in water. The ozone demand or decay is associated with:

- Reactions with natural organic matter (NOM) in the water. The oxidation of NOM leads to the formation of aldehydes, organic acids, and aldo- and ketoacids.[11] Organic oxidation by-products are generally more amenable to biological degradation and appear as assimilable organic carbon (AOC).
- Reactions with synthetic organic compounds (SOC) that can be mineralized under favorable conditions, usually by hydroxyl radicals.
- Oxidation of bromide ion, which leads to the formation of hypobromous acid, bromate, brominated organics, and bromamines. These reactions often lead to the formation of brominated DBPs.
- Bicarbonate or carbonate ions, which are commonly measured as alkalinity, and will scavenge the hydroxyl radicals and form carbonate radicals.[15,16] These reactions are of importance for advanced oxidation processes where the radical oxidation pathway is preferred.

Ozone is manufactured on-site using an ozone generator. The generated ozone-rich gas is then dissolved into the liquid stream. Generator output is heavily dependent on the oxygen content of the feed gas. By using a gas source with more oxygen than air, such as pure oxygen or enriched air using molecular sieves, the generator output can be improved. Generator design is rapidly evolving, and new generators regularly enter the market.

Ozone is formed by combining an oxygen atoms according to the following stoichiometry:

$$3O_2 \Leftrightarrow 2O_3 \qquad (19\text{–}15)$$

This reaction is endothermic and requires a considerable input of energy.

A corona discharge, also known as silent electrical discharge, is commonly used to generate ozone. In this process, an oxygen-containing gas passes through two electrodes separated by a dielectric and an air gap. A voltage is applied to the electrodes, causing an electron flow across the air gap. These electrons provide the energy to disassociate the oxygen molecules, leading to the formation of ozone.

Ozone Disinfection Facilities Ozone systems have four basic components, as shown in Figure 19–12: a gas feed system, an ozone generator, an ozone contactor, and an off-gas destruction system. The gas feed system provides a clean, dry source of oxygen to the generator. The ozone contactor transfers the ozone-rich gas into the water to be treated, and provides contact time for disinfection (or other reactions). The final process, off-gas destruction, is required as ozone is toxic in the concentrations present in the off-gas. Some plants include an off-gas recycle system that returns off-gas to the first contact chamber to reduce the ozone demand in the subsequent chambers. Some systems also include a quench chamber to remove ozone residual in solution.

Gas Feed Systems. Two types of gas feed systems are used. One uses pure oxygen and the other air.

PURE OXYGEN SYSTEMS. Pure oxygen systems are used to produce ozone at high concentration. The gas feed is either pure oxygen or oxygen-enriched air, or purchased as liquid oxygen. The commonly used gas systems are as follows:

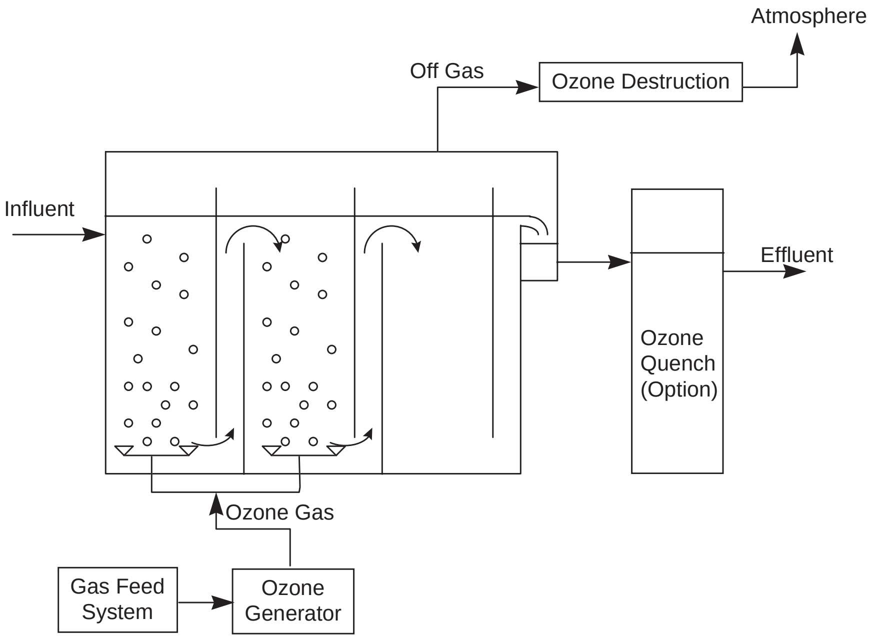


Fig. 19–12. Simplified ozone facilities schematic

- Cryogenic oxygen generation, where oxygen is distilled under pressure from air. Cryogenic oxygen generation is economically feasible only for large systems.
- Pressure swing adsorption (PSA), where a special molecular sieve is used under pressure to remove nitrogen, carbon dioxide, water vapor, and other compounds to produce 90–95 percent pure oxygen.
- Vacuum swing adsorption (VSA), which is similar to the PSA system, operating under vacuum to reduce energy cost.
- Liquid oxygen (LOX) systems, which rely on oxygen delivery from a chemical supplier. Liquid oxygen is maintained on-site in a cryogenic tank. Pressure regulators and evaporators are used to feed oxygen to the ozone generator. Liquid storage tanks range from 45-gallon dewers to 5,000-gallon bulk tanks.

AIR FEED SYSTEMS. Air feed systems for ozone generators are fairly complicated, as the air must be cleaned to protect the ozone generator. Air must be clean and dry, with a dew point of at least −60°C (−80°F) and free of contaminants. Air preparation systems typically consist of air compressors, filters, dryers, and pressure regulators. Figure 19–13 is a schematic of an air preparation system.

Air preparation systems include the following:

- Filters to remove particles above 1 m and oil droplets above 0.05 m
- Granular activated carbon filter or similar device if hydrocarbons are present in feed gas
- Air compressors can be classified as low (<30 psig), medium (30 to 60 psig), or high (>60 psig) pressure. Rotary lobe, rotary screw, centrifugal, vane, liquid ring, and reciprocating compressors can be used (see Table 19–9). Oil-less compressors are often used to avoid hydrocarbons in the feed gas.

Table 19–10 is a comparison of the advantages and disadvantages of air and pure oxygen gas feed systems.

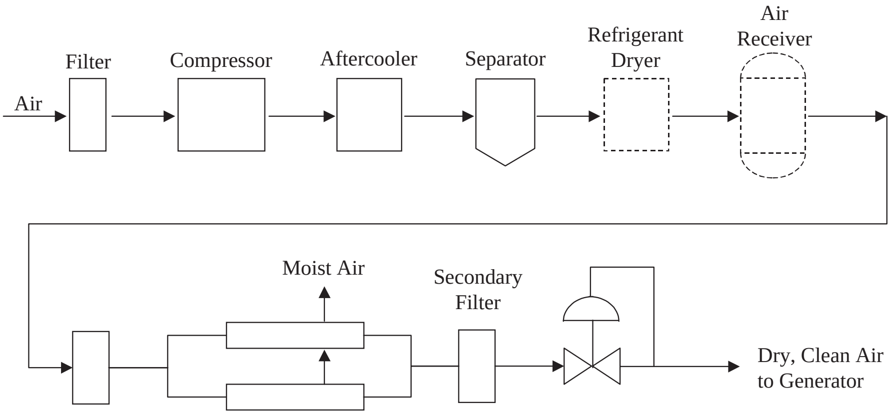


Fig. 19–13. Schematic of a typical air preparation system for ozone generation

TABLE 19–9. Types of Compressors Used in Air Preparation Systems

Compressor Type	Pressure	Volume	Comments
Rotary lobe	Low—15 psi	Constant or variable with unloading	Common in Europe
Centrifugal	30–100 psi depending on no. of stages	Variable, high volume	Medium efficiency, cost effective in high volumes
Rotary screw	50 psi (single stage) to 100 psi (two-stage)	Variable with unloading	Slightly more efficient than rotary lobe, draws approximately 40% of full load power in unloaded state, available in nonlubricated design for larger capacities.
Liquid ring	10–80 psi	Constant volume	Does not require lubrication or aftercooler, relatively inefficient, common in United States.
Vane	High—to 100 psi	Constant or variable	Relatively inefficient, not common in U.S.

TABLE 19–10. Comparison of Air and High-Purity Oxygen Feed Systems

Source	Advantages	Disadvantages
Air	• Commonly used equipment • Proven technology • Suitable for small and large systems	• More energy consumed per ozone volume produced • Extensive gas-handling equipment required • Maximum ozone concentration of 3–5%
Oxygen (general)	• Higher ozone concentration (8–12%) • Approximately doubles ozone concentration for same generator	• Safety concerns • Oxygen-resistant materials required
LOX	• Less equipment required • Simplest to operate and maintain	• Variable LOX costs • Storage of oxygen on-site (fire codes) • Loss of LOX in storage when not in use
High-purity oxygen generation	• Equipment similar to air-preparation systems • Feasible for large systems • Can store excess oxygen to meet peak demands	• More complex than LOX, similar to air • Extensive gas handling equipment required • Capital intensive • Complex systems to operate and maintain

Ozone Generators Ozone is produced in a generator by an electric spark called the corona discharge. Electrons moving through the oxygen-rich gas collide with oxygen molecules and generate ozone molecules. Ozone generation is determined by the following:

- Applied voltage. A high voltage increases the ozone yield.
- Discharge gap. The gap determines the voltage required to produce a corona discharge.
- Frequency. Operating at higher frequency increases ozone production, but also increases the heat production. Heat accelerates the ozone decomposition.
- Dielectric material. A high dielectric constant and thin dielectric will produce the higher ozone yield. However, a thin dielectric is more likely to break.
- Temperature. Because ozone decomposition increases as the temperature rises, ozone generators are equipped with cooling water. Small generators can be air cooled. Approximately 85 percent of energy input into an ozone generator is lost as heat.

The ozone generator designer must balance the increased yield and operational reliability and reduced maintenance.

Ozone generators are classified by the frequency of the power applied to the electrodes. Low-frequency (50 or 60 Hz) and medium-frequency (60 to 1000 Hz) generators are most common found in the water industry. Table 19–11 is a comparison of the two types of generators. Medium-frequency generators are more efficient and can produce higher ozone concentrations, but they generate more heat and require a more complicated power supply to step up the frequency supplied by utility power.

Ozone Dissolution Ozone gas is sparingly soluble in water. The saturation concentration of pure ozone at 20°C is approximately 570 mg/L. Air-fed ozone generators produce only 0.5 to 2.5 percent ozone. The saturation solubility in practical systems is therefore 5 to 50 mg/L.

Ozone transfer into water is commonly achieved in one of three types of mass transfer systems:

- Bubble contactor
- Eductor
- Turbine mixer

Bubble Diffuser Contactor. The bubble diffuser contactor is commonly used for ozone contacting in the United States. Figure 19–14 illustrates a typical three-stage ozone bubble diffuser contactor. This illustration shows a countercurrent flow configuration (ozone and water flowing in opposite directions), a concurrent flow configuration (ozone and water flowing in the same direction), and an alternating cocurrent/countercurrent arrangement. The number of stages can vary from two to six for ozone disinfection, with one or two stages typically used for dissolution and the remainder for contacting. Bubble diffuser contactors are typically constructed with 18- to 22-ft water depths to achieve 85 to 95 percent ozone-transfer efficiency.

To generate bubbles, bubble diffuser contactors use ceramic diffusers that are either rod-type or disc-type. Design considerations for the diffusers layout include:

TABLE 19–11. Comparison of Primary Characteristics of Low-, Medium-, and High-Frequency Ozone Generators

Characteristic	Low Frequency (50–60 Hz)	Medium Frequency (up–1,000 Hz)	High Frequency (>1,000 Hz)
Degree of electronics sophistication	Low	High	High
Peak voltages	19.5	11.5	10
Turndown ratio	5:1	10:1	10:1
Cooling water required (gal/lb of ozone produced)	0.5–1.0	0.5–1.5	0.25–1
Typical application range	<500 lb/day	To 2,000 lb/day	To 2,000 lb/day
Operating concentrations			
wt—% in air	0.5–1.5%	1.0–2.5%$^+$	1.0–2.5%$^+$
wt—% in oxygen	2.0–5.0%	2–12%	2–12%
Optimum ozone production (as a proportion of total generator capacity)	60–75%	90–95%	90–95%
Optimum cooling water differential	8°–10°F	5°–8°F	5°–8°F
Power required (kWh/lb O_3)	Air feed: 8–12 O_2 feed: 4–6	Air feed: 8–12 O_2 feed: 4–6	Air feed: 8–12 O_2 feed: 4–6
Air feed system power requirements (kWh/lb O_3)	5–7	5–7	5–7

Source: Reference 17, with modifications.

- Gas flow range of 0.5 to 4.0 scfm/diffuser
- Maximum headloss of 0.5 psig
- Porosity of 35 to 45 percent
- Avoid channeling by proper spacing of diffusers
- Clogging, which can occur due to iron and manganese oxidation

Table 19–12 summarizes the advantages and disadvantages of the bubble diffuser contactor.[18]

Eductor Dissolution. The eductor contacting method is commonly used in Europe and Canada.[18] Ozone is injected into a water stream under negative pressure, which is generated in a venturi section, pulling the ozone into the water stream. Figure 19–15 illustrates typical in-line and sidestream ozone injection systems. Ozone is either injected directly into the waterstream or injected into a sidestream and then blended with the main flow.

The gas-to-liquid ratio should be less than 0.067 cfm/gpm to optimize ozone-transfer efficiency.[18] This is feasible only in systems that require low ozone dosages and use ozone gas concentrations greater than 6 percent by weight.[19] High concentration ozone gas can be generated using a medium-frequency generator and pure oxygen as the feed gas.

Table 19–13 summarizes the advantages and disadvantages of injection contacting.[18]

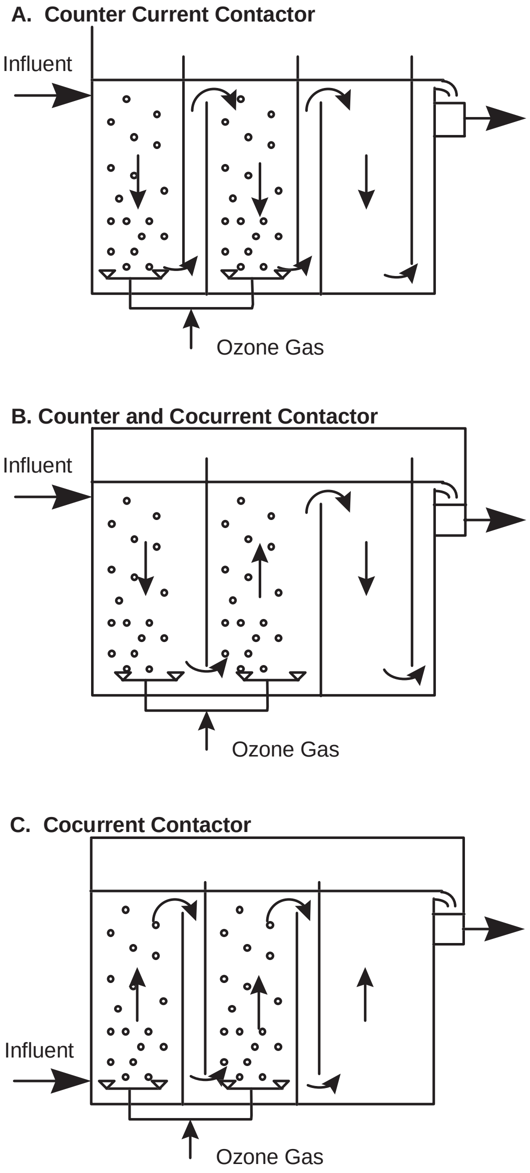


Fig. 19–14. Ozone bubble contactor

TABLE 19–12. Bubble Diffuser Contactor Advantages and Disadvantages

Advantages	Disadvantages
No moving parts	Deep contact basins
Effective ozone transfer	
Low hydraulic headloss	Vertical channeling of bubbles
Operational simplicity	

Source: Adapted from Reference 18.

Ozone Contactors Once ozone is dissolved in the water, a contact period is required to achieve the desired *CT* or disinfection intensity. Contactors include the following:

- Bubble contactor
- Mixed contactors
- Reactive flow or pipe contactors

The bubble contactor and turbine mixed contactor can serve both to dissolve ozone and provide contact time. Reactive flow contactors can be a simple plug flow container such as a baffled reactor or pipe.

The contact time requirements are set forth in regulations as the *CT* required to achieve the desired log inactivation. The contact time, *T,* in the *CT* expression is the T_{10} value. T_{10} is the time required for 10 percent of a dye slug to pass through the

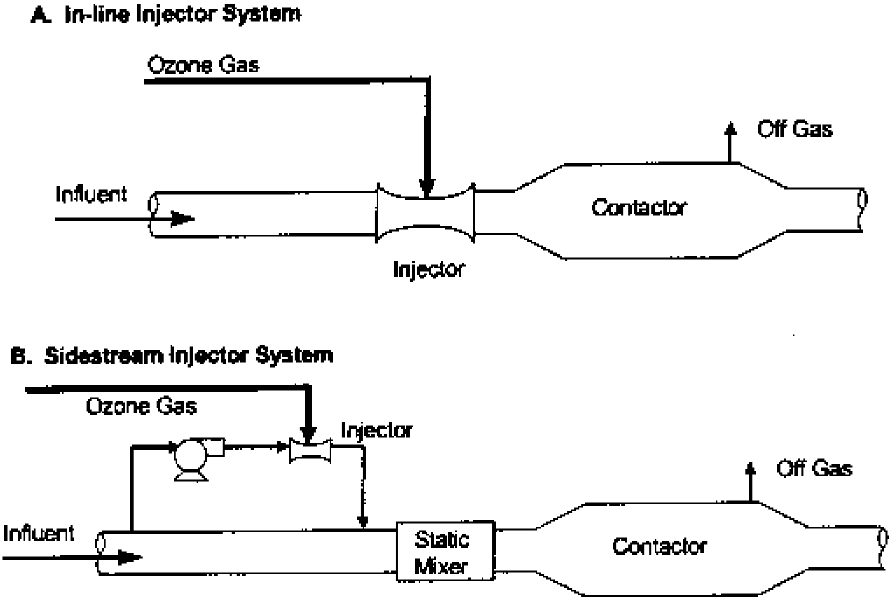


Fig. 19–15. Sidestream ozone injection system

TABLE 19–13. Injection Contacting Advantages and Disadvantages

Advantages	Disadvantages
Injection and static mixing have no moving parts	Additional headloss (energy usage) due to static mixers that may require pumping
Very effective ozone transfer	Turndown capability limited by injection system
Contactor depth less than bubble diffusion	More complex operation

Source: Adapted from Reference 18.

contactor. In other words, 90 percent of the liquid remains in the contactor for longer than T_{10}.

Off-Gas Destruction The concentration of ozone in the off-gas from a contactor can readily exceed 1,000 ppm of ozone. This is above the fatal concentration. Off-gas must therefore be collected and the ozone destroyed prior to release to the atmosphere. Ozone is readily destroyed by a catalyst operating at high temperature to prevent moisture buildup. The off-gas destruct unit is designed to reduce the concentration to 0.1 ppm by volume, the current limit set by OSHA for exposure in an 8-hour period. A blower is used on the discharge side of the destruct unit to pull the air from the contactor, placing the contactor under a slight vacuum to ensure that no ozone escapes.

Instrumentation Instrumentation must be provided for ozone systems to protect both personnel and the equipment. The typical instrumentation includes the following:

- Gas phase ozone detectors in spaces such as generator rooms where ozone gas and personnel are routinely present
- A gas phase ozone detector on the outlet from the off-gas destruct unit
- A dew point detector on the feed gas supply
- Flow switches on the cooling water supply to ensure continued supply
- A pressure switch to prevent overpressurization
- Temperature monitoring of cooling time and ozone gas
- Ozone residual concentration in the contactor. Residual measurements can be taken at several points to calculate *CT* values

Distribution System Impacts Ozonation of organic compounds increases the biodegradability of the organic compounds. This is a benefit if GAC or some other filter follows the ozone application for organic removal. This will reduce the organic concentrations in the water and reduce the potential for producing DBPs in subsequent chlorination. However, the increase in biodegradable organics also increases the potential for biological growth in the distribution system. The designer must be aware of these impacts and take appropriate action to manage the bacterial regrowth. In organic-containing waters, ozone is typically applied upstream of granular filters designed to remove AOC. The design of filters to remove AOC is discussed in Chapter 12, ‘‘Filtration.’’

Advanced Oxidation Processes

Advanced oxidation processes (AOPs) generate highly reactive hydroxyl radicals to oxidize various compounds in the water. See Chapter 16, ‘‘Oxidation,’’ for a discussion of AOPs.

Advanced organic processes will also provide significant disinfection. A key issue with the use of AOPs as a disinfection process is that the process does not leave a measurable disinfectant residual. Therefore, it is not possible to calculate *CT* values comparable to those of other disinfectants.

AOPs are commonly used to provide oxidation of some undesirable compound. Disinfection is of secondary importance. Ozone systems are often designed for disinfection and/or for oxidation. Table 19–14 summarizes the key differences between ozone and perozone as it relates to its application in drinking water disinfection.

Studies have indicated that the disinfection effectiveness of peroxone and ozone are comparable.[20–22] These studies showed that the f2 and MS-2 coliphages were comparable in their resistance to ozone and peroxone. Also, results of the *E. coli* and HPC studies showed that peroxone and ozone had comparable results with respect to bacteria inactivation.

Table 19–15 lists *CT* values derived for inactivation of *Giardia muris* cysts by ozone and peroxone from another study conducted by MWD. The results of this study suggest that peroxone is slightly more potent than ozone based on the fact that *CT* values for ozone were greater than those for peroxone.

Ultraviolet Radiation

Unlike most disinfectants, ultraviolet (UV) radiation does not inactivate microorganisms by chemical interaction. UV radiation inactivates organisms by absorption of light, which causes a photochemical reaction that alters molecular components essential to cell function. As UV rays penetrate the cell wall of the microorganism, the energy reacts with nucleic acids and other vital cell components, resulting in injury or death of the exposed cells. There is ample evidence to conclude that if sufficient dosages of UV energy reach the organisms, UV can disinfect water to whatever degree

TABLE 19–14. Comparison Between Ozone and Peroxone Oxidation

Process	Ozone	Peroxone
Ozone decomposition rate	‘‘Normal’’ decomposition producing hydroxyl radical as an intermediate product	Accelerated ozone decomposition increases the hydroxyl radical concentration above that of ozone alone.
Ozone residual	Relatively long-lived	Very short-lived due to rapid decomposition.
Oxidation path	Usually direct aqueous ozone oxidation	Primarily hydroxyl radical oxidation
Disinfection ability	Excellent	Excellent
Ability to detect residual for disinfection monitoring	Good	Poor. Cannot calculate CT value for disinfection credit.

TABLE 19–15. Calculated CT Values for the Inactivation of *Giardia muris* with Ozone and Peroxone

Inactivation (%)	Ozone C_1T_1* (mg · min/L)	Ozone C_2T_2* (mg · min/L)	Peroxone† C_1T_1* (mg · min/L)	Peroxone C_2T_2* (mg · min/L)
90	1.6	2.8	1.2	2.6
99	3.4	5.4	2.6	5.2

Source: See Reference 20. Results at 14°C. (Reprinted with permission from *Environ. Sci. Technol.*, Vol. 23, No. 6, p. 744. Copyright © 1989 American Chemical Society.)

* C_1, ozone residual; C_2 (ozone dose + ozone residual)/2; T_1 and T_2 time (in minutes) to reach 10 percent and 50 percent breakthrough, respectively.

† The H_2O_2/O_3 ratio for all results was 0.2.

is required. However, there have been some public health concerns with respect to the overall efficiency of UV to disinfect potable water.

Based on the available research literature, it appears that although exceptional for disinfection of small microorganisms such as bacteria and viruses, UV doses required to inactivate larger protozoa such as *Giardia* and *Cryptosporidium* are several times higher than for bacteria and virus inactivation.[4,19] Recent improvement in protozoan enumeration techniques suggest, however, that protozoa can be inactivated by disruption of their reproductive process at much lower UV doses than was previously thought.

UV radiation quickly dissipates into water, to be absorbed or reflected off material within the water. As a result, no residual is produced. This process is attractive from a DBP formation standpoint; however, a secondary chemical disinfectant is required to maintain a residual throughout the distribution system.

UV radiation energy waves are the range of electromagnetic waves 100 to 400 nm long (between the X-ray and visible light spectrums). The division of UV radiation may be classified as Vacuum UV (100–200 nm), UV-C (200–280 nm), UV-B (280–315 nm) and UV-A (315–400 nm). In terms of germicidal effects, the optimum UV range is between 245 and 285 nm. UV disinfection utilizes either low-pressure lamps that emit maximum energy output at a wavelength of 253.7 nm; medium-pressure lamps that emit energy at wavelengths from 180 to 1370 nm; or lamps that emit at other wavelengths in a high-intensity "pulsed" manner.

UV Disinfection Reactions The degree to which the destruction or inactivation of microorganisms occurs by UV radiation is directly related to the UV dose. The UV dosage is calculated as:

$$D = I \cdot t \qquad (19\text{–}16)$$

where:

D = UV Dose, $mW \cdot s/cm^2$
I = Intensity, mW/cm^2
t = Exposure time, s

Research indicates that when microorganisms are exposed to UV radiation, a constant fraction of the living population is inactivated during each progressive increment in time. This dose–response relationship for germicidal effect indicates that high-intensity UV energy over a short period of time would provide the same kill as a lower-intensity UV energy at a proportionally longer period of time.

The UV dose required for effective inactivation is determined by site-specific data relating to the water quality and log removal required. Based on first-order kinetics, the survival of microorganisms can be calculated as a function of dose and contact time.[4,23] For high removals, the remaining concentration of organisms appears to be solely related to the dose and water quality, and not dependent on the initial microorganism density. Tchobanoglous[24] suggested the following relationship between coliform survival and UV dose:

$$N = f \cdot D^n \tag{19–17}$$

where:

N = effluent coliform density, /100 mL
D = UV dose, $\text{mW} \cdot \text{s/cm}^2$
n = empirical coefficient related to dose
f = empirical water quality factor

The empirical water quality factor reflects the presence of particles, color, etc. in the water. For water treatment, the water quality factor is expected to be a function of turbidity and transmittance (or absorbance).

UV Disinfection Variables Since UV radiation is energy in the form of electromagnetic waves, its effectiveness is not limited by chemical water quality parameters. For instance, it appears that pH, temperature, alkalinity, and total inorganic carbon do not impact the overall effectiveness of UV disinfection.[7] However, hardness may cause problems with keeping the lamp sleeves clean and functional. The presence, or addition, of oxidants (e.g., ozone and/or hydrogen peroxide) enhances UV radiation effectiveness. The presence of some dissolved or suspended matter may shield microorganisms from the UV radiation. For instance, iron, sulfites, nitrites, and phenols all absorb UV light.[19] Accordingly, the absorbance coefficient is an indication of this demand and is unique for all waters. As a result, specific ‘‘design’’ parameters vary for individual waters and should be determined empirically for each application.

UV demand of water is measured by a spectrophotometer set at a wavelength of 254 nm, using a 1-cm-thick layer of water. The resulting measurement represents the absorption of energy per unit depth, or absorbance. Percent transmittance is a parameter commonly used to determine the suitability of UV radiation for disinfection. The percent transmittance is related to the absorbance (A) by the equation:

$$\text{Percent Transmittance} = 100 \times 10^{-A} \tag{19–18}$$

Generation of UV Radiation Producing UV radiation requires electricity to power UV lamps. The lamps typically used in UV disinfection consist of a quartz tube filled with an inert gas, such as argon, and small quantities of mercury. Ballasts control the power to the UV lamps.

UV Lamps. UV lamps operate in much the same way as fluorescent lamps. UV radiation is emitted from electron flow through ionized mercury vapor to produce UV energy in most units. The difference between the two lamps is that the fluorescent lamp bulb is coated with phosphorus, which converts the UV radiation to visible light. The UV lamp is not coated, so it transmits the UV radiation generated by the arc.[4]

Both low-pressure and medium-pressure lamps are available for disinfection applications. Low-pressure lamps emit their maximum energy output at a wavelength of 253.7 nm, whereas medium-pressure lamps emit energy with wavelengths ranging from 180 to 1,370 nm. The intensity of medium-pressure lamps is much greater than that of low-pressure lamps. Thus, fewer medium-pressure lamps are required for an equivalent dosage. For small systems, the medium-pressure system may consist of a single lamp. Although both types of lamps work equally well for inactivation of organisms, low-pressure UV lamps are recommended for small systems because of the reliability associated with multiple low-pressure lamps[19] as opposed to a single medium-pressure lamp, and for adequate operation during cleaning cycles.

Recommended specifications for low-pressure lamps include:[19]

- L-type ozone-free quartz
- Instant start (minimal delay on start-up)
- Designed to withstand vibration and shock
- Standard nonproprietary lamp design

Typically, low-pressure lamps are enclosed in a quartz sleeve to separate the water from the lamp surface. This arrangement is required to maintain the lamp surface operating temperature near its optimum of 40°C. Although Teflon sleeves are an alternative to quartz sleeves, quartz sleeves absorb only 5 percent of the UV radiation, while Teflon sleeves absorb 35 percent.[25] Therefore, Teflon™ sleeves are not recommended.

Ballasts. Ballasts are transformers that control the power to the UV lamps. Ballasts should operate at temperatures below 60°C to prevent premature failure. Typically, the ballasts generate enough heat to warrant cooling fans or air-conditioning.[4]

Two types of transformers are commonly used with UV lamps—namely, electronic and electromagnetic. Electronic ballasts operate at a much higher frequency than electromagnetic ballasts, resulting in lower lamp operating temperatures, less energy use, less heat production, and longer ballast life.[19] Typical ballast selection criteria include:[19]

- Underwriters Laboratories (UL) approval
- Compatibility with UV lamps
- Waterproof enclosure in remote location

UV Reactor Design Most conventional UV reactors are available in two types—namely, closed vessel and open channel. For drinking water applications, the closed vessel is generally the preferred UV reactor for the following reasons:[23]

- Smaller footprint
- Minimized pollution from airborne material

- Minimal personnel exposure to UV
- Modular design for installation simplicity

Figure 19–16 shows a conventional closed-vessel UV reactor. This reactor is typically used to treat flows up to 600 gallons per minute.

Additional design features for conventional UV disinfection systems include:

- UV sensors to detect any drop in UV lamp output intensity
- Alarms and shutdown systems
- Automatic or manual cleaning cycles
- Telemetry systems for remote installations

Hydraulic Design Considerations The major elements that should be considered in the hydraulic design of a UV closed-vessel reactor are: dispersion, turbulence, effective volume, residence time distribution, and flow rate.[23]

Poor geometry within the UV contactor (which creates spacing between lamps) can leave dead areas where inadequate disinfection occurs.[26] A key consideration to improving disinfection is to minimize the amount of dead spaces where limited UV exposure can occur. Plug flow conditions should be maintained in the contactor; however, some turbulence should be created between the lamps to provide radial mixing of flow. In this manner, flow can be uniformly distributed through the varying regions of UV intensity, allowing exposure to the full range of available UV radiation.[26]

UV systems typically provide contact times on the order of seconds. Therefore, it is extremely important that the system configuration limit the extent of short circuiting.

Dispersion. Dispersion is the characteristic of water elements to scatter spatially. The ideal UV reactor is plug flow, where water particles are assumed to discharge from the reactor in the same sequence they entered and each element of water passing through the reactor resides in the reactor for the same period of time. An ideal plug flow reactor has no dispersion and is approximated by a long tank with high length-to-width ratio in which dispersion is minimal.[23]

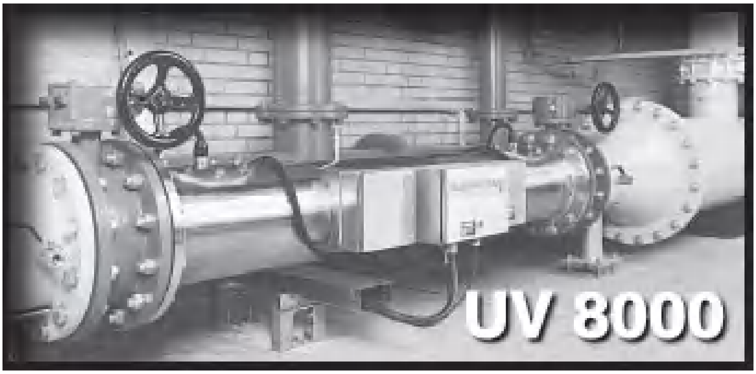


Fig. 19–16. Closed vessel UV reactor (Courtesy of Trojan Technologies Inc.)

Turbulence. In addition to plug flow characteristics, the ideal UV reactor has a flow that is turbulent radially from the direction of flow, to eliminate dead zones. This radially turbulent flow pattern promotes uniform application of UV radiation. A negative of having a radially turbulent flow pattern is that some axial dispersion results, thus disrupting the plug flow characteristics. Techniques such as misaligning the inlet and outlet, and using perforated stilling plates, have been employed to accommodate the contradicting characteristics of plug flow and turbulence.[23]

UV Application for Disinfection UV radiation is a physical disinfectant that leaves no residual. Therefore, it should be used only as a primary disinfectant followed by a chemical secondary disinfectant to protect the distribution system.

The most common point of application for UV radiation is the last step in the treatment process train just prior to the distribution system and after filtration. The use of UV disinfection has no impact on other processes at the water treatment facility.

UV Pathogen Inactivation

INACTIVATION MECHANISM. UV radiation is efficient at inactivating vegetative and sporous forms of bacteria, viruses, and other pathogenic microorganisms. Electromagnetic radiation in the wavelengths ranging from 240 to 280 nanometers (nm) effectively inactivates microorganisms by irreparably damaging their nucleic acid. The most potent wavelength for damaging deoxyribonucleic acid (DNA) is approximately 254 nm.[27] Other UV wavelengths, such as 200 nm, have been shown to exhibit peak absorbance in aqueous solutions of DNA;[28] however, there is no practical application for UV inactivation of microorganisms in the wavelength range from 190 to 210 nm.[23]

The germicidal effects of UV light involve photochemical damage to RNA and DNA within the microorganisms. Microorganism nucleic acids are the most important absorbers of light energy in the wavelength of 240 to 280 nm.[29] DNA and RNA carry genetic information necessary for reproduction; therefore, damage to either of these substances can effectively sterilize the organism. Damage often results from the dimerization of pyrimidine molecules. Cystosine (found in both DNA and RNA), thymine (found only in DNA), and uracil (found only in RNA) are the three primary types of pyrimidine molecules. Replication of the nucleic acid becomes very difficult once the pyrimidine molecules are bonded together due to the distortion of the DNA helical structure by UV radiation.[30] Moreover, if replication does occur, mutant cells that are unable to replicate will be produced.[23]

Two phenomena of key importance when using UV disinfection in water treatment are the dark repair mechanisms and the capability of certain organisms to photoreactivate following exposure to certain light wavelengths. Under certain conditions, some organisms are capable of repairing damaged DNA and reverting back to an active state in which reproduction is again possible. Typically, photoreactivation occurs as a consequence of the catalyzing effects of sunlight at visible wavelengths outside of the effective disinfecting range. The extent of reactivation varies among organisms. Coliform indicator organisms and some bacterial pathogens such as *Shigella* have exhibited the photoreactivation mechanism; however, viruses (except when they have infected a host cell that is itself photoreactive) and other types of bacteria cannot photoreactivate.[26,31,32] Because DNA damage tends to become irreversible over time, there is a critical period during which photoreactivation can occur. To minimize the effect of photoreactivation, UV contactors should be designed to either shield the process stream or limit the exposure of the disinfected water to sunlight immediately following disinfection.

Water Quality Impacts on UV Disinfection Efficiency. Several factors that are known to affect disinfection efficiency of UV are:

- Chemical and biological films that develop on the surface of UV lamps
- Dissolved organics and inorganics
- Clumping or aggregation of microorganisms
- Turbidity
- Color
- Short circuiting in water flowing through the UV contactor

Accumulation of solids onto the surface of the UV sleeves can reduce the applied UV intensity and, consequently, disinfection efficiency. In addition to biofilms caused by organic material, buildup of calcium, magnesium, and iron scales have been reported.[19] Waters containing high concentrations of iron, hardness, hydrogen sulfide, and organics are more susceptible to scaling or plating (i.e., the formation of a thin coat on unit surfaces), which gradually decreases the applied UV intensity. Scaling is likely to occur if dissolved organics are present and inorganic concentrations exceed the following limits:[19]

- Iron greater than 0.1 mg/L
- Hardness greater than 140 mg/L
- Hydrogen sulfide greater than 0.2 mg/L

A variety of chemical substances can decrease UV transmission,[33] including humic acids, phenolic compounds, and lignin sulfonates,[30] as well as chromium, cobalt, copper, and nickel. It has been reported that coloring agents, such as Orzan S, tea, and extract of leaves reduce intensity within a UV contactor.[34] In addition, iron, sulfites, nitrites, and phenols can absorb UV.[19]

MICROORGANISM CLUMPING AND TURBIDITY. Particles can affect the disinfection efficiency of UV by harboring bacteria and other pathogens, partially protecting them from UV radiation and scattering UV light (see Fig. 19–17). Typically, the low turbidity of groundwater results in minimal impact on disinfection efficiency. However, the higher turbidities of surface water can impact disinfection efficiency.

Similar to particles that cause turbidity, microorganism aggregation can impact disinfection efficiency by harboring pathogens within the aggregates and shade pathogens that would otherwise be inactivated.

Disinfection By-Products of UV Radiation The literature suggests that UV radiation of water can result in the formation of ozone or radical oxidants.[35,36] Because of this reaction, there is interest in determining whether UV forms by-products similar to those formed by ozonation or advanced oxidation processes.

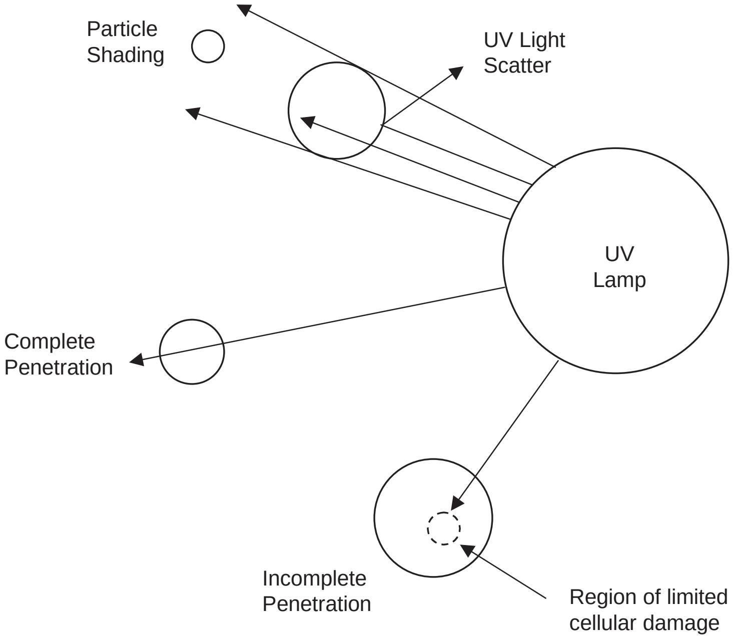


Fig. 19–17. Particle interactions that impact UV effectiveness (Courtesy of Dr. G. T. Tchobanoglous)

Groundwater. Malley et al. analyzed 20 groundwater samples for aldehydes and ketones before and after UV radiation.[37] Only one groundwater sample, which contained 24 mg/L nonpurgeable DOC and was highly colored, contained DBPs after exposure to UV. Low levels of formaldehyde were measured in duplicated experiments for this UV-treated groundwater sample. GC-ECD chromatographs before and after UV radiation for the other 19 groundwaters studied showed significant shifts or unknown peaks after exposure to UV.

Malley et al. also determined the influence of UV on DBP formation during subsequent chlorination.[37] To examine these effects, the 20 groundwater samples were subjected to simulated distribution system (SDS) DBP tests with chlorine, before and after UV radiation. The data indicate that UV radiation did not significantly alter the SDS/DBP formation by chlorine in the groundwaters studied.

Surface Water. UV radiation was found to produce low levels of formaldehyde in the majority of surface waters studied.[37] The highest formaldehyde concentrations, ranging up to 14 g/L, were observed in UV treatment of raw water, whereas trace levels (1 to 2 g/L) were found in UV treatment of conventionally treated water. Since formaldehyde formation was also observed for one of the groundwater samples, it appears that UV radiation of waters containing humic matter (i.e., color-producing, UV-absorbing organic macromolecules) will result in low levels of formaldehyde formation. Chromatographic examination of the surface water samples before and after UV radiation showed no other significant changes in the GC-ECD chromatograms.

Because of the chlorine demands of surface waters, higher chlorine dosages were required for post-disinfection following UV radiation. This resulted in larger DBP concentrations than in the ground-waters studied.[37] However, the overall effect of UV radiation on SDS/DBPs was insignificant. As in the groundwater studies, UV radiation did not significantly alter the total concentration or the speciation of the disinfection by-products (e.g., THMs, HAA5, HANs, or HKs).

DBP Formation with Chlorination and Chloramination Following UV Radiation. The question of whether UV radiation influences the rate of DBP formation by post-disinfection is important, and has been addressed by several studies. Two surface waters that produced significant concentrations of a wide variety of DBPs in previous tests were chosen as samples. With the chlorine residuals carefully monitored to ensure they were consistent for pre-UV and post-UV samples, the results of the experiments suggested that UV radiation did not significantly affect the rate of DBP formation.

Studies were only performed to determine the pentane extractable DBP formation rate of a surface water sample for varying pH conditions. The results showed that UV radiation did not affect the rate of chloroform formation at pH 8.0.[37] Similarly, UV did not affect the DBP formation rate at pH 5.0. At pH 8.0, chloroform was the only pentane extractable detected, whereas at pH 5.0, chloroform, bromodichloromethane, chlorodibromomethane, and 1,1,1-trichloroacetone were formed.

The effects of UV radiation on the DBP formation rate following chloramination were also tested in this study using a surface water sample.[37] Chloroform, trichloroacetic acid, dichloroacetonitrile (at low levels), and cyanogen chloride (at low levels) were the only detectable DBPs. Chloroform was the only compound formed at pH 8.0, and its rate of formation was not affected by UV radiation. At pH 5.0, chloroform and dichloroacetonitrile were formed, but their rate of formation was unaffected by UV radiation. Data showed that the effects of UV radiation on cyanogen chloride formation at pH 8.0 and pH 5.0 had no significant trends.

In summary, the DBP formation rate studies indicated that UV radiation did not significantly affect DBP formation rates when chlorine or chloramines were used as the post-disinfectant.

Operational Considerations On-site pilot plant testing is recommended to determine the efficiency and adequacy of UV disinfection for a specific quality of water. The efficiency test involves injecting select microorganisms into influent water and sampling effluent water to determine survival rates.

As previously discussed, some constituents that adversely interfere with UV disinfection performance by either scattering and/or absorbing radiation are iron, chromium, copper, cobalt, sulfites, and nitrites. Care should be taken with chemical processes upstream of the UV disinfection process to minimize increasing concentrations of these constituents, since disinfection efficiency may be adversely affected.

Equipment Operation. UV disinfection facilities should be designed to provide flexibility in handling varying flow rates. For lower flow rates, a single reactor vessel should be capable of handling the entire flow rate. A second reactor vessel with capacity equal to that of the first reactor vessel should be provided for redundancy should the first reactor vessel be taken out of service. For higher flow rates, multiple reactor vessels should be provided with lead/lag operation and flow split capacity to balance run time for each reactor vessel, if desired, and to avoid hydraulic overloading. Valves should be provided within the interconnecting piping to isolate one reactor vessel from another. There should also be a positive drainage system to remove water from within a reactor vessel when it is taken out of service.

UV Lamp Aging. The output of UV lamps diminishes with time. Two factors that affect their performance are:

- Polarization, which is the effect UV radiation has on the UV lamp that causes it to become opaque
- Electrode failure, which occurs when electrodes deteriorate progressively each time the UV lamp is cycled on and off

Frequent lamp cycling will lead to premature lamp aging. When determining the requirement for UV disinfection, a 30 percent reduction of UV output should be used to estimate end of lamp. Average life expectancy for low-pressure UV lamps is approximately 8,800 hours.

Quartz Sleeve Fouling. Fouling of the quartz sleeve reduces the amount of UV radiation reaching the water. The quartz sleeve has a transmissibility of over 90 percent when new and clean. Over time, the surface of the quartz sleeve that is in contact with the water starts collecting organic and inorganic debris (e.g., iron, calcium, silt), causing a reduction in transmissibility.[23] When determining the requirements for UV disinfection, a 30 percent reduction of UV transmission should be used to reflect the effect of quartz sleeve fouling.

Equipment Maintenance

UV LAMP REPLACEMENT. Adequate space should be provided around the perimeter of the reactor vessels to allow access for maintenance and replacement of UV lamps. With modular electrical fittings, lamp replacement consists of unplugging the pronged connection of the old lamp and plugging in the new.

QUARTZ SLEEVE CLEANING. Quartz sleeve cleaning may be accomplished by physical or chemical means. Physical alternatives include:

- Automatic mechanical wiper
- Ultrasonic devices
- High-water-pressure wash
- Air scour

Chemical cleaning agents include sulfuric and hydrochloric acid. A UV reactor vessel may contain one or more physical cleaning system with provision for an occasional chemical cleaning.

Standby Power Producing UV radiation requires electricity to power the electronic ballasts, which in turn power the UV lamps. Since disinfection is of utmost importance in producing potable water, the UV system should remain in service during periods of primary power failure. A dual power feed system or essential circuitry powered by a standby generator are typical ways to achieve the desired reliability. Each lowpressure UV lamp requires approximately 100 watts of standby power. A second precaution that should be considered is not powering the UV system from the same motor control center (MCC) that powers variable- frequency drives (VFDs). The electronic ballasts produce harmonics that may require mitigation (active harmonic filters) for the VFDs.

Interactive Disinfectants

In 1988, several reports appeared on the combined efficiency of some disinfectants on pathogen inactivation. Higher inactivation of pathogens was found by Worley and Williams[38] using a mixture of free chlorine and organic *N*-halamine, and by Alleman et al.[39] using free chlorine and sodium bromide. Others found similar results.

Interactive disinfectants can be very significant in water treatment because some of these combinations are effective for inactivating *Cryptosporidium*.[40] Interactive disinfectants for primary pathogen inactivation can include many different combinations of disinfectants:

- Chlorine followed by monochloramine
- Chlorine dioxide followed by chlorine
- Chlorine dioxide followed by chlorine dioxide
- Chlorine dioxide followed by monochloramine
- Ozone followed by chlorine
- Ozone followed by chlorine dioxide
- Ozone followed by monochloramine

Inactivation Mechanism The mechanism by which interactive disinfectants provide a synergistic effect is not clearly understood. Several hypotheses have been put forth.[41–44]

The prevailing thought is that the two disinfectants perform separate functions. For example, one disinfectant could react with the outer cell membrane, compromising the membrane integrity. The second disinfectant can now penetrate the cell and attack a functional group to inactivate the organism.

Finch presented the results of laboratory scale investigations of an AWWARF study into the efficiency of interactive disinfectants.[45] The preliminary results of an AWWARF study that investigated the application of multiple disinfectants was presented at a American Water Works Association Technology Transfer Conference in Portland, Oregon, in August 1997. The objectives of this study were to screen sequential chemical disinfectants (ozone, chlorine, chlorine dioxide, and monochloramine) for inactivation of *Cryptosporidium parvum*, *Giardia muris*, and *Bacillus cereus* and develop design criteria for *Cryptosporidium parvum* inactivation using the best combinations. The results of the study (see Table 19–16) show the improvement of the disinfection efficiency as a result of the interaction of the disinfectants.

The improved disinfection efficiency due to synergism is highly variable, ranging from negative (antagonistic) effects to positive enhancement of disinfection efficiency. The data at this time show that:

- Coliform bacteria inactivation appears to increase with interactive disinfectants.
- *Giardia* cyst inactivation appears to increase with interactive disinfectants.

TABLE 19–16. Synergistic Effects with Interactive Disinfectants

Organism	*C. parvum*			*G. muris*				
First disinfectant	O_3	ClO_2	ClO_2	ClO_2	ClO_2	O_3	O_3	O_3
Second disinfectant	ClO_2	Cl_2	NH_2Cl	Cl_2	NH_2Cl	Cl_2	NH_2Cl	NH_2Cl
First disinfectant only	0.8	1.4	1.5	0.8	0.8	0.8	0.8	0.6
Second disinfectant only	1.4	0	0	0.6	0.5	0.6	0.5	0.5
Combined inactivation	3.6	3	2.8	2	1.7	2.2	2.1	2.4
Synergy	1.4	1.6	1.3	0.6	0.4	0.8	0.8	1.3

(Adapted from *Control of Cryptosporidium Through Disinfection: Current State of the Art,* AWWARF Technology Conference, Portland, OR, August 1997)

- *Cryptosporidium* oocyst inactivation appears to increase with interactive disinfectants.
- Inactivation of spores appears neutral.

Interactive disinfection is emerging technology. *CT* credits for interactive disinfectants have not been established.

DISINFECTANT SELECTION

The Alternative Disinfectants and Oxidants Guidance Manual, authored by HDR Engineering for EPA, provides an outline for selecting alternative disinfectants.[6] The following discussion is taken from this outline. The selection of primary disinfectant is based on providing pathogen-free drinking water and avoiding production of DBPs. Residual disinfectant selection is based on the ability of the disinfectant to provide a long living residual and minimum production of DBPs.

Alternative Disinfectant Properties

Table 19–17 summarizes the key technical and regulatory considerations associated with the use of the various disinfectants. The table addresses the propensity of the disinfectant to react with organics to form DBPs and its efficiency as a disinfectant. The disinfectant efficiency is judged on the practical use of the particular disinfectant to achieve high inactivations without excessively high doses. The use as a secondary disinfectant is judged based on the ability of the disinfectant to maintain a residual in the distribution system.

Disinfection Strategy Evaluation

The selection of a disinfection strategy, as presented below, is divided into several tasks:

- Evaluate the effectiveness of the current disinfection system.
- Provide decision trees to select an alternative disinfectant, if a change is needed.
- Evaluate primary and secondary disinfection requirements.

TABLE 19–17. Summary of Disinfectant Properties

Condition	Chlorine	Ozone	Chlorine Dioxide	Permanganate	Chloramine	Ozone / Peroxide	Ultraviolet	Interactive Disinfectant
Produce THM with TOC	y	s	n	n	n	s	n	x
Produce oxidized organics	s	y	s	s	n	y	s	x
Produce halogenated organics	y	s	n	n	n	s	n	x
Produce inorganic by-products	n	s	y	n	n	s	n	x
MRDL applies	y	n	y	n	y	n	n	x
Lime-softening impacts	y	n	n	n	y	n	y	x
Turbidity impacts	n	s	n	n	n	s	y	x
Meet *Giardia:* <2.0 log	y	y	y	n	n	y	n	x
Meet *Giardia:* >2.0 log	n	y	y	n	n	y	n	y
Meet *Crypto:* <2.0 log	n	y	y	n	n	y	n	x
Meet *Crypto:* >2.0 log	n	y	n	n	n	y	n	y
Meet virus: <2.0 log	y	y	y	n	n	y	y	y
Meet virus: >2.0 log	y	y	y	n	n	y	y	y
Secondary disinfectant	y	n	s	n	y	n	n	y
Operator skill (1 = low; 5 = high)	1	5	5	1	2	5	3	2–5
Applicable to large utilities	y	y	y	y	y	y	n	y
Applicable to small utilities	y	y	y	y	y	y	y	y

Source: Reference 6. (Reprinted from *Proceedings of the 1998 Water Quality Technology Conference*, by permission. Copyright © 1998, American Water Works Association.)

y = yes; n = no; s = sometimes; ? = unknown; x = depend on selected disinfectants; - = no impact.

For simplicity, the decision tree is subdivided into four phases, as described below.

Evaluating the Current Primary Disinfection Practice Figure 19–18 shows the decision tree used to determine whether the present disinfectant can meet disinfection and by-product requirements. The decision points in Figure 19–18 include:

- **Meeting microbial limits.** Microbial limits are defined by the primary drinking water standards. The regulated pathogens include *Giardia lamblia*, *Legionella*, HPC, total coliform, and viruses. The requirements for these organisms and *Cryptosporidium* are modified in the IESWTR, Long Term-1 ESWTR, and Long Term-2 ESWTR. The disinfectant must be capable of meeting the inactivation requirements of disinfection. If not, the plant must determine if the current disinfectant can meet the microbial inactivation requirements solely by operational changes. A change may be to move the application point, increase dose, increase contact time, or adjust pH. If not, a new disinfectant is needed.
- **Meeting DBP limits.** The secondary limit imposed on disinfectant usage is the ability to meet DBP requirements. At this decision point, the type of DBP is deter-

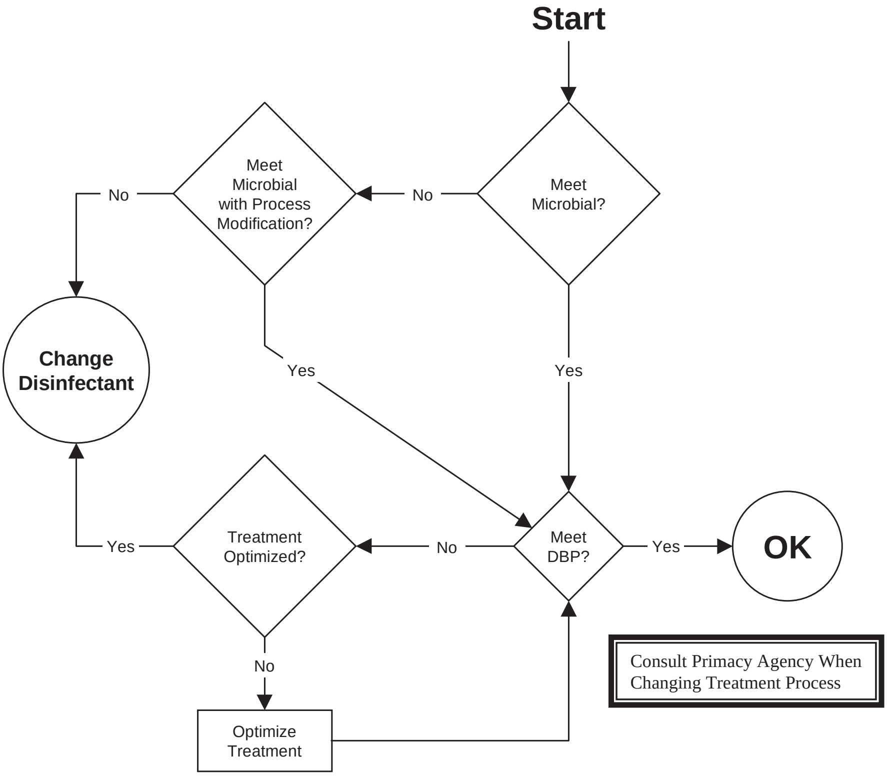


Fig. 19–18. Flow diagram 1 to evaluate current disinfection practice (*Source:* Reference 6. Reprinted from *Proceedings of the 1998 Water Quality Technology Conference,* by permission. Copyright © 1998, American Water Works Association.)

mined by existing conditions and the type of disinfectant in use. The DBP limits are established in the D/DBP Rule. To meet these limits on a consistent basis under normal varying water quality conditions, 80 percent of the MCL will serve as an action level that requires a change in treatment practice.

- **Optimizing treatment.** By optimizing existing treatment processes, the production of DBPs can be reduced. Optimization may include pretreatment optimization (coagulation, filtration, etc.) or process modifications such as moving the point of disinfection. Enhanced coagulation is required by the Stage 1 D/DBP Rule. The objective of this step is to determine if the current treatment technologies can meet the D/DBP requirements. If optimized treatment cannot meet DBP and microbial requirements, a new disinfectant is needed.

Raw Water Filtration-Surface and Groundwater Impacts One key component of the decision tree is the differentiation between surface and groundwater as raw water source. Figure 19–19 shows the second phase in the decision process, which differentiates systems that include some form of filtration (sand, membrane, natural, etc.) that provides removal of protozoa and pathogens resistant to UV and chloramines.

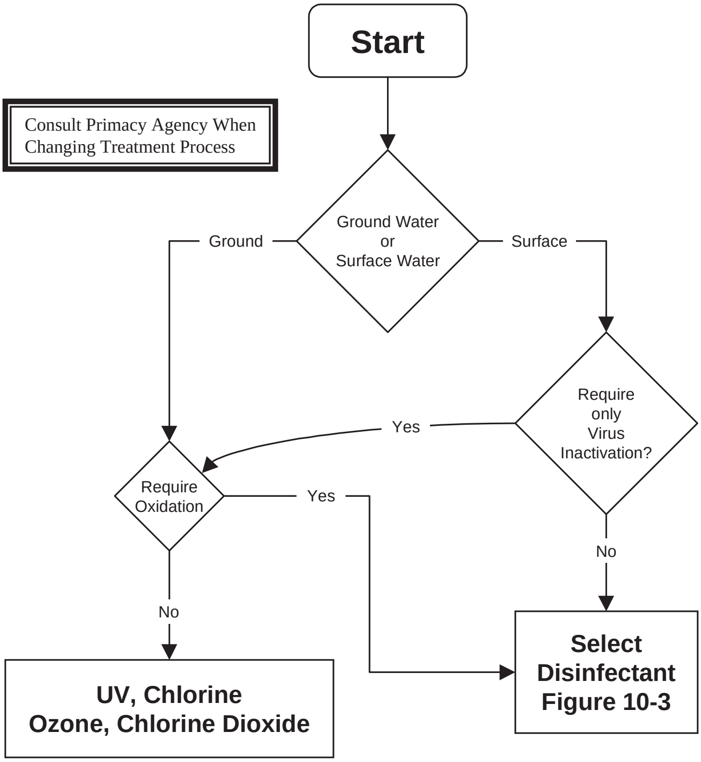


Fig. 19–19. Flow diagram 2 for surface and groundwater systems and treatment plants that include complete bacteria and protozoa removal (*Source:* Reference 6. Reprinted from *Proceedings of the 1998 Water Quality Technology Conference,* by permission. Copyright © 1998, American Water Works Association.)

In addition to groundwater sources, this distinction may also apply to waters that do not require filtration (i.e., sources that would qualify for filtration avoidance) or systems with natural filtration.

The second selection point is for disinfection systems that require only virus inactivation. This situation is created in groundwater supplies, or treatment systems that include membrane filtration, which serve as a complete barrier for bacteria and protozoa.

Selecting a New Primary Disinfectant If it is determined, using the schematic in Figure 19–18 or Figure 19–19, that a new disinfectant is required, the third phase in the decision process (Fig. 19–20) addresses the factors concerning selection of a primary disinfectant:

- **TOC Concentration.** A high TOC concentration indicates a high potential for DBP formation. In these cases, the decision tree will favor those disinfectants that will not produce DBPs or will produce the least amount of DBPs. Note that precursor removal and enhanced coagulation are used to reduce TOC during treatment optimization in Step 1 (see Fig. 19–20). "High TOC" quantifies the potential to produce DBPs and is defined as a condition meeting one of the following criteria:
  - TOC exceeds 2 mg/L
  - TTHM exceeds MCL (0.08 mg/L under Stage 1 D/DBP Rule)
  - HAA5 exceeds MCL (0.06 mg/L under Stage 1 D/DBP Rule)
- **Bromide Concentration.** The reactions of strong oxidants (ozone and peroxone) with bromide to produce hypobromous acid and bromate precludes their usage with waters containing high concentrations of bromide. High bromide is defined as concentrations exceeding 0.10 mg/L.
- **Filtered versus Nonfiltered Systems.** Two separate decision trees are presented: for those systems that include filtration and those that do not filter. This distinction addresses the degree of inactivation required. A high log inactivation is required for non-filtered surface waters, requiring a high disinfectant dose or a long contact time (*CT*). Consequently, the potential for DBP formation increases. In addition, some disinfectants, such as chlorine dioxide, have an upper-limit dose that can be applied; therefore, these disinfectants cannot be considered for systems requiring high disinfectant doses unless chlorite removal is practiced (e.g., with ferrous salt).

Note that in some instances a clear decision is not possible because the exact oxidation reactions and site-specific conditions play a significant role. For example, a decision to use ozone in the presence of bromide (which can form brominated DBPs) must be evaluated against potential DBPs formed by other disinfectants. In these cases, a bench or pilot study may be required to determine the extent and nature of DBP formation using the feasible disinfectants while meeting inactivation requirements.

Selecting a Secondary Disinfectant The selection of a secondary disinfectant depends on the selected primary disinfectant. In addition, Figure 19–21 identifies three decision points:

- **Assimilable organic carbon (AOC) concentration.** AOC is produced when a strong oxidant (e.g., ozone or peroxone) is used as the primary disinfectant in the presence of high-TOC water. High AOC is defined as concentrations exceeding 0.10

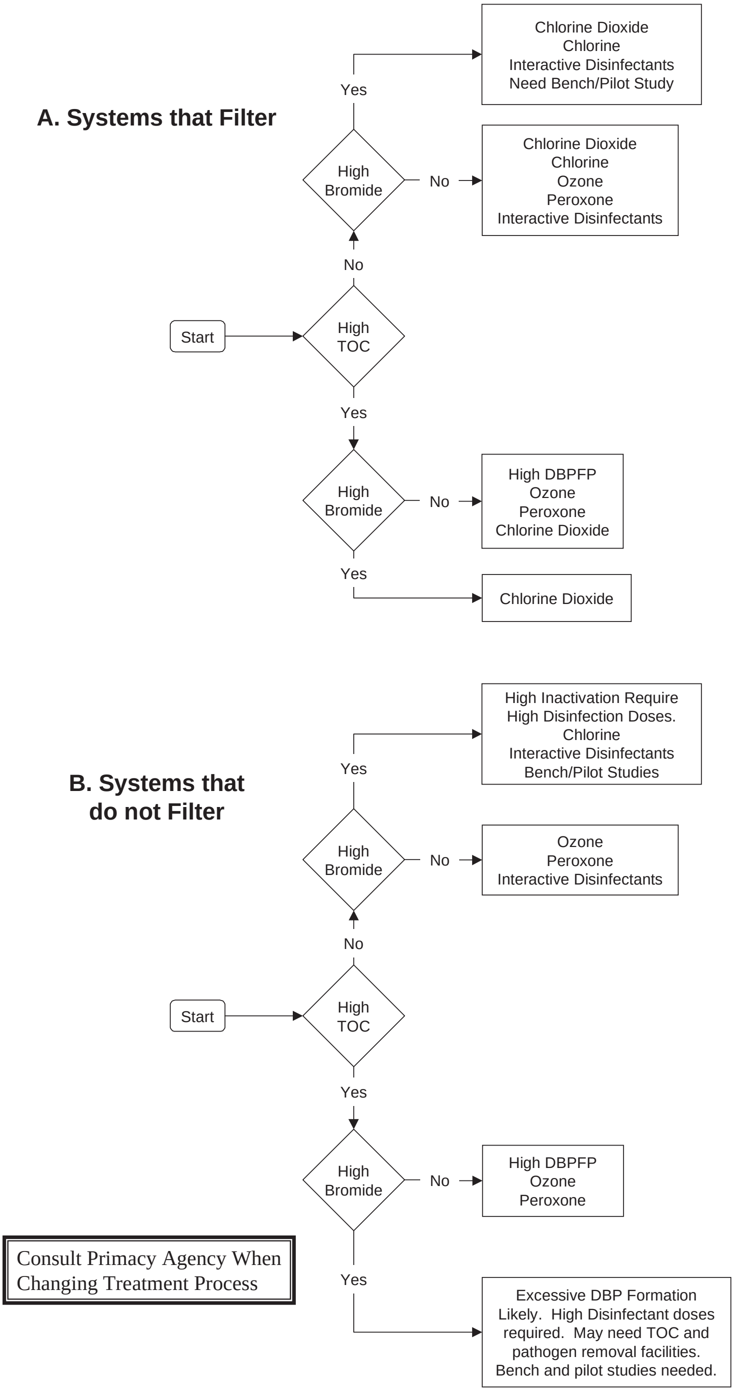


Fig. 19–20. Flow diagram 3 to select a new primary disinfectant (*Source:* Reference 6. Reprinted from *Proceedings of the 1998 Water Quality Technology Conference*, by permission. Copyright © 1998, American Water Works Association.)

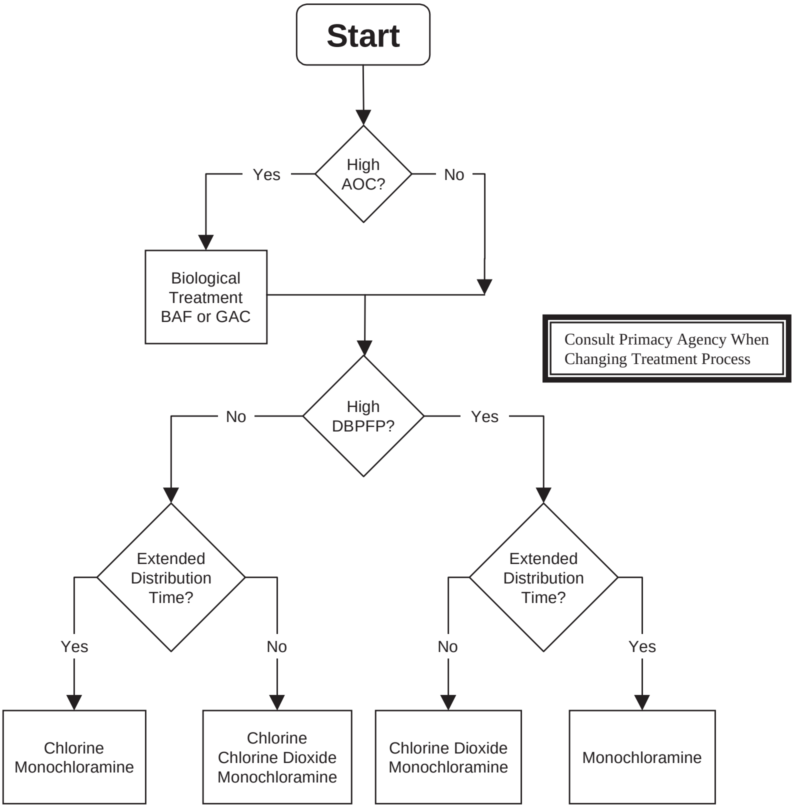


Fig. 19–21. Flow diagram 4 to select a new secondary disinfectant (*Source:* Reference 6. Reprinted from *Proceedings of the 1998 Water Quality Technology Conference,* by permission. Copyright © 1998, American Water Works Association.)

mg/L. In these cases, additional biological or GAC treatment should be considered to stabilize the finished water and prevent regrowth in the distribution system.

- **DBP Formation Potential (DBPFP).** The DBPFP serves as an indication of the amount of organic by-products that could be expected to form in the distribution system if chlorine is used. Because DBP formation continues in the distribution system, the DBP content at the plant effluent should be limited. A high DBPFP is defined as a water meeting one of the following criteria:
  - TTHM seven-day formation exceeds the MCL (0.08 mg/L under Stage 1 D/DBP Rule)
  - HAA5 seven-day formation exceeds the MCL (0.06 mg/L under the Stage 1 D/DBP Rule)
- **Distribution System Retention Time.** In a large distribution system, booster stations may be required to maintain the disinfection residual. Since chlorine dioxide has an upper limit for application, its usage may not be feasible if relatively high doses are required to maintain a residual in the distribution system. A distribution system retention time is considered high if it exceeds 48 hours.

Bench and Pilot Studies It is not possible to cover all the possible water quality scenarios that the designer can face in designing a disinfection system. In particular, some instances arise where the formation of DBPs and inactivation requirements appear to be at odds. The objective of such studies is to determine the trade-off balance between increased disinfectant dose and DBP production. In addition, bench studies are often done to evaluate alternative pretreatment scenarios (coagulation, filtration, PAC addition, etc.) to increase the DBP precursor removal.

REFERENCES

1. Montgomery, J. M., *Water Treatment Principles and Design*, John Wiley & Sons, Inc., New York, 1985.
2. USEPA, *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Works Systems Using Surface Water Sources,* Malcolm Pirnie, Inc, and HDR Engineering, Inc., for U.S. EPA Science and Technology Branch, Contract No, 68-01-6989 (1990), AWWA, 1991.
3. Connell, G. F., *The Chlorination/Chloramination Handbook*, American Water Works Association, Denver, CO, 1996.
4. White, G. C., *Handbook of Chlorination and Alternative Disinfectants,* Van Nostrand Reinhold, New York, NY, 1992.
5. AWWA (American Water Works Association), *Water Quality and Treatment* (F. W. Pontius, ed.), McGraw-Hill, New York, 1990.
6. USEPA, ''Alternative Disinfectants and Oxidants Guidance Manual,'' Report Number EPA 815-R-99-014, prepared by SAIC and HDR Engineering, Inc., April 1999.
7. AWWA and ASCE (American Water Works Association and American Society of Civil Engineers), *Water Treatment Plant Design,* McGraw-Hill, New York, 1997.
8. Palin, A., ''A Study of the Chloro Derivatives of Ammonia,'' *Water and Water Engineering* 54:248–258, 1950.
9. Werdehoff, K. S., and Singer, P. C., ''Chlorine Dioxide Effects on THMFP, TOXFP and the Formation of Inorganic By-Products,'' *JAWWA* 79(9):107, 1987.
10. Gates, D. J., *The Chlorine Dioxide Handbook; Water Disinfection Series,* AWWA, Denver, CO, 1998.
11. Singer, P. C., ''Formation and Characterization of Disinfection By-Products,'' presented at the First International Conference on the Safety of Water Disinfection: Balancing Chemical and Microbial Risks, 1992.
12. Aieta, E. M., Roberts, P. V., and Hernandez, M., ''Determination of Chlorine Dioxide, Chlorine and Chlorate in Water,'' *JAWWA* 76(1):64–70, 1984.
13. Gallagher, D. L., Hoehn, R. C., Dietrich, A. M., *Sources, Occurrence, and Control of Chlorine Dioxide By-Product Residuals in Drinking Water,* AWWARF, Denver, CO, 1994.
14. Hoigne J., and Bader, H., ''The Role of Hydroxyl Radical Reactions in Ozonation Processes in Aqueous Solutions,'' *Water Res.,* Vol. 10, pp. 377–386, 1977.
15. Staehelin, J., Buhler, R. E., and Hoigne, J., ''Ozone Decomposition in Water Studies by Pulse Radiolysis, 2 OH and HO_4 as Chain Intermediates,'' *J. Phys. Chem.*, Vol. 88, pp. 5999–6004, 1984.
16. Glaze, W. H., and Kang, J.-W., ''Advanced Oxidation Processes for Treating Groundwater Contaminated With TCE and PCE: Laboratory Studies,'' *JAWWA*, Vol. 88, No. 5, pp. 57–63, 1988.
17. Rice, R. G., *Ozone Reference Guide,* Electric Power Research Institute, St. Louis, MO, 1996.

18. Langlais, B., Reckhow, D. A., and Brink, D. R. (eds.), *Ozone in Water Treatment: Application and Engineering,* AWWARF and Lewis Publishing, Chelsea, MI, 1991.
19. DeMers, L. D., and Renner, R. C., *Alternative Disinfection Technologies for Small Drinking Water Systems,* AWWA and AWWARF, Denver, CO, 1992.
20. Wolfe, R. L., et al., ''Inactivation of *Giardia muris* and Indicator Organisms Seeded in Surface Water Supplies by Peroxone and Ozone,'' *Environmental Science and Technology* 23:6:744, 1989.
21. Ferguson, D. W., McGuire, M. J., Koch, B., Wolfe, R. L., and Aieta, E. M., ''Comparing PEROXONE and Ozone for Controlling Taste and Odor Compounds, Disinfection By-Products, and Microorganisms,'' *JAWWA* 82(4):181–191, 1990.
22. Scott, K. N., et al., ''Pilot-Plant-Scale Ozone and PEROXONE Disinfection of *Giardia muris* Seeded Into Surface Water Supplies,'' *Ozone Sci. Engrg.,* Vol. 14, No. 1, p. 71, 1992.
23. USEPA (U.S. Environmental Protection Agency), *Ultraviolet Light Disinfection Technology in Drinking Water Application: An Overview,* EPA 811-R-96-002, Office of Ground Water and Drinking Water, 1996.
24. Tchobanoglous, G. T., ''UV Disinfection: An Update,'' presented at Sacramento Municipal Utilities District Electrotechnology Seminar Series, Sacramento, CA, 1997.
25. Combs, R., and McGuire, P., ''Back to Basics: The Use of Ultraviolet Light for Microbial Control,'' *Ultrapure Water Journal* 6(4):62–68, 1989.
26. Bryant, E. A., Fulton, G. P., and Budd, G. C., *Disinfection Alternatives for Safe Drinking Water,* Van Nostrand Reinhold, New York, 1992.
27. Wolfe, R. L., ''Ultraviolet Disinfection of Potable Water,'' *Env. Sci. Tech,* Vol. 24, No. 6, pp. 763–773, 1990.
28. Von Sonntag, C., and Schuchmann, H., ''UV Disinfection of Drinking Water and By-product Formation—Some Basic Considerations,'' *J. Water SRT-Aqua,* Vol. 41, No. 2, pp. 67–74, 1992.
29. Jagger, J., *Introduction to Research in Ultraviolet Photobiology,* Prentice-Hall, Inc., Englewood Cliffs, NJ, 1967.
30. Snider, K. E., Darby, J. L., and Tchobanoglous, G., *Evaluation of Ultraviolet Disinfection for Wastewater Reuse Applications in California,* Department of Civil Engineering, University of California, Davis, 1991.
31. USEPA, *Technologies for Upgrading Existing and Designing New Drinking Water Treatment Facilities,* EPA/625/4-89/023, Office of Drinking Water, 1980.
32. USEPA, *Design Manual: Municipal Wastewater Disinfection,* EPA/625/1-86/021, Office of Research and Development, Water Engineering Research Laboratory, Center for Environmental Research Information, Cincinnati, OH, 1986.
33. Yip, R. W., and Konasewich, D. E., ''Ultraviolet Sterilization of Water—Its Potential And Limitations,'' *Water Pollut. Control* (*Canada*) 14:14–18, 1972.
34. Huff, C. B., ''Study of Ultraviolet Disinfection of Water and Factors in Treatment Efficiency,'' *Public Health Reports* 80(8):695–705, 1965.
35. Ellis, C., and Wells, A. A., *The Chemical Action of Ultraviolet Rays,* Reinhold Publishing Co., New York, 1941.
36. Murov, S. L., *Handbook of Photochemistry,* Marcel Dekker, New York, 1973.
37. Malley Jr., J. P., Shaw, J. P., and Ropp, J. R., ''Evaluations of By-Products by Treatment of Groundwaters With Ultraviolet Irradiation,'' AWWARF and AWWA, Denver, CO, 1995.
38. Worley, S. D., and Williams, D. E., ''Disinfecting Water with a Mixture of Free Chlorine and an Organic B-Halamine,'' *JAWWA,* Vol. 80, No. 1, p. 69, 1988.
39. Alleman, J. E., et al., ''Comparative Evaluation of Alternative Halogen-Based Disinfection Strategies,'' *Proceedings of the 42nd Industrial Waste Conference,* p. 519, 1988.

40. Finch, G. R., Black, E. K., and Gyurek, L. L., ‘‘Ozone and Chlorine Inactivation of *Cryptosporidium,*’’ *Proceedings 1994 Water Quality Technology Conference,* Part II, pp. 1303–1318, San Francisco, CA, November 6–10, 1994.
41. Straub, T. M., et al., ‘‘Synergistic Inactivation of *Escherichia coli* and MS-2 Coliphage by Chloramine and Cupric Chloride,’’ AWWA Water Quality Technology Conference, San Francisco, CA, November 6–10, 1994.
42. Kouame, Y., and Haas, C. N., ‘‘Inactivation of *E. coli,* by Combined Action of Free Chlorine and Monochloramine,’’ *Water Res.*, Vol. 25, No. 9, p. 1027, 1991.
43. Liyanage, L., Gyurek, L., Belosevic, M., and Finch, G., ‘‘Effect of Chlorine Dioxide Preconditioning on Inactivation of *Cryptosporidium* by Free Chlorine and Monochloramine,’’ AWWA Water Quality Technology Conference, Boston, MA, November 17–21, 1996.
44. Gyurek, L., Liyanage, L., Belosevic, M., and Finch, G., ‘‘Disinfection of *Cryptosporidium Parvum* Using Single and Sequential Application of Ozone and Chlorine Species,’’ *Proceedings AWWA Water Quality Technology Conference,* November 17–21, 1996, Boston, MA,1996.
45. Finch, G. R., ‘‘Control of *Cryptosporidium* Through Chemical Disinfection: Current State-of-the-Art,’’ AWWARF Technology Transfer Conference, Portland, Oregon, August 7–8, 1997.

CHAPTER 20

Fluoridation and Defluoridation

INTRODUCTION

The fluoride ion (reduced form of fluorine) is found in every water supply used for drinking purposes.[1] It is required for the formation of bones and teeth, and is essential to the normal growth and development of humans.

This chapter reviews the history of research on fluorides in drinking water, followed by a description of the chemicals used for fluoridation and the techniques used for fluoride addition to drinking water. Also, methods are described for the removal of fluorides in situations where their concentration is too great to allow use as a potable water supply.

FLUORIDE RESEARCH

During the 1800s, U.S. immigration officials noticed that people arriving from certain parts of Europe were severely afflicted by a disfigurement of the teeth known as mottled enamel or dental fluorosis. This led dental authorities to believe that the disfigurement was due to a local factor endemic to the immigrants' native land. Soon after, reports began to appear of mottled enamel among people native to the United States. These reports came largely from cities in the Great Plains and Rocky Mountain states.

Substantial evidence that fluorides were the cause of mottled enamel was obtained by H. V. Churchill in 1930.[2] The people of Bauxite, Arkansas, reported a high incidence of mottled enamel. Churchill, by spectrographic analysis, found appreciable amounts of fluoride ion in the Bauxite water supply. In collaboration with F. S. McKay, a dentist, he studied waters from five areas where mottling was endemic and 40 areas where it was not a problem. From these studies, it was concluded that excessive fluoride levels in the drinking water caused the mottled enamel.[3] Further proof was reported by Smith et al., who found that mottled enamel could be produced in white rats by adding to their diets either small amounts of fluoride salts or the concentrated residues from waters known to cause mottled teeth in humans.[4]

Finally, Gottlieb in 1934 reported on the relationship between fluoride concentration and mottling.[5] She found that in Kansas communities reporting mottled enamel, the concentrations of fluoride in the public drinking water supplies were in excess of 2 mg/L.

In 1938, Dean presented data demonstrating that dental caries were less prevalent when mottled enamel occurred.[6] This led to extensive correlation studies on dental caries versus fluoride levels in drinking waters throughout the United States. The results obtained, as summarized by Dean, are presented in Figure 20–1.[7] From this information, and the dental fluorosis data, a dental caries–fluoride relationship evolved:

1. When the fluoride level exceeds about 1.5 mg/L, any further increase does not significantly decrease the incidence of decayed, missing, or filled teeth, but does increase the occurrence and severity of mottling.
2. At a fluoride level of approximately 1.0 mg/L, the optimum effect occurs, that is, maximum reduction in caries with no aesthetically significant mottling.
3. At fluoride levels below 1.0 mg/L, some benefit occurs, but dental caries reduction is not great, and it gradually decreases as the fluoride levels decrease until, as zero fluoride is approached, no observable improvement occurs.

It was noted earlier that all water supplies contain fluoride. Therefore, it can be said that all water supplies are fluoridated. These "naturally fluoridated" waters with

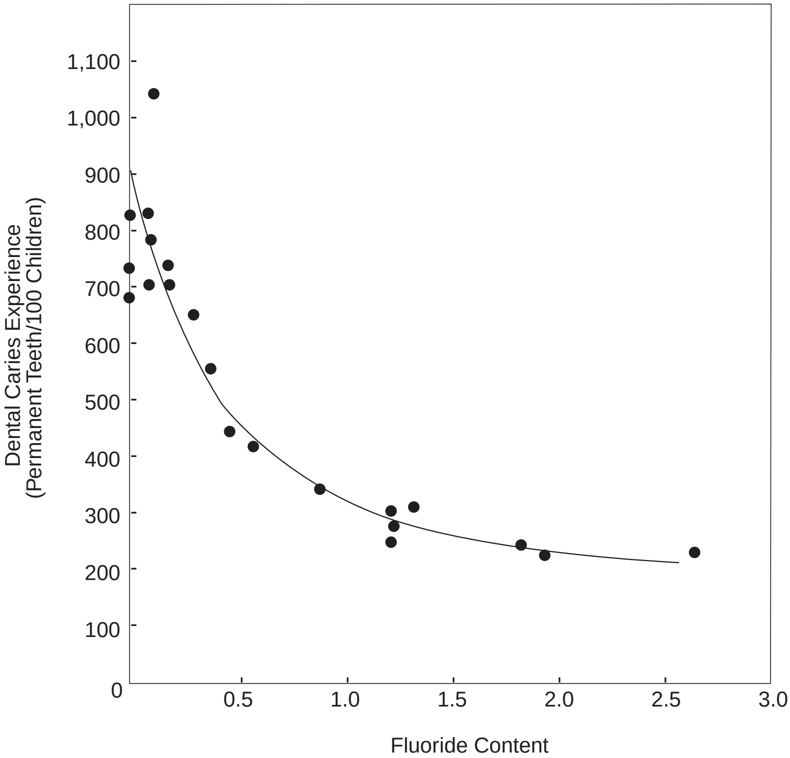


Fig. 20-1. Relationship between dental caries and fluoride level in drinking water (Reprinted from *Journal AWWA*, Vol. 35, No. 9 (September 1943), by permission. Copyright © 1943, American Water Works Association.)

fluoride concentrations of less than 0.7 mg/L do not have appreciable dental significance. For drinking water with fluoride concentrations below 0.7 mg/L, ‘‘controlled fluoridation’’ is used to increase the concentrations.

The effectiveness of controlled fluoridation has been tested in 10-year studies in such cities as Newburgh, New York, and Grand Rapids, Michigan. The investigations demonstrated the safety of controlled fluoridation and its effectiveness in controlling dental caries. The program for fluoridation of public water supplies deficient in natural fluorides has been sponsored by many organizations interested in public health, such as the American Dental and Medical Associations.[1]

Nearly 145 million Americans are currently receiving the benefits of optimally fluoridated water (62.2 percent of those 232.5 million residents on public water supplies). This total includes about 135 million people on community water supplies where fluoride levels are adjusted to optimum levels and about 10 million people whose water supplies have naturally occurring fluoride levels in the optimum range. This also represents over 14,300 water systems that serve more than 10,500 American communities. Four of the five largest U.S. cities are fluoridated; and, of the 50 largest U.S. cities, 43 are fluoridated. Throughout the world, including the United States, fluoridated water benefits over 300 million people in more than 40 countries.

With the 1995 enactment of Assembly Bill 733 in California, 10 states and territories in the United States now mandate fluoridation through legislation. Besides California, these include seven other states (Connecticut, Georgia, Illinois, Minnesota, Nebraska, Ohio, and South Dakota), as well as the District of Columbia and Puerto Rico. Additionally, the Commonwealth of Kentucky mandates statewide community water fluoridation by administrative regulation. Three states (South Dakota, Rhode Island, and Kentucky), as well as the District of Columbia, have achieved the success of 100 percent of their treated community water systems providing the benefits of fluoridation to their citizens.[8]

In spite of our long-term experience with fluoridation, fluoride continues to remain the most controversial constituent added to drinking water. Proponents of fluoridation like the American Dental Association cite studies documenting its efficacy. Opponents of fluoride offer differing positions, maintaining:

- Fluoride is not effective in reducing tooth decay.
- Public application of fluoride limits individual freedom of choice in selecting means of dental treatments.
- Fluoride causes adverse long-term health impacts (possibly including various cancers).
- Since most of the fluoridated drinking water supplied in public systems is not consumed, but passes through wastewater treatment systems, the fluoride poses a potential threat to the environment.
- When fluoride addition is legislated by state regulations, it becomes an ‘‘unfunded mandate.’’

While it must be stressed that there is no reliable evidence of adverse health effects associated with appropriately controlled fluoride addition, clearly, the initiation of fluoridation in public water supplies is not an action to be undertaken lightly. Due to the controversy surrounding fluoridation, a full public hearing carried through the political process is necessary to support the decision.

FLUORIDATION STANDARDS AND RECOMMENDATIONS

From the relationship between dental caries and fluoride concentration, the recommended fluoride concentrations shown in Table 20–1 were determined.[9] The recommended fluoride concentrations decrease with increasing temperature because of the hypothesis that more water is consumed during warm-weather periods than at cooler temperatures.

Maximum fluoride levels in drinking water are regulated under Section 1412 of the Safe Drinking Water Act (SDWA). When regulating a contaminant under this Act, EPA promulgates both a Maximum Contaminant Level Goal (MCLG, a nonenforceable health goal), and a Maximum Contaminant Level (MCL, which is a federally enforceable standard). The MCL is set as close to the MCLG as technically feasible, taking costs and other factors into consideration. When establishing an MCLG, the Act requires EPA to protect against adverse health effects with a margin of safety. The SDWA leaves the question of what constitutes an adverse health effect to the EPA.

EPA may also promulgate nonenforceable secondary standards that are designed to protect the public welfare. Secondary standards are usually based on aesthetic considerations such as taste or odor.

In 1986, EPA promulgated both the fluoride MCLG and MCL at 4 mg/L. This level protects humans from crippling skeletal fluorosis, an adverse health effect.

At the same time, EPA also set a nonenforceable Secondary Maximum Contaminant Level of 2 mg/L for fluoride in drinking water to protect against objectionable dental fluorosis (i.e., a staining and/or pitting of the teeth). While community water systems are not required to reduce the level of fluoride if it exceeds 2 mg/L, they are required to distribute a public notice which advises that children are likely to develop objectionable dental fluorosis. In developing the MCL and the MCLG at 4 mg/L (vs. 2 mg/L to protect against dental fluorosis), EPA concluded that dental fluorosis is a cosmetic effect and not an adverse health effect.[10]

FLUORIDATION

Fluorine is the most chemically active element known. Like chlorine, it is always found in the reduced state (F^-) in combination with other elements. In water solution, the

TABLE 20–1. Recommended Limits for Fluoride in Drinking Water

Annual Average of Maximum Daily Air Temperature, °F (°C) Based on Temperature Data Obtained for a Minimum of 5 Years	Recommended Limits, mg/L		
	Lower	Optimum	Upper
50–54 (10–12)	0.9	1.2	1.7
54–58 (12–15)	0.8	1.1	1.5
58–64 (15–18)	0.8	1.0	1.3
64–71 (18–21)	0.7	0.9	1.2
71–79 (21–26)	0.7	0.8	1.0
79–91 (26–33)	0.6	0.7	0.8

Source: Reference 9.

compounds dissociate into ions, the form in which fluorine is assimilated by humans. Theoretically, any compound that gives fluoride ions in water solution can be used for increasing the fluoride content of a water supply. However, there are several practical considerations involved in selecting the fluoride source. First, it must be sufficiently soluble to be used in routine water practice. Second, the compound must not have any undesirable characteristics. Third, the material used should be readily available and relatively inexpensive.

The three most commonly used fluoride compounds in water treatment are sodium fluoride (NaF), sodium silicofluoride (Na_2SiF_6), and fluosilicic acid (H_2SiF_6). Other compounds used for fluoridation include calcium fluoride (CaF_2), ammonia silicofluoride ($(NH_4)_2SiF_6$), and hydrofluoric acid (H_2F_2). The most commonly used compounds are described in detail.

Sodium Fluoride

Sodium fluoride (NaF) was the first fluoride compound used for fluoridation. Although it is one of the most expensive fluoridation compounds for the amount of available fluoride, it is still the most widely used.

NaF is a white, odorless material available either as a powder or in crystalline form. Table 20–2 lists the characteristics of NaF and the other two popular fluoridation compounds. The maximum solubility of NaF is 4.0 percent, resulting in a fluoride concentration of 18,000 mg/L. Its solubility is practically constant over the temperature range generally encountered in water treatment. Solution pH varies with the type of amount of impurities, but solutions prepared from the usual grades of NaF exhibit near-neutral pH.

Powdered NaF is produced in densities ranging from 65 to 90 pounds per cubic foot (1,054 to 1,458 kg/m^3). Crystalline NaF is produced in various size ranges, usu-

TABLE 20–2. Characteristics of Fluoride Compounds

Item	Sodium Fluoride, NaF	Sodium Silicofluoride, Na_2SiF_6	Fluosilicic Acid, H_2SiF_6
Form	Powder or crystal	Powder or very fine crystal	Liquid
Molecular weight	42	188	144
Commercial purity, %	97–99	98–99	22–30
Fluoride ion % (100% pure material)	45.3	60.7	79.2
lb (kg) required per MG (ML) for 1.0 mg/L F at indicated purity	18.8 (2.25) (98%)	14.0 (1.68) (98.5%)	46 (5.51) (23%)
pH of saturated solution	7.6	3.5	1.2 (1% solution)
Solubility, g per 100 g water, at 25°C	4.05	0.762	Infinite

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ally designated as coarse, fine, and extra-fine. The crystalline form is preferred when manual handling is involved because the absence of fine powder results in a minimum of dust.

Sodium Silicofluoride

Sodium silicofluoride (Na_2SiF_6) is the cheapest of the compounds currently in use. Its cost makes it very popular for fluoridation.

Na_2SiF_6 is a white, odorless, crystalline powder. Its solubility varies from 0.44 percent at 32°F (0°C) to 2.45 percent at 212°F (100°C). Saturated solutions exhibit an acid pH, usually between 3.0 and 4.0. The density of Na_2SiF_6 ranges from 55 to 72 lb/cu ft (900 to 1,200 kg/m^3). Experience has shown that for best feeding results with mechanical feeders, the Na_2SiF_6 should have a low moisture content plus a relatively narrow size distribution.

Fluosilicic Acid

This fluoridation compound is a 20 to 35 percent solution of H_2SiF_6 in water. When pure it is a colorless, corrosive liquid with a pungent odor and can cause skin irritation. Upon vaporizing, the acid decomposes to form hydrofluoric acid and silicon tetrafluoride. All solutions of fluosilicic acid are characterized by a low pH. A concentration sufficient to produce 1 mg/L of fluoride ion can cause a significant pH depression in poorly buffered waters. For example, in water at pH 6.5 and containing 30 mg/L of total dissolved solids (TDS), the addition of H_2SiF_6 to produce 1 mg/L of fluoride ion caused the pH to drop to 6.2.[1]

Fluosilicic acid is a solution containing a high proportion of water. Consequently, large quantities can be expensive to ship. Economics generally restricts the use of fluosilicic acid to smaller waterworks.

Other Fluoride Compounds

Calcium fluoride (CaF_2), ammonium silicofluoride ($(NH_4)_2SiF_6$), and hydrofluoric acid (H_2F_2) have been used for water fluoridation. Each has particular properties that make the compound desirable in a specific application; however, none has widespread use.

Fluorspar (CaF_2) is the cheapest of the compounds used for fluoridation, but it has the disadvantage of being the least soluble. It has been successfully fed by first dissolving it in an alum solution, and then utilizing the resultant liquid to supply both the alum needed for coagulation and the fluoride for fluoridation. Some attempts have been made to feed fluorspar directly in the form of ultra-fine powder, on the premise that the powder would eventually dissolve or would remain in suspension until consumed.

Ammonia silicofluoride ($(NH_4)_2SiF_6$) has the advantage of supplying the ammonium ion necessary for the production of chloramines when this form of disinfectant is preferred to chlorine. Otherwise, it has found little use in fluoridation.

Hydrofluoric acid (H_2F_2) has been used in a few specially designed installations. It is low in cost, but it presents a safety and corrosion hazard and is generally not used for fluoridation.

FLUORIDATION SYSTEMS

No one type of fluoridation system is applicable to all water treatment plants. Selection is based on size and type of water facility, chemical availability, cost, and operating personnel available. For small utilities, usually some type of solution feed is selected, and batches are manually prepared. A simple system consists of a solution tank and a solution metering pump with appropriate piping from the tank to the water main for application (Fig. 20–2). If fluosilicic acid is used, it is either used at full strength (Fig. 20–3) or diluted with water in the feed tank (Fig. 20–4). When sodium fluoride is used, the feed solution may be prepared to a desired strength or as a saturated solution. Because NaF has a maximum solubility of 4.0 percent, a saturated solution is prepared by passing water through a bed containing an excess of NaF (Fig. 20–5). While NaF is quite soluble, calcium and magnesium fluorides form precipitates that can scale and clog feeders and lines. Consequently, dissolution water should be softened whenever the hardness exceeds 75 mg/L.

Large waterworks usually use either gravimetric dry feeders to apply sodium silicofluoride (Fig. 24–13) or solution feeders to apply fluosilicic acid. Often their systems incorporate automatic control systems to regulate flow and adjust feed rates. Fluoride feed must be paced to water flow to maintain a consistent fluoride ion concentration.

Fluoride must be injected into all of the water entering the distribution system. If there is more than one supply point, separate fluoride feeding installations are required for each water facility. In a well system, application can be in the discharge line of each pump or in a common line leading to a storage reservoir. Fluoride can be applied in a treatment plant in a channel or line from the filters, or directly into the clearwell.

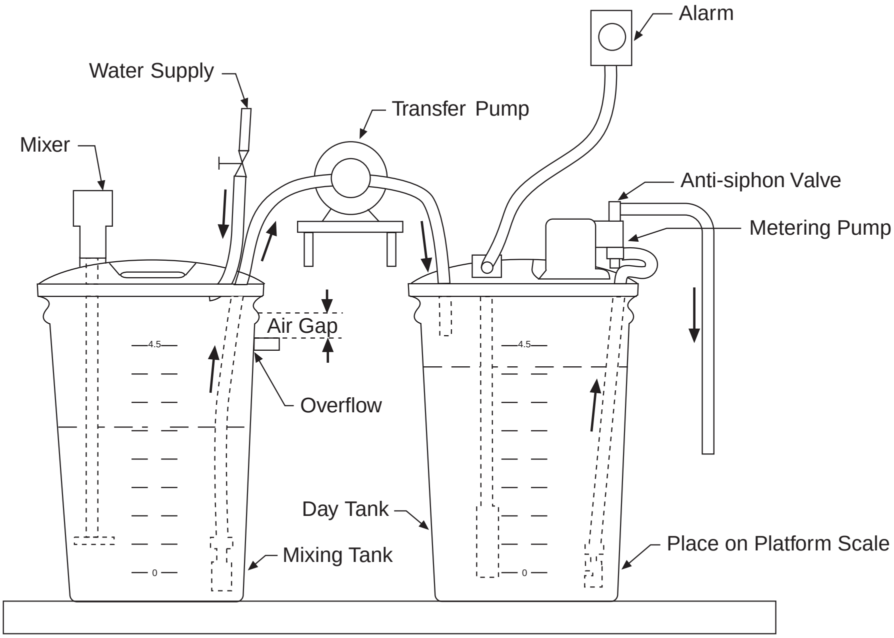

Fig. 20–2. Solution feed installation

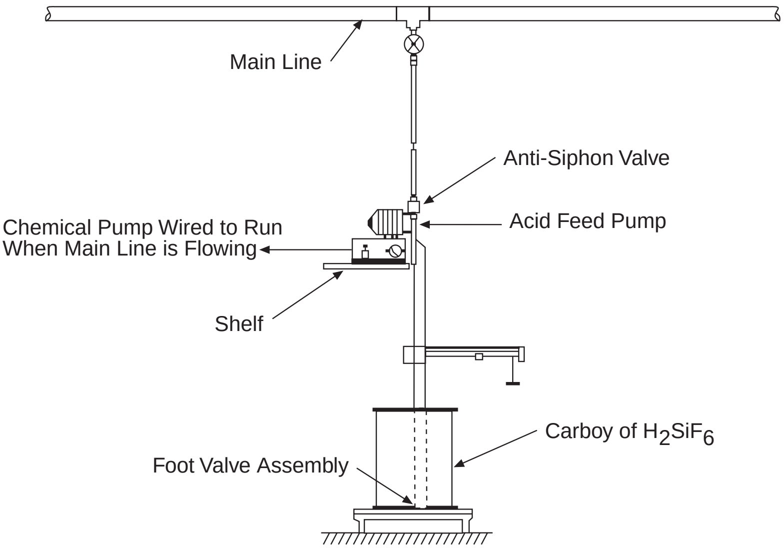

Fig. 20-3. Acid feed installation

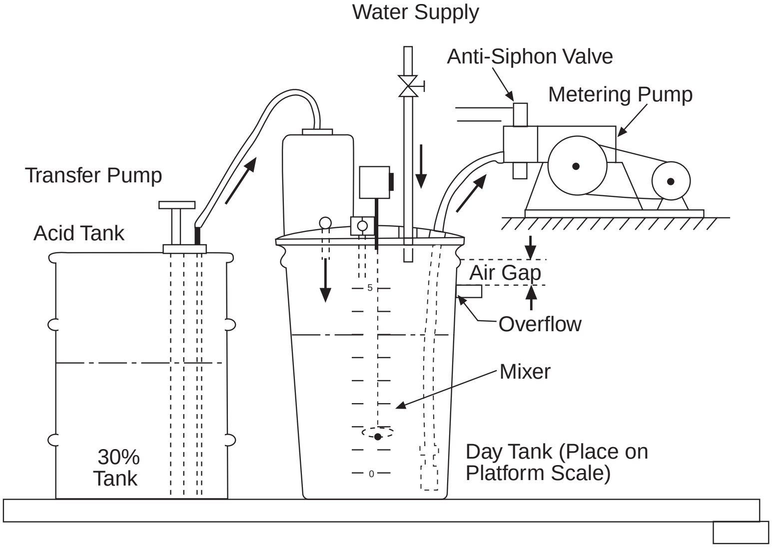

Fig. 20-4. Diluted acid feed system

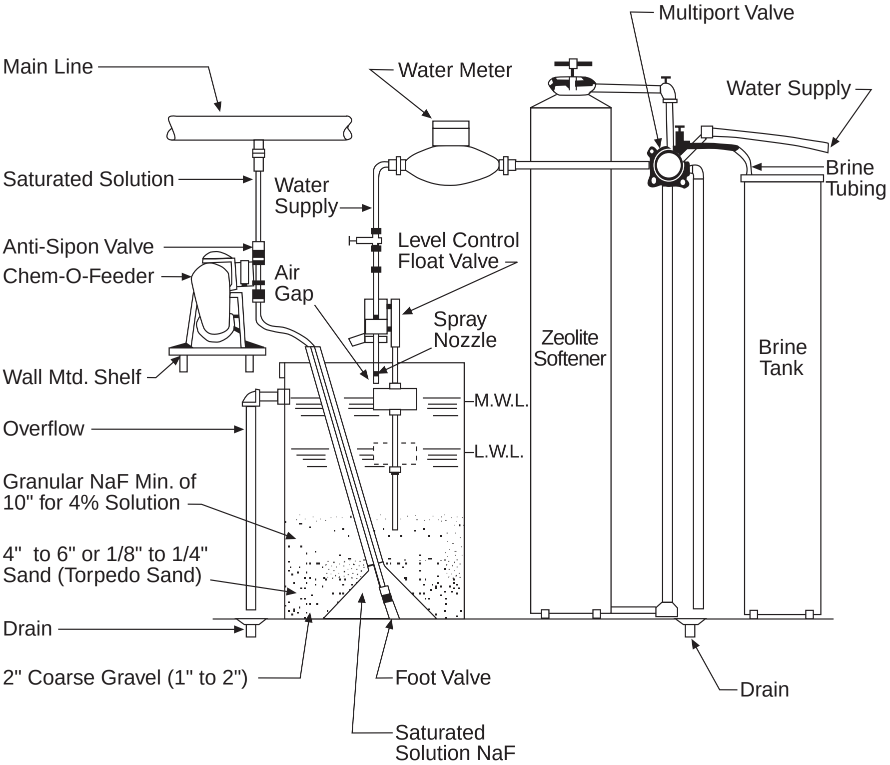

Fig. 20–5. Downflow saturator

Whenever possible, it should be added after filtration to avoid possible losses due to reactions with other chemicals. Of particular concern are coagulation with alum and lime-soda softening. Fluoride injection points should be as far away as possible from the addition of chemicals that contain calcium because of the insolubility of CaF_2.

Surveillance of water fluoridation involves testing both the raw and treated water for fluoride ion concentration. Records of the weight of chemical applied and the volume of water treated should be kept to confirm that the correct amount of fluoride is being added. The fluoride concentration in the treated water should be that recommended by drinking water standards (Table 20–1).

DEFLUORIDATION

As soon as excessive amounts of fluorides in drinking water supplies had been established as the cause of dental fluorosis, methods for their removal were studied. However, comparatively little research has been performed on theoretical design models for the removal of fluorides. Consequently, the design of defluoridation processes is largely based on past experience and empirical models.

Fluoride removal is complicated by the presence of other ions in the water that compete with fluoride for removal. The design of defluoridation systems requires laboratory and pilot-scale work prior to the design of full-scale treatment systems.

Two methods of defluoridation have found practical application. One involves passage of water through defluoridation media such as bone meal, bone char, ion-exchange resins, or activated alumina. The second method involves the addition of chemicals such as lime or alum prior to rapid mixing, flocculation, and sedimentation in a waterworks, for the removal of fluoride only or the concurrent removal of fluoride and other ions (e.g., calcium and magnesium removal for water softening).

The following discussion presents past experience with media filter and chemical addition defluoridation systems.

Defluoridation Media

The uptake of fluoride onto the surface of bone was first reported by Smith and Smith in 1937.[11] They suggested that fluoride was removed by ion exchange in which the carbonate radical of the apatite comprising bone [i.e., $Ca(PO_4)_6 \cdot CaCO_3$] was replaced by fluoride to form an insoluble fluorapatite, according to:

$$Ca(PO_4)_6 \cdot CaCO_3 + 2F^- = Ca(PO_4)_6 \cdot CaF_2\downarrow + CO_3^{-2} \qquad (20\text{–}1)$$

Similarly, bone char or tricalcium phosphate ($Ca_3(PO_4)_2$), produced by carbonizing bone at temperatures of 2012 to 2912°F (1100 to 1600°C), has been used for defluoridation. When exhausted, the column is regenerated by application of a 1.0 percent solution of caustic soda, which converts the fluorapatite to hydroxyapatite ($Ca(PO_4)_2 \cdot Ca(OH)_2$). The fluoride is removed as soluble sodium fluoride. The caustic is followed by a rinse and then an acid wash to lower the pH.[12] In the regenerated form, the hydroxyl radical becomes the exchange anion in the defluoridation reaction.[13] While bone char has been used successfully for full-scale defluoridation, over the past several decades it has been supported by the availability of more cost-effective sorption media.[14]

Paired cationic and anionic exchange resin beds have also been used for defluoridation. In this process, illustrated by Fig. 20–6, water first passes through a cationic resin (R^+) bed, which exchanges sodium with hydrogen to form the equivalent acid:

$$2NaF + H_2R^+ = H_2F_2 + Na_2R^+ \qquad (20\text{–}2)$$

The hydrogen fluoride is then removed during passage through the anionic bed (R^-):

$$2R^- + H_2F_2 = 2R^-HF \qquad (20\text{–}3)$$

Periodically, the resins are regenerated with acid and alkaline solutions. Synthetic ion exchange resins for defluoridation are available from several manufacturers.

The most widely used defluoridation method involves beds of granular activated alumina (Al_2O_3). Activated alumina is useful for controlling a number of drinking water constituents.[15] A discussion of the sorptive properties of activated alumina and the design criteria for sorption columns is provided in Chapter 18—“Ion Exchange and Activated Alumina Sorption.”

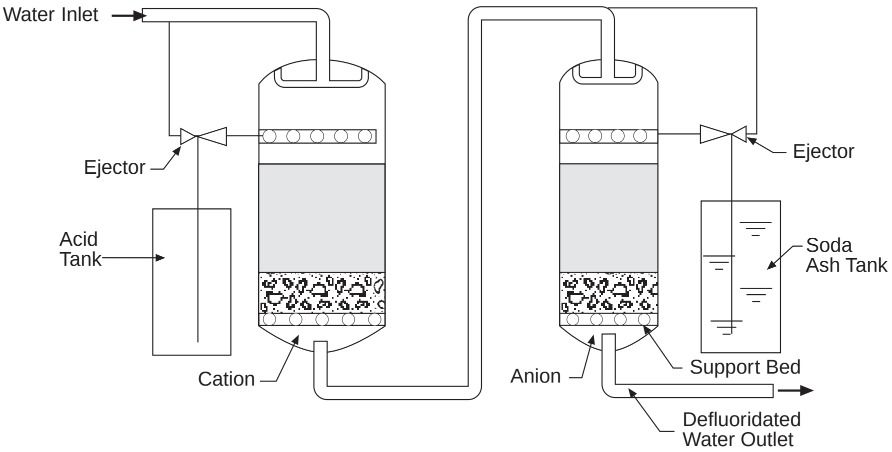

Fig. 20-6. Paired cationic and anionic exchange resin beds (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems*. Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Chemical Addition for Defluoridation

Lime and alum have been used successfully for fluoride removal. The defluoridation system generally consists of lime or alum addition to a rapid mix chamber, followed by flocculation and sedimentation.

Alum was one of the first chemicals investigated for use in removing fluoride from drinking water supplies.[12] When added to water, alum reacts with the alkalinity in the water to produce insoluble aluminum hydroxide, according to the following equation:

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Ca(HCO_3)_2 = 2Al(OH)_3 + 3CaSO_4 + 14H_2O + 6CO_2 \qquad (20\text{–}4)$$

Rabosky and Miller suggest that fluoride is removed by adsorption onto the $Al(OH)_3$ particles.[16] Figure 20–7 presents data for Scott et al. and Culp and Stoltenberg on fluoride removal at various alum dosages.[17,18] The latter reported on the lowering of the fluoride concentration of a soft, highly mineralized water from 3.6 to 1.0 mg/L by the addition of 315 mg/L alum followed by 30 minutes of flocculation.

Finally, fluorides have been observed to be removed during lime softening of drinking water. Fluoride precipitation occurs according to the following reaction:

$$Ca(OH)_2 + 2HF = CaF_2\downarrow + 2H_2O \qquad (20\text{–}5)$$

Theoretical solubility calculations suggest that an effluent fluoride concentration of approximately 7.8 mg/L can be achieved by lime precipitation.[12] However, much lower fluoride concentrations have been observed. Scott et al. reported the defluoridation during lime softening could have been described by the following equation:[17]

$$F_r = 0.07 \cdot F_i \cdot (Mg)^{0.5} \qquad (20\text{–}6)$$

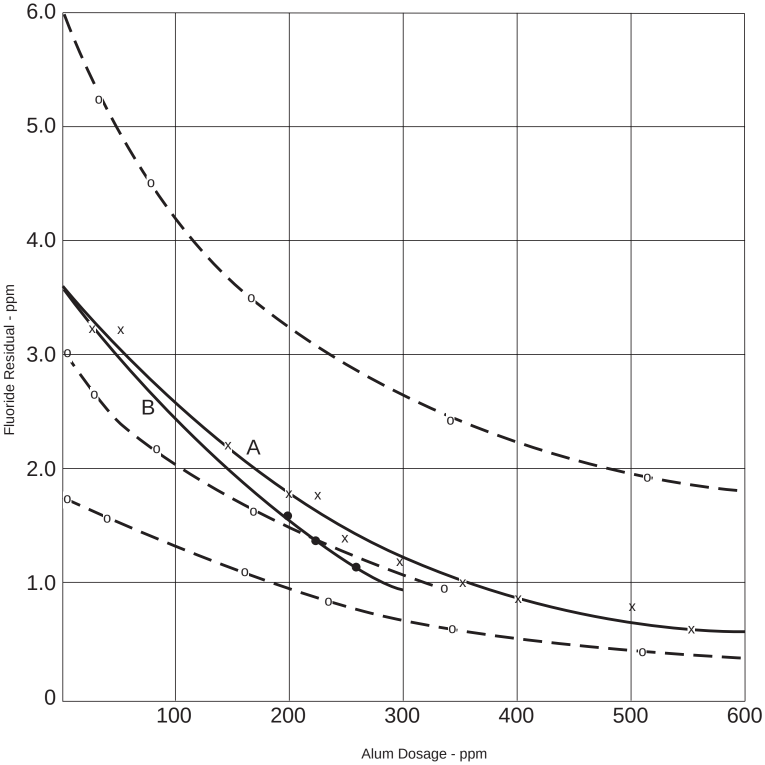

Fig. 20-7. Fluoride removal by alum coagulation (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

where:

F_r = residual fluoride
F_i = initial fluoride
Mg = magnesium concentration removed

Based on this formula, reduction of fluoride from 3.3 mg/L to 1.0 mg/L requires that 100 mg/L of magnesium be removed. Consequently, lime addition for defluoridation is appropriate only in high-magnesium waters.

REFERENCES

1. Bellack, E., *Fluoridation Engineering Manual,* Office of Water Programs, Water Supply Programs Division, EPA, 1972.
2. Churchill, H. V., ‘‘Occurrence of Fluoride in Some Waters of the United States,’’ *Ind. Eng. Chem.* 23:996–998, 1931.

3. McKay, F. S., ‘‘The Present Status of the Investigation of the Cause and the Geographical Distribution of Mottled Enamel, Including a Complete Bibliography on Mottled Enamel,’’ *J. Dental Res.* 10:561–568, 1930.
4. Smith, M. C., Lantz, E. M., and Smith, H. V., ‘‘The Cause of Mottled Enamel, and Defect of Human Teeth,’’ *University of Arizona Tech. Bulletin* No. 32, 253–282, 1931.
5. Gottlieb, S., ‘‘Fluorides in Kansas Waters and Their Relation to Mottled Enamel,’’ *Trans. Kansas Academy of Science* 37:129–131, 1934.
6. Dean, H. T., *U.S. Public Health Reports* 53:1443, 1938.
7. Dean, H. T., *JAWWA* 35:1161, 1943.
8. ‘‘Fluoridation Facts,’’ American Dental Association, Chicago, The Association, p. 30, 1993.
9. ‘‘Drinking Water Standards and Guidelines,’’ EPA, Water Division, 1974.
10. ‘‘Fluoride in Drinking Water,’’ USEPA, EPA 822-F-03-01, December 1993.
11. Smith, H. R., and Smith, L. C., ‘‘Bone Contact Removes Fluoride,’’ *Water Works Engineering* 90:5, 600, 1937.
12. Benefield, L. D., Judkins, J. F., Jr., and Weand, B. L., *Process Chemistry for Water and Wastewater Treatment*, Prentice-Hall, Englewood Cliffs, NJ, 1982.
13. Maier, F. J., ‘‘Defluoridation of Municipal Water Supplies,’’ *JAWWA* 45:8, 879, 1953.
14. Harmon, J. A., and Kalichman, S. G., ‘‘Defluoridation of Drinking Water in Southern California,’’ *JAWWA* 57:2, 245, 1965.
15. Savinelli, E. A., and Black, A. P., ‘‘Defluoridation of Water with Activated Alumina,’’ *JAWWA* 50:1, 33, 1958.
16. Rabosky, J. G., and Miller, J. P., ‘‘Fluoride Removal by Lime Precipitation and Alum and Polyelectrolyte Coagulation,’’ *Proceedings*, 29th Purdue Industrial Waste Conference, pp. 669–676, 1974.
17. Scott, R. D., et al., ‘‘Fluoride in Ohio Water Supplies—Its Effect, Occurrence, and Reduction,’’ *JAWWA* Vol. 29, 1937.
18. Culp, R. L., and Stoltenberg, H. A., ‘‘Fluoride Reduction at La Crosse, Kan.,’’ *JAWWA* 50: 423–431, 1958.

CHAPTER 21

Distribution System Corrosion: Monitoring and Control

INTRODUCTION

Corrosion is an electrochemical interaction between a metal and its environment that results in changes in the property of the metal. For drinking water utilities, internal corrosion in a water distribution system leads to at least two major problems. The first and most obvious is the failure of the distribution system pipes. Corrosion may cause both leakage and occlusion of the pipes. National estimates of the average treated water volume lost to leakage run as high as 10 percent of the distributed total; that's over a billion gallons of water per day. Increased pumping costs due to scaling, the buildup of corrosion products in the distribution mains, and lost hydraulic capacity exceed $200 million annually.[1] The second problem is an unwanted change in water quality as the water is carried through the distribution system. Examples of this problem include increases in lead and copper levels, which are released from pipes and solder and are important because of their toxic properties. The release of corrosion products has implications for both public health and consumer satisfaction.

The costs associated with the second major type of corrosion problem are often subtle and difficult to quantify—specifically, unwanted changes in the chemical and aesthetic quality of the distributed water resulting from the release of corrosion products. Economics and aesthetics aside, the overriding corrosion concern today relates to public health, with an emphasis on the perceived threat generated by the release of metals (particularly lead) from the dissolution of metallic surfaces. The association between chronic low-level lead exposure and adverse health impacts is well documented for a variety of public health concerns. Of these, the production of neuralgic deficits in the pediatric population has the gravest long-term societal implications, and because the emerging body of evidence describing the association between these impacts and ambient lead levels indicates a stronger causality than previously believed, the U.S. Environmental Protection Agency (as well as the regulatory bodies of other nations) has enacted new regulations substantially reducing permissible drinking water lead levels.[2–5]

In the United States, the centerpiece of this new regulation has been the 1991 Lead and Copper Rule. It has significantly broadened water utility responsibility and liability, extending both beyond the distribution system into the homeowner's residence. Corrosion criteria must now be monitored and regulated throughout the distribution system, up to and including the point of delivery at the consumer's tap. The act requires that public water systems demonstrate ‘‘optimal’’ corrosion control, a nebulous term which in practice means that most large water utilities will implement additional treatment strategies to reduce corrosion-induced metal release from plumbing and distribution surfaces.[6] The lead concern spotlights corrosion of lead pipe, solders, copper tubing, brasses, and other lead-bearing alloys. This emphasis is moving corrosion control to a higher technological level and giving it a new focus directed at process control and systems optimization.

This chapter begins with a discussion of the factors that influence corrosion reactions, then goes on to develop the techniques by which corrosion can be both mitigated and monitored in the distribution system. Portions of this chapter are excerpted (and reprinted with permission) from the American Water Works Association’s *Research Foundation Report on Internal Corrosion of Water Distribution Systems* (2nd ed.).

CORROSION BASICS

Corrosion occurs because the metals that make up the plumbing materials of the distribution system are not at equilibrium with the water they contact. In almost all cases, the elemental metal of the plumbing systems, whether it be iron, copper, lead, or zinc, exists in a higher oxidation state then it would were it dissolved in the water. Hence, there is a driving force for the metal to be oxidized. This driving force represents itself as a difference in electrical potential that can actually be measured at the interface between the metal and the water. This electrified interface is characteristic of corrosion reactions and actually provides us a tool to measure corrosion rates, as we will see later in this chapter. The magnitude of the electrical potential reflects the extent of the disequilibration.

When corrosion takes place the metal is oxidized; individual metal atoms give up electrons, assume a lower oxidation state, and are ejected from the crystalline latticework that was the pure metal surface. This takes place at a region on the plumbing surface called the anode. Coupled to and simultaneous with the oxidation is a corresponding reduction reaction, wherein some species, usually dissolved in the solution contacting the metal, takes up the electrons. This reaction occurs on the portion of the plumbing surface called the cathode. The anode and the cathode are coupled both by an internal circuit (pipe wall) that allows the electrons to flow from the site of oxidation (anode) to the site of reduction (cathode), as well as by an external circuit (electrolyte solution) that enables the dissolved ionic species generated at the respective anodic and cathodic sites to migrate between sites so as to achieve electroneutrality of the solution (see Fig. 21–1).

Because oxidation always occurs at the anode, a surplus of electrons depress the electrical potential in this area. Reduction at the cathode serves as a sink for electrons; hence, the electron flow is from the low-potential anode to the higher-potential cathode. The difference in electrical potential between anodic and cathodic regions can be easily measured on most corroding surfaces.

Uniform Versus Local Corrosion

The nature of the anode and cathode interactions are important to the serviceability of the metal. Depending on these interactions, the corrosion processes can take several different morphologic forms, the different forms having varying consequences for the

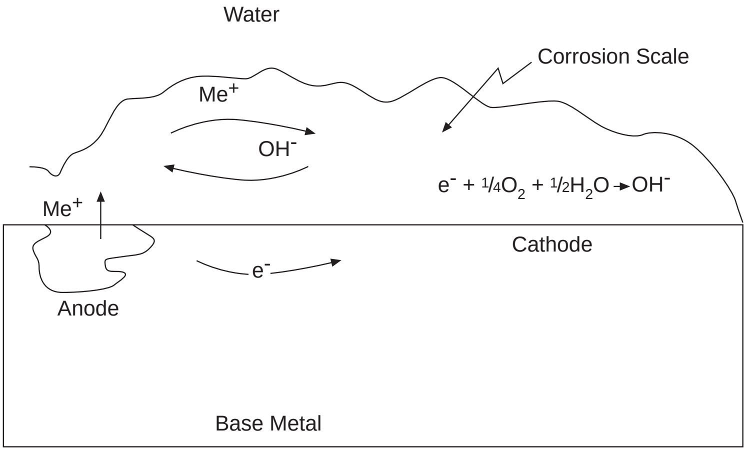

Fig. 21–1. Schematic of a corrosion cell

service life and utility of the metal. In generic terms, we speak of corrosion morphology as being either uniform or localized.

Localized Corrosion In localized corrosion the anodic processes are fixed in one location on the plumbing surface. Metal oxidation occurs only in that area, with the surrounding area serving as the site of the cathodic reactions. In copper plumbing systems, this type of corrosion is often called pitting corrosion and it can lead to rapid penetration of the pipe wall and leakage. In some circumstances, penetration is complete within three months of the inception of the pitting process (see Figs. 21–2 and 21–3). On iron and steel systems, penetration of the pipe wall is of concern, but an even greater problem is the tuberculation that accompanies the localized corrosion processes. The tuberculation is the result of the accumulation of corrosion products in the immediate vicinity of the localized corrosion. The tubercles generally have a distinctive structure, can be quite large, and ultimately occlude the pipe dramatically, reducing flows (see Fig. 21–4). The intense localized corrosion produces heavy metal loss. Oxidized metal in different mineral forms often accumulates immediately above the pit site, creating a tubercle.

Pipe wall penetration can be rapid, often requiring less than six months. The protected and largely enclosed area of the pit creates a chemical environment that promotes the pitting process; hence passivation processes that would retard corrosion are not effective, and the pit becomes self-propagating. The chemistry of the pit also leads to the formation of unusual mineral deposits, as seen in the micrograph. The tuberculation on this pipe section has developed over many years. Due to occlusion and increased surface roughness, the hydraulic capacity of the line has been reduced by over half.

Uniform Corrosion As the name implies, on some metals the corrosion process appears to be uniformly distributed over the entire surface of the metal so that it is not possible to distinguish distinctive anodic or cathodic sites. When uniform corrosion occurs, it is because the metal is acting as a ‘‘polyelectrode,’’ meaning that any one

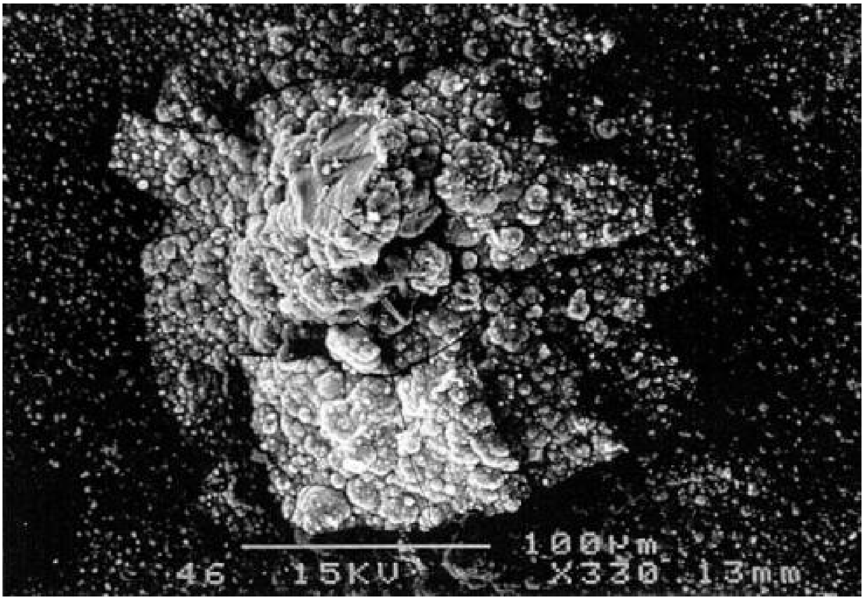

Fig. 21–2. Scanning electron micrograph of incipient-stage pitting on residential copper tubing

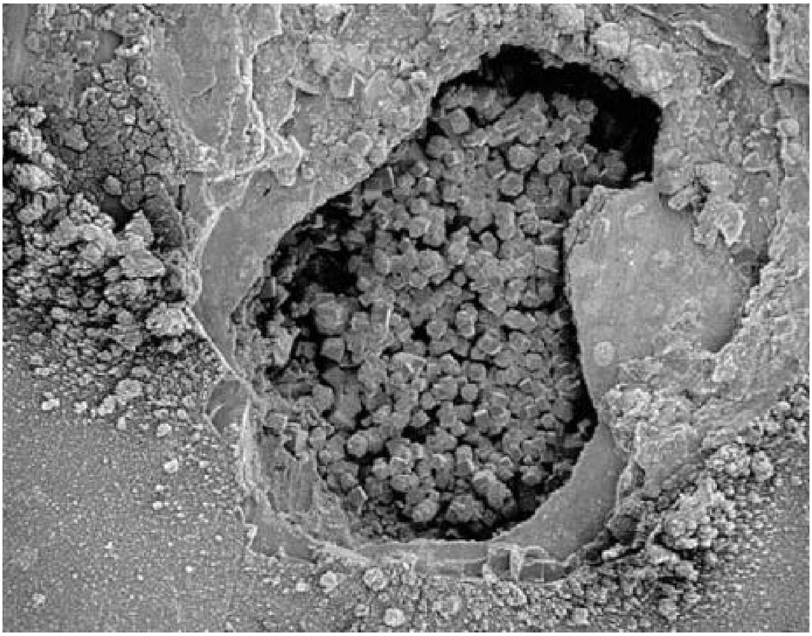

Fig. 21–3. Scanning electron micrograph of copper pit with overlying tubercle removed

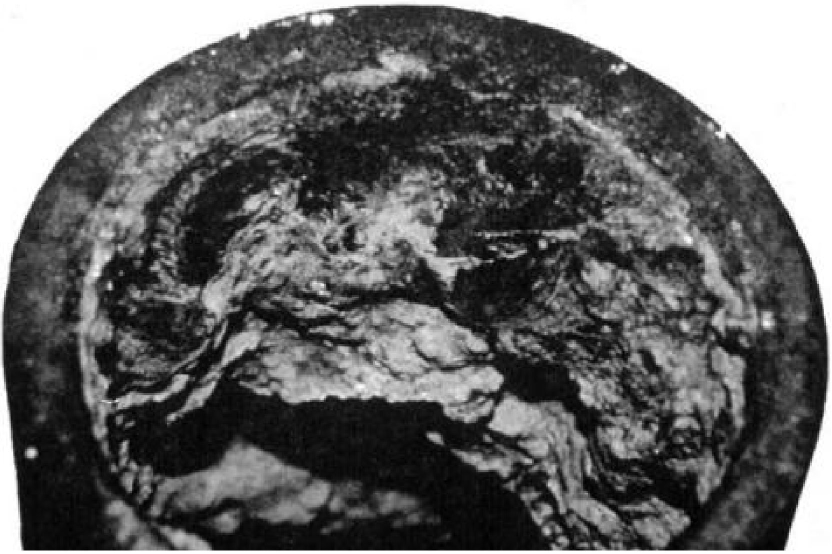

Fig. 21–4. Tuberculation on a cast-iron pipe

site on the metal surface may be an anode one instant and a cathode the next. Why this should happen is not well understood, but it clearly relates to the heterogeneity in the crystalline structure of the metal and possible rapid variations in oxidant and reductant concentrations on the metal surface.

Uniform corrosion is generally a desirable condition, since the metal loss from the plumbing systems is distributed over the entire surface, ensuring a relatively long service life. Corrosion scales that develop on these surfaces are usually compact and evenly distributed. Unlike the tuberculation associated with localized corrosion, they do not reduce the hydraulic capacity of the plumbing system. Often, the scales on uniformly corroding surfaces contribute to the passivation of the surface and serve to limit the underlying corrosion processes (see Fig. 21–5). Metal systems that frequently corrode in a uniform pattern include copper, lead, zinc, and most bimetallic solders. Effective passivation layers need only be a few microns thick. The passivation layer on this copper specimen consists largely of the cuprous and cupric oxides (Cu_2O and CuO).

Pourbaix Diagrams

The most important chemical factors influencing the corrosion of a particular metal in an aqueous environment are the pH of the water and the electrical (redox) potential established between the water and the corrosion surface. These two parameters largely determine the magnitude of the driving force for corrosion, as well as which corrosion products are thermodynamically stable. Using electrochemical principles outside the scope of this chapter, it is possible to graph the relationship between electrical potential for a particular metal and the solution pH. Such a graph is known as a Pourbaix

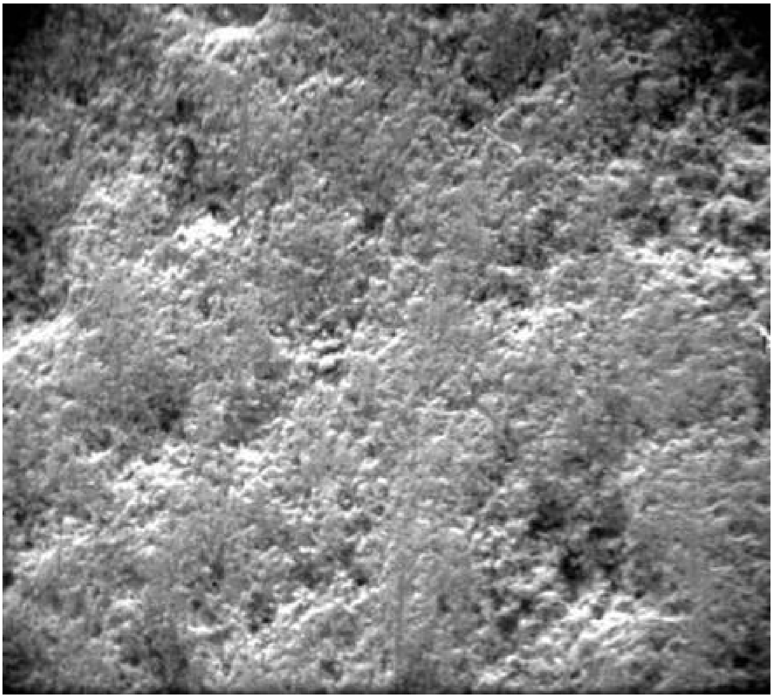

Fig. 21–5. Scanning electron micrograph of the passivation layer on a uniformly corroding copper pipe section

diagram. It is useful for indicating the likelihood of a particular corrosion product, and hence the possible occurrence of insoluble corrosion products that will form on a particular surface.

Figure 21–6 is a Pourbaix diagram of the iron–water system. The upper and lower boundaries of the diagram show the pH-potential regions for H_2–H_2O–O_2 stability. At very low potential, water is reduced to H_2, while at very high potential water is oxidized to O_2. The redox stability of water thus limits the range over which the potential of the metal can vary in its contact with water. The diagram shows that metallic iron and water are never thermodynamically stable simultaneously, and that metallic iron is not stable in the presence of oxygen. The diagram shows that at very low potentials the iron will not corrode (i.e., it is immune). At low pH and intermediate to high potential regions, the diagram shows the stable iron species to be Fe^{3+} or Fe^{2+}. Corrosion will occur at high rates under these conditions. In the high-potential and high-pH region, a variety of solid corrosion products such as $Fe_2O_3(s)$ and $Fe_3O_4(s)$ may form and deposit as a layer on the surface of the iron. These solids generally decrease the corrosion rate by influencing the anodic and cathodic reactions. This diagram is only one possible representation of the iron–water system. It considers only a few of the solid iron oxide species and assumes $[Fe^{2+}]$ and $[Fe^{3+}]$ at 10^{-6} and a temperature of 25°C.

Pourbaix diagrams illustrate a portion of the complexity of the corrosion process. It is important to note that corrosion can be influenced by many other parameters and

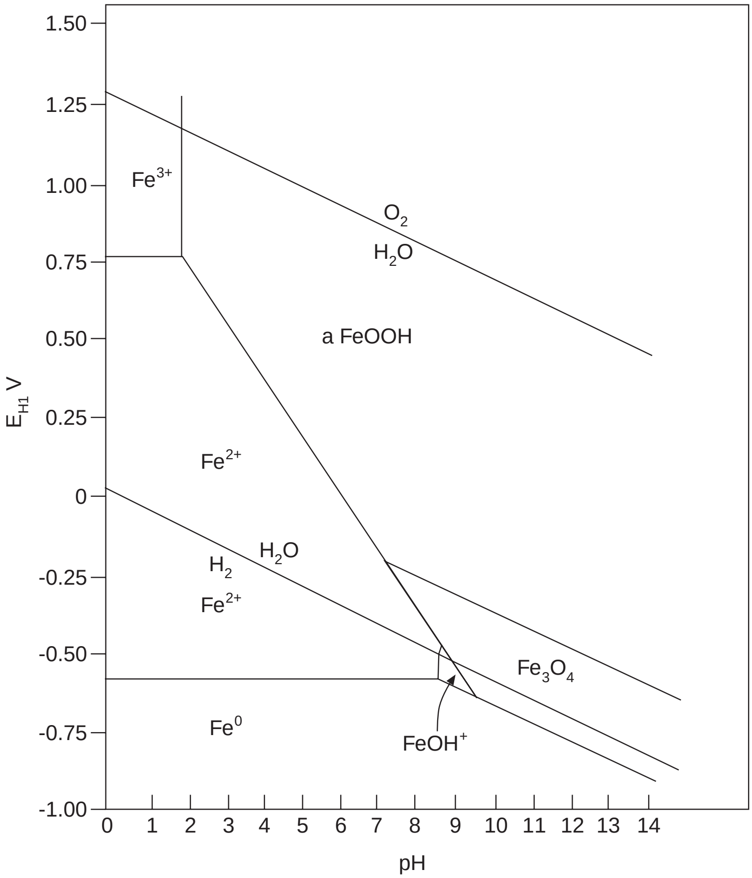

Fig. 21–6. Pourbaix diagram of the iron-water system

that the actual corrosion rate of a metal covered by a corrosion scale depends upon the structure and chemical nature of the scale. Thermodynamic considerations alone never give any information about the velocity of the corrosion reactions or the effectiveness of the solids deposited on the pipe at reducing the corrosion rate.

Passivation and Protective Scales

Passivation is a generic term referring to the process whereby a corrosion surface becomes covered with a dense oxide layer, the presence of which serves to substantially reduce the rate of corrosion on the underlying surface. In some instances an effective passivating layer may be only a few molecules thick; in others, several millimeters of corrosion scale may produce only a partially protective corrosion scale. Some metal alloys, such as stainless steels, can rapidly and spontaneously passivate, whereas others, such as mild carbon steels, never achieve a true state of passivation.

The nature of the passivation process is neither well understood nor well defined. The passivating layers that form on a stainless steel are not distinguishable by the naked eye, yet they can be such an effective corrosion deterrent that, once formed, the rate of oxidation on the underlying metal becomes negligible. The protection they afford the alloy is probably achieved via a combination of effects, including development of a diffusion barrier (literally separating the metal from the aqueous environment) and alteration of the electrochemical reactions that take place on the metal surface. In the latter case, the electrochemical effect may be accomplished by interfering with the electron-exchange reactions necessary to support anodic/cathodic reactions on the corrosion surface. Although the electrochemical nature of the passivation process is not well understood, the passivation film is likely more important than the diffusion barrier impacts.

Passivating layers on stainless steel form almost immediately upon exposure to a humid or aqueous environment. Drinking water distribution materials that form effective and thin passivating layers include copper and brass. But most of the common distribution system metals and alloys, such as cast and ductile iron, galvanized steel, mild steel, and lead do not form spontaneous passivating layers like the stainless steels. Passivation for these materials generally means the gradual buildup of a relatively thick corrosion scale that acts as a diffusion barrier. When first exposed to an aqueous environment, the bare metal surfaces will experience a high initial corrosion rate followed by a gradual reduction as the protective scale develops (see Fig. 21–7). The chemical nature of the scale, its density and thickness, are determined by reactions of the corrosion products with the water chemistry; hence the protective qualities of the passivation corrosion scale vary from one water quality to another, with corresponding differences in underlying corrosion rates.

Figures 21–8 and 21–9 present photographs of two distinctly different corrosion scales formed on unlined cast-iron surfaces under different water quality conditions. These scales required many years to form and demonstrate the morphologic and chemical complexity of corrosion scales. They consist of several different mineral layers of varying thickness and densities. The photographs illustrate that voluminous corrosion scales can obviously act as diffusion barriers to the transport of dissolved species to and from the corrosion surface. More important, the photographs demonstrate that in addition to being a product of corrosion, corrosion scales are likely to create unique microenvironments in close proximity to the pipe surface that influence the corrosion

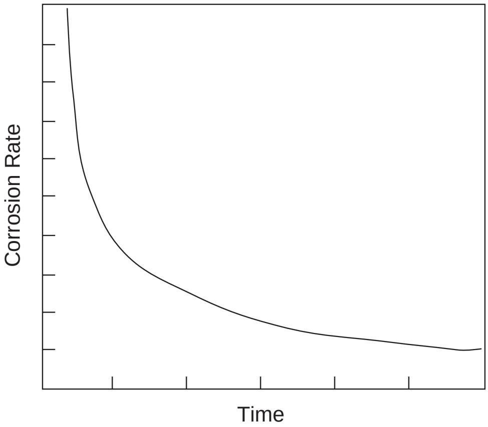

Fig. 21–7. Corrosion rate versus time as a protective scale develops

Fig. 21–8. Corrosion scale cross-section on an unlined cast-iron pipe

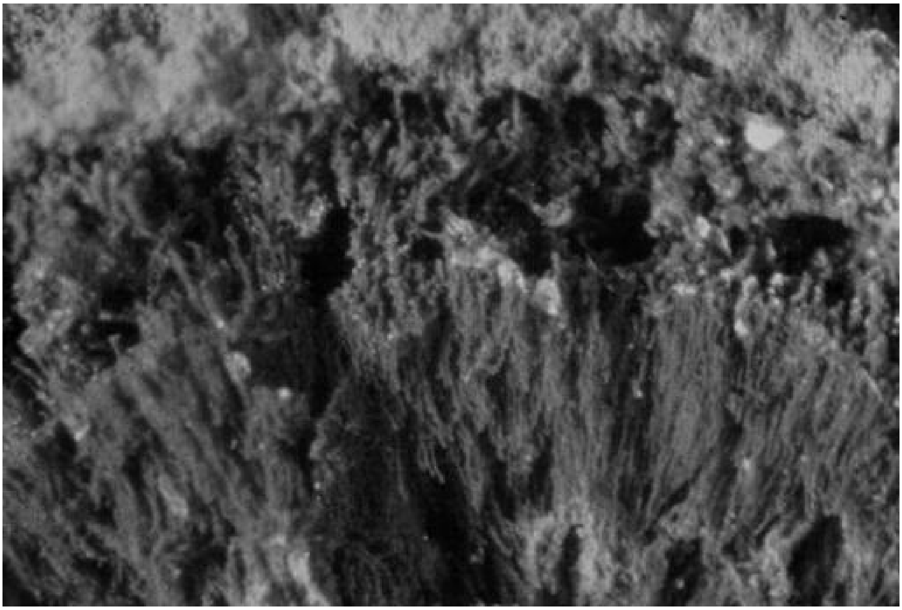

Fig. 21–9. Filamentous corrosion scale on an unlined cast-iron pipe

process. Oftentimes, corrosion scales become a reservoir of oxidized metal that may be periodically sloughed or released to the water flow, depending upon hydraulic and chemical considerations. The thickness and volume of the scale make it conducive to biologic growth, shielding the microorganisms from disinfectants carried in the pipeflow, and, in the case of iron, actually providing an energy source (ferrous iron oxidation to the ferric form) that can be utilized by some bacteria (gallionella).

The corrosion scales on iron and steel pipes are made up of many different minerologies that form over long periods. In this example, the silvery band is a dense layer of Geothite (ferrous carbonate). This mineral layer is thought to be protective of the underlying iron surfaces.

Corrosion scales serve both as a reservoir of the products of corrosion as well as a microenvironment that can support the growth of microorganisms. The filamentous structures in the photograph are the remnants of organisms that mediated the oxidation of ferrous iron to the ferric form. Heavy microbial inclusions in corrosion scales can cause serious taste and odor problems when the corrosion scales are sloughed or disrupted.

CORROSION ASSESSMENT

Calcite Saturation Indices

In its broadest sense, corrosion monitoring falls into one of two categories: direct measurement of the corrosion process (via chemical, electrochemical, or gravimetric means), or some form of indirect measurement relying on water quality information presumably related to the corrosion process. Calcium carbonate saturation indices, the traditional measures of a water's corrosiveness, fall into this latter category.

Conventional wisdom holds that a properly maintained calcium carbonate layer (calcite) is protective of the underlying metal pipe. Although not explicitly stated, the assumption is that the calcite scale is impervious, or relatively so, and provides a form of barrier protection that prevents electrolyte contact and restricts the diffusion of reduced species to and oxidized species from the metal surface. For utilities accepting this premise, the operational goal becomes the maintenance of a deposition layer thick enough to give the desired protection but not so thick as to substantially diminish the hydraulic capacity of the pipeline. The implicit assumption is that corrosion control can be achieved through the manipulation of those water quality parameters influencing calcite saturation (hardness, alkalinity, pH).

A variety of corrosion-control indices based on the concept of calcite saturation have been developed. Some indices are based solely on chemical modeling of calcium carbonate solubility, whereas others are operationally defined. Table 21–1 lists the common calcite saturation indices and the basis for their application. Also included in the table are several general corrosion indices developed from empirical considerations.

Corrosion control via calcium carbonate saturation assumes no distinction between metal types; it is an all-purpose theory that can be applied regardless of the metal, the nature of corrosion on the metal, or the corrosion products produced by the metal. In saturation theory, all corrosion is equally controllable, and all metals can be protected by carbonate scale formation. Langelier popularized the concept in the United States in the early 1930s, and it was Langelier who developed the concept of a $CaCO_3$ saturation pH (pH_s) derived from analysis of the energetics of $CaCO_3$ solubility.[12]

TABLE 21–1. Common Corrosion Indices

Title	Description	Formulae	Interpretation	Comments	Reference
Langelier Index (LI)	The most common calcite saturation index. Compares the measured pH (pH_m) of a water with a hypothetical calcite saturation pH (pH_s). That is, the equilibrium pH at saturation with calcite assuming equivalent alkalinity and hardness levels. pH_s can be calculated several different ways. The method below is a simplified approach developed by Larson and Buswell.	$LI = pH_s - pH_m$ where: $pH_s = A + B - \log_{10} [Ca^{2+}] - \log_{10}[Alk]$ $[Ca^{2+}]$ expressed as mg/L $CaCO_3$ [Alk] is alkalinity expressed as mg/L $CaCO_3$ A and B are constants specific to the alkalinity and hardness level—see reference for tabulated values.	LI > O water is potentially scale-forming (supersaturated with respect to $CaCO_3$) LI = O water is in equilibrium with $CaCO_3$ LI < O water will dissolve $CaCO_3$ deposits (undersaturated with respect to $CaCO_3$)	The presence of a calcite scale is assumed to be protective of the underlying metal. Although there is little evidence to support this assumption, many utilities maintain a slightly positive LI as a corrosion control objective.	7
Calcium Carbonate Precipitation Potential (CCPP)	Predicts the milligram per liter of $CaCO_3$ that will dissolve or precipitate based on solution chemistry. Requires a rigorous interpretation of $CaCO_3$ solubility with ionic strength corrections.	$CCPP = 50{,}045\ (Alk_i - Alk_{eq})$ where: 50,045 is a unit conversion factor Alk_i = measured total alkalinity Alk_{eq} = equilibrium total alkalinity Calculation of Alk_{eq} is complicated and usually achieved by iterative solution of the following equation: $Alk_i = -\frac{2K^1_{sp}\, r_{eq}\, P_{eq}}{t_{eq}(Acy_i - S_{eq})} + \frac{t_{eq}(Acx_i - S_{eq})}{P_{eq}} - S_{eq} + 2[Ca^{2+}]_i$ For a definition of terms and suggested mathematical approach, see reference.	This approach gives a direct estimate of the mass of calcite that will either dissolve or deposit per liter of water at equilibrium conditions.	Frequently used in softening operations to predict potential scaling.	8

Riddick Corrosion Index (RCI)	Empirically derived. Relates water chemistry to solution corrosiveness.	$RCI = 75/Alk[CO_2 + .5(TH\text{-}Alk) + [Cl^-] + 2[NO_3^-](10^2/S_iO_2) \times (DO^{+2})/(DO_{sat})]$ where: TH = total hardness as mg/L $CaCO_3$ Alk expressed as mg/L $CaCO_3$ DO = dissolved oxygen (mg/L) all other parameters as mg/L	$RCI \leq 5$ extremely noncorrosive $6 \leq RCI \geq 25$ noncorrosive $26 \leq RCI \geq 50$ moderately corrosive $51 \leq RCI \geq 75$ corrosive $76 \leq RCI \geq 100$ very corrosive $RCI \geq 100$ extremely corrosive	Development was based on corrosion experiences in soft waters; may not be applicable to heavily mineralized water.	9
Larson's Ratio (LR)	Empirically derived. Relates the concentration of supposed reactive anions to the potential for pitting on iron surfaces.	$LR = \dfrac{[Cl^-] + 2[SO_4^{2-}]}{[HCO_3]}$ where:	$LR \geq 0.4$ are increasingly corrosive $LR < 0.4$ are minimally corrosive	Not applicable to metals other than cast and ductile iron.	10
Feigenbaum Index (FI)	Empirically derived. Relates anion balance to calcium carbonate precipitation potential.	$Y = AH + B[Cl - +SO_4^2\text{-}]\exp(-1/AH) + C$ where: A = 0.00035 B = .34 C = 19.0 $H = [Ca2+][HC_3^-]2/[CO_2]$ Ca, CO_2, Cl^-, $SO_4^32 -$ expressed as mg/L	$Y > 500$ low corrosion $200 < Y > 500$ moderate corrosion $Y < 200$ very corrosive	Developed to predict the effect of heavily mineralized irrigation water on agricultural equipment. Widely used in Israel.	11

Langelier's approach for predicting $CaCO_3$ deposition was straightforward, and received wide acceptance. Early experimentation with the index focused on its ability to predict deposition, not on its precision as a corrosion measure. Its value as an actual predictor of corrosion rates went essentially untested until the 1950s with the work of Stumm, who attempted to correlate the predictive qualities of the index with physical measures of corrosion rates.[13] These studies indicated that calcium carbonate deposition and metal corrosion were not necessarily related. A similar conclusion was reached by Kuch in his studies of German water treatment practices and distribution system corrosion.[14]

Arguably, the most thorough study of the relationship between saturation indices and corrosion was conducted by Singley et al.[15] In a series of tests conducted on several metal types using both pipe loop and batch studies, they attempted to define just how accurate a predictor of metal loss rates the Langelier index could be. Their results showed that, on its own, the saturation index had little or no correlation with corrosion rates, and hence has little value as a corrosion predictor. Moreover, their results suggest that, at least for iron and zinc, pH, by itself, is a better predictor of corrosion rates than the saturation indices; and, since pH is a major determinant of the of the indices, it may account for the observed minimal association between corrosion rate and the saturation value.

The fundamental supposition of the saturation index approach to corrosion control—that calcium carbonate films provide barrier protection for the underlying metal—overlooks the fact that calcium carbonate scales are both porous and friable, and unable to provide meaningful barrier protection in a distribution system unless laid down to a depth that is hydraulically prohibitive. The supposition also disregards the tendency of the corroding metal to develop its own scale, which will consist of the oxidized metal, usually in a variety of mineral configurations. Such scales, while not always providing barrier protection, may nonetheless effectively passivate the surface. Calcium carbonate deposition may be incorporated into these scales, but there is no evidence to suggest that its inclusion provides for more effective passivation.

Maintenance of a positive saturation value does have utility with regard to establishing water quality conditions that may minimize corrosion on some surfaces (i.e., elevated pH and buffer intensity). Saturation indices also retain their utility as an operational tool in precipitation softening. However, it is important to stress that they do not relate directly to corrosion and are not a useful tool in predicting actual corrosion rates.

Direct Measures of Corrosion

Over the past decade a variety of techniques and analytical procedures have been developed that allow a more direct measure of corrosion and its consequences. These procedures make it possible to collect a variety of information about distribution systems and treatment processes that had not been previously available. Specific analytic objectives often include:

1. Determining corrosion rates on the variety of distribution materials, including consumer plumbing materials
2. Determining the concentration of corrosion by-products in water served to the consumer

3. Optimizing corrosion-control programs to minimize both corrosion and the release of metals from corrosion surfaces
4. Conducting forensic studies to determine the mechanisms and cause of corrosion-related materials failure
5. Evaluating corrosion scale, pitting, and tubercle morphologies

Analytic tools vary from simple weight-loss procedures to complex and sophisticated impedance spectroscopy. The most common direct corrosion measures are grouped into the categories summarized in Table 21–2, along with applications and limitations. Although the basis of the methodology is different for each category, all techniques relate to a fundamental aspect of the corrosion process and hence give a direct indication of a corrosion rate.

Direct measures of corrosion derive from the rate at which the base metal of the test surface is oxidized. Although the oxidized metal may remain attached to the surface as a corrosion scale, the rate of metal oxidation is commonly expressed in units of a base metal weight loss ($\text{g-metal/cm}^2 \cdot \text{day}$). If the corrosion process is uniformly distributed over the test surface, the weight loss is convertible to a penetration rate (cm/day) by dividing the unit area weight loss rate by the metal density (g-metal/cm^3). The penetration rate (usually expressed as millinches per year, MPY) describes the rate at which the metal surface is receding due to the corrosion-induced mass loss. It is often used to predict the useful service life of the pipe material. Electrochemical assessment methodologies frequently use current density (Amps/cm^2) as a representation of the electron-exchange rate on the corroding surface. Current density can be converted to a weight loss by application of Faraday's law and the equivalent weight of the test specimen. Table 21–3 presents corrosion rate conversion factors for a number of different metals.

Coupon Techniques Coupon measures represent the cumulative weight loss from a test specimen over an extended exposure period. Of the different analytical methodologies, long-term coupon weight loss is the most useful and readily interpretable corrosion rate measure. Coupon tests give a positive indication of the extent of metal oxidation (the mass of metal lost), as well as visible evidence of corrosion morphology (pitting depth and area). They are useful in the analysis of scale adhesion and mineralogy, and are applicable to any type of metal plumbing surface. Coupon exposures are the referee measurement against which most other forms of analysis are calibrated and compared.

Important coupon testing criteria include:

1. The coupon metal must be representative of the piping material of interest.
2. The water chemistry of the coupon exposure must be indicative of the distribution system.
3. The hydraulic regime across the coupon must reflect the flow regime across the distribution system pipe wall.
4. The duration of the test must allow for development of corrosion scales and/or passivating films that influence the corrosion rate of the underlying metal.

Coupon data are commonly presented as a unit area mass loss per day of exposure ($\text{g-metal/cm}^2 \cdot \text{day}$). It is common, though not necessarily correct, to assume the metal

TABLE 21–2. Corrosion Assessment Options for Metal Plumbing Materials

Methodology	Measurement Form	Applications	Limitations
Coupon exposures	The cumulative measure of metal loss from a well-defined surface over an extended exposure period yields a mass loss rate per unit of surface area.	Metal loss rates generally converted to corrosion penetration rate. Provides visible evidence of both corrosion and scale morphology. Results regarded as the definitive corrosion measure, frequently used as referee technique for other corrosion measures. Can be used to assess pitting propensity and penetration rates.	Requires prolonged exposure durations ($<$90 days). High variability between individual coupon measures. Multiple exposures required to establish statistical confidence limits. Substantial expense. Coupon processing is labor intensive.
Electrochemistry	Most electrochemical techniques give a point estimate of the corrosion current density (electron exchange rate per unit area). The current density can be converted to a metal oxidation rate by application of Faraday's law.	Nearly instantaneous measure suitable for on-line process control configuration. Useful for screening relative efficacy of large numbers of corrosion control strategies. Capable of forensic analysis of corroding surfaces. Can be used to develop electrical model of corroding surface.	Specimen conditioning critical. Generally restricted to uniformly corroding surfaces. Absolute correlation with coupon referee techniques can be poor (linear polarization measures).
Metals release	The corroding surface releases metals to a known volume of contact water. Rate of metals accumulation in solution assumed equivalent to rate of corrosion.	Pipe loops can be designed that represent a laboratory-based facsimile of a plumbing distribution system (tap water metals concentration). Measurements usually made following period of flow stagnation.	Test materials require prolonged conditioning in water quality of interest. Particulate sloughing from corrosion scales induces high degree of release rate variability. Expensive and time consuming. Presence of corrosion scales may interfere with test.

TABLE 21–3. Corrosion Rate Conversion Table

			Weight Loss Rates		Penetration Rates			Current Density, $\mu A/cm^2$
Metal	Density, g/cm^3	Equivalent Weight, g	$(g/m^2)/d$	$(mg/dm^2)/d$	$\mu m/year$	mpy*	mm/year	
Iron	7,860	27.9	1.0	0.010	46.4	1.81	0.046	3.98
Copper	8,920	31.8	1.0	0.010	40.9	1.60	0.041	3.49
Zinc	7,150	32.7	1.0	0.010	51.0	1.99	0.51	3.39
Lead	11,340	104	1.0	0.010	33.2	1.29	0.33	1.07

*mpy = mils per year.

loss is evenly distributed over the test surface, making it possible to present the data as a surface penetration rate. On metals that generally corrode uniformly (i.e. copper, copper alloys, Pb/Sn Solders, etc.), this is acceptable and can be used to calculate an approximation of service life. This type of analysis may, however, be inappropriate for pitting surfaces or other localized forms of corrosion, where a uniform representation of metal loss is likely to grossly overestimate remaining service life. In such cases, penetration of the pipe wall will occur in a small fraction of the time predicted by a uniform assumption of corrosion.

There is no single standard regarding coupon geometry, materials, or exposure protocols in drinking water systems. While some coupon techniques have been developed specifically for drinking water distribution systems, others have been borrowed from different industries. The American Society for Testing and Materials (ASTM) has certified several methodological variants for use in the evaluation of metal loss on coupon exposures. The most widely used technique (ASTM D2688-83 method B) relies on flat rectangular coupon specimens mounted on nonmetallic stems and inserted directly into the flowstream of the pipe, usually at an elbow or tee. This technique can be used to make relative assessments of corrosion at different locations in a distribution system, and for comparative analyses of corrosion inhibitors. However, since the hydraulic flow lines around a flat coupon positioned midpipe are substantially different from the flow lines at the pipe wall, this coupon technique may be inappropriate when a precise estimate of piping corrosion rates is required. Also, it is frequently difficult to obtain flat coupons that are truly representative of pipe materials. Table 21–4 presents a comparative summary of published coupon protocols, including geometry and exposure conditions. Figure 21–10 provides schematics of some of the coupon exposure rigs and mounting hardware.

A rigorous coupon evaluation involves measuring weight loss over an extended period of time and requires coupon sacrifices at multiple points in the exposure cycle. A weight loss against time curve is drawn, and the corrosion rate at any point is the gradient of that curve at that exposure duration. Experience has shown that on most metal surfaces corrosion rates change over the course of the exposure, with the highest corrosion rates occurring at the beginning of the exposure and then rapidly decreasing to a lesser and more constant rate. Hence, any comparison of corrosion rates for a particular metal must be standardized to a specific exposure duration. Figure 21–11 presents a weight-loss curve typical of cast-iron coupon exposures.[21] In this example, the weight-loss gradient at 120 days was interpreted as the stable long-term corrosion rate. Note that the 120-day gradient is approximately one half the gradient at 30 days.

Electrochemical Techniques Corrosion is an electrochemical (EC) process, and electrochemistry can be a powerful tool in its assessment. EC techniques can determine the underlying rate of corrosion as well as characterize the surface reactions that control or limit it. In the past decade, there have been substantial strides in hardware and technique development. The methodology has made the evolutionary adaptation from a purely laboratory based technology to an automated, operational tool useful for compiling a corrosion history, screening a set of corrosion inhibitors, or optimizing a water quality regime for corrosion control.

The suitability of an EC methodology is dependent on the corrosion morphology; EC corrosion measures may be inappropriate on surfaces subject to heavy pitting–type corrosion (i.e., mild steel or cast iron). But for uniformly corroding surfaces such as copper, solders, zinc, and brass, EC methods can often provide an accurate and nearly

TABLE 21–4. Summary of Coupon Protocols Used for Distribution System Corrosion Measures

Protocol Name	Coupon Geometry	Mounting Arrangement	Exposure Duration	Comments	References
ASTM flat coupon standard. (Corrosivity of water in the absence of heat transfer.)	Flat rectangular coupons stamped from sheet stock (0.5 in. × 4 in. [1.27 cm × 10.2 cm]) Both sides exposed to water flow	Held on insulating stem and inserted into flow stream of distribution main at elbow or tee	12 to 24 months	Generally used to monitor corrosion processes on distribution system materials. Used primarily with mild steel or cast-iron materials. Useful for pitting evaluations.	16, 17, 18
Illinois State Water Survey (ISWS) Machined-Nipple Test Protocol	Machined pipe nipples (0.75 in. [1.9 cm] ID, 4 in. [10.2 cm] length) Exterior coated with epoxy resin (60 cm^2 [9.2 in.^{2}] exposed surface area)	Fitted into a machined PVC pipe section and plumbed into bypass off distribution main	Minimum 120 days recommended.	Coupon fabrication requires extensive machining. Coupon processing is chemically based (acid baths).	19
Modified ISWS Coupon sleeve tester (University of Washington)	Short pipe section (0.75 in. [1.9 cm] ID,* 1.5 in. [3.8 cm] length) Exterior phenolic resin coated (20 cm^2 [3 in.^{2}] exposed surface area)	Coupons gasketed and held in clear acrylic sleeve (up to 10 coupons per sleeve). Flow restricted to coupon interior. Easy coupon substitution. Flow controlled to duplicate alternating turbulent and stagnation conditions.	Coupons sacrificed at 30-, 90-, and 180-day intervals	Coupon processing is mechanically based. Measurement precision improved by use of multiple coupon exposures. Can be used to monitor corrosion process on residential plumbing materials.	20

TABLE 21–4. (*Continued*)

Protocol Name	Coupon Geometry	Mounting Arrangement	Exposure Duration	Comments	References
Water Research Center Coupon Rig (WRC, Swindon, United Kingdom)	Flat rectangular coupons (75 mm × 12.5 mm × 4 mm [3 in. × 0.5 in. × 0.15 in.]; 25 cm^2 [3.8 $in.^2$] exposed surface)	Coupons mounted on insulating stems and held in flow control rig	Coupons sacrificed at periodic intervals for up to 22 weeks	Developed for use with cast-iron coupons, but suitable for use with other materials. Flow rig has high- and low-velocity sections	21
Corps of Engineers Research Lab (CERL tester)	Flat rectangular coupons similar to the ASTM D2688-83 standard	Mounted on insulating stems in special pipe rig, designed to hold a large number of coupons under controlled hydraulic conditions Large number of coupons under controlled hydraulic conditions	Not specific	Coupon surface exposure not typical of pipe wall hydraulics	22, 23

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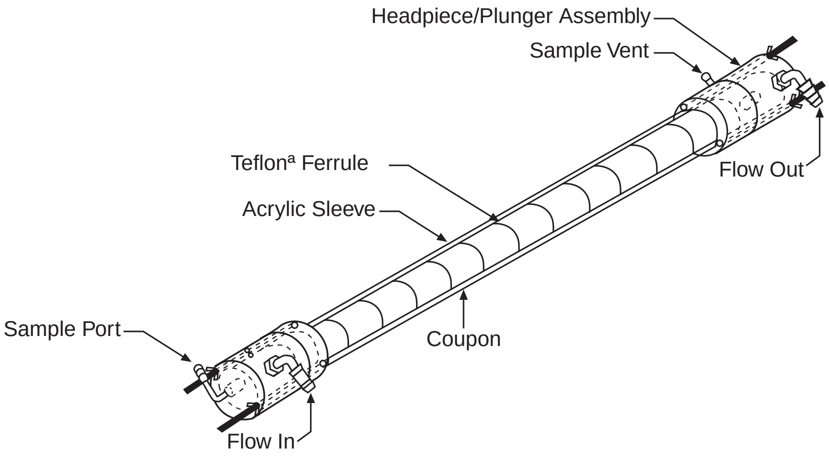
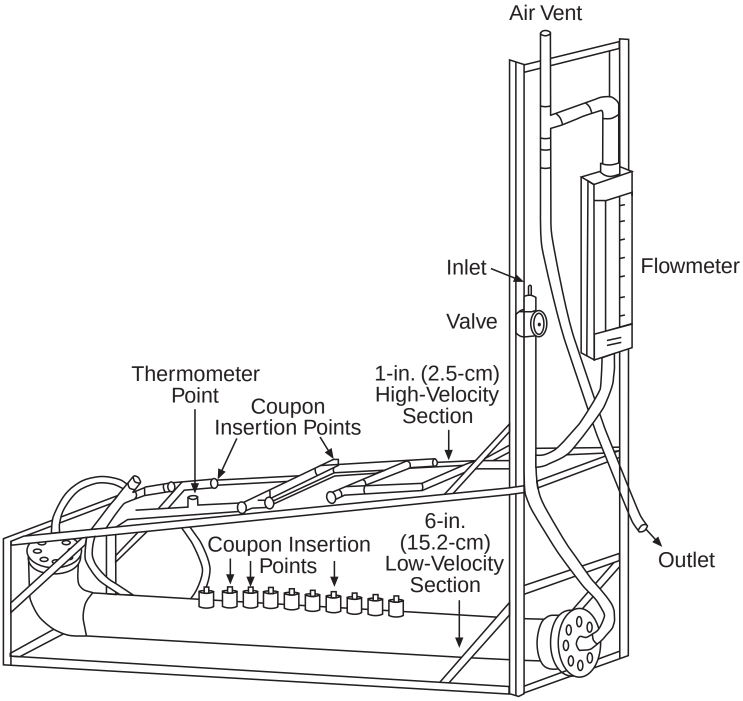
*ID = internal diameter.



Modified ISWS Coupon Sleeve (University of Washington)
Mounting up to 10 Pipe Segment Coupons



WRC Corrosion Rig with Low- and High-Flow Sections
(Uses Flat Coupon Held on an Insulating Stem)

Fig. 21–10. Schematics of common coupon exposure systems (Reprinted from *Internal Corrosion of Water Systems,* 2d ed., by permission. Copyright © 1996, American Water Works Association.)

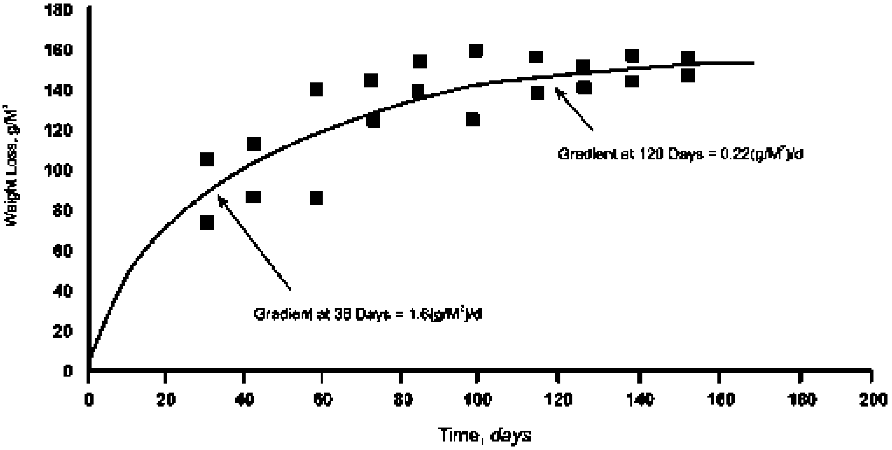


Fig. 21–11. Typical weight-loss curve for iron coupon exposures

instantaneous measure of the corrosion process. [24,25] Unlike weight-loss methodologies, which are cumulative, the EC techniques give a snapshot of the corrosion process. They define the corrosion rate at a specific point in time and, hence, are of value in determining how short-term changes in water quality and flow conditions influence corrosion processes, or how a chemical additives program may be optimized to limit corrosion.

The electrochemical methodologies that have found widest application in distribution system corrosion assessment are those based on polarization measures. Descriptions of three common polarization measures (potentiodynamic scans, linear polarization, and impedance spectroscopy), along with application limitations and references, are summarized in Table 21–5. The fundamentals of polarization were defined 50 years ago in the work of Wagner and Traud, Stern and Geary, and Butler.[37,38,39] The principles are straightforward: a measured current is impressed upon a well-defined surface, and the degree to which this alters (polarizes) the surface potential is measured. The reciprocal relationship between potential and impressed current is obtained for a number of potential offsets (in some cases, a continuous scan over a limited anodic and cathodic range). Figure 21–12 presents a hypothetical polarization curve for a homogeneous metal surface. The corrosion current density, i_{corr}, is estimated from the current-versus-voltage data by fitting the measured polarization data to a model of the corrosion process. The model assumes that both the anodic and cathodic processes are controlled by the kinetics of the electron-transfer reactions on the metal surface, which means that for a freely corroding homogeneous metal surface there are two opposing reactions (the anodic and cathodic exchange reactions) to be considered. The mathematical description of the combination of the cathodic and anodic exchange reactions takes the following form:

$$i = i_{\mathrm{corr}}(\exp(2.3(E - E_{\mathrm{corr}})/B_a) - \exp^{-}(2.3(E - E_{\mathrm{corr}})/B_c))$$

TABLE 21–5. Summary of Electrochemical Corrosion Assessment Methodologies

Name	Data Form	Operating Principle	Applications	Precision	Equipment Requirements	References
Electrical resistance	Resistance change in slide wire probe	Increased electrical resistance on a corroding wire is correlated to a decrease in cross-sectional area due to corrosion-induced metal loss.	On-line continuous corrosion measure	Poor. Is useful only as a relative indicator of change in corrosion rate.	Slide wire probe Kelvin Bridge Package cost: <$5,000	26, 27
Potentiodynamic scans	Anodic-cathodic current-potential curve developed from polarization scans on both sides of the freely corroding surface potential	Electrokinetic interpretation of polarization data—application of the Butler Volmer equation.	Point measure of corrosion current and Tafel constants Pitting potential	Yields reliable results on uniformly corroding surfaces. Interpretation of data from pitting surfaces is difficult.	Polarization cell Potentiostat Data logger Data interpretation software Package cost: <$15,000	28, 29
Linear polarization	Polarization resistance (*Rp*)	Single-point polarization sequence—linear relationship between *Rp* and corrosion rate.	Intermittent on-line corrosion rate measurements	Variable (dependent upon accuracy of Tafel constant)	Polarization probe (two or three electrode models) Single point potentiostat Package cost: <$5,000	30, 31

TABLE 21–5. (*Continued*)

Name	Data Form	Operating Principle	Applications	Precision	Equipment Requirements	References
Electrochemical impedance spectroscopy (EIS)	Impedance spectrum (102 –107 Hz)	Polarization frequency response of impressed AC yields impedance spectrum	Corrosion rate Electrical model of corrosion surface Coating effectiveness	Interpretation of model accuracy is controversial	Polarization cell Potentiostat Frequency analyzer Data interpretation software Package cost: >$25,000	32, 33, 34
Electrochemical noise	Low-frequency changes in freely corroding surface potential	Localized corrosion events emit a base signal characteristic of the magnitude and type of corrosion process	Remains a research tool	Unproven	Galvanic coupling cell Frequency analyzer Zero-resistance ampmeter Package cost: >$20,000	35, 36

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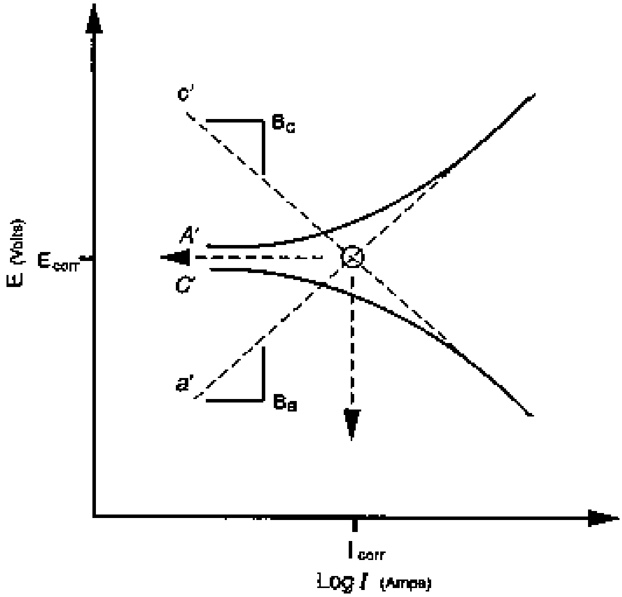

Fig. 21–12. A hypothetical polarization curve for a homogeneous metal surface

where:

i = measured cell current (Amps)
i_{corr} = corrosion current (Amps)
E_{corr} = corrosion potential (Volts)
B_a = anodic Tafel slope (Volts/decade)
B_c = cathodic Tafel slope (Volts/decade)

The model predicts that at E_{corr} each of the exponential terms equals one, and thus the net overall current is zero and unmeasurable. Slightly removed from E_{corr} the anodic and cathodic exchange currents are no longer balanced and the net exchange becomes measurable. Each of the exponential terms in this region makes a substantial contribution to the overall current flux and must be considered together. Further removed from E_{corr}, one of the exponential terms predominates and the other term can be ignored. When this occurs, a plot of the log current versus potential becomes a straight line with a slope equal to the respective Tafel value. The electrokinetic model allows us to extrapolate the linear portions of the log current plot back to their intersection at E_{corr}. The values of the anodic and cathodic currents are equal at that point and represent the freely corroding current density, i_{corr}, which also represents the internal rate of electron exchange between anodic and cathodic sites on the unperturbed surface. The rate of electron exchange can be equated with the conversion rate of metal from an elemental to an oxidized state, and hence can be converted to a mass loss rate. The mass loss rate can be presented as a penetration rate (MPY), which is the conventional representation of a uniform corrosion rate.

Recent electrochemical advances have been made in interpretive procedures of polarization data. The utility of these procedures lies in their ability to characterize—

with a minimum of analyst effort—the form of the current–potential curve. As the statistical and interpretive power of these programs has increased, it has become possible to develop accurate representations of the full polarization spectrum using only the polarization data obtained in the non-Tafel region close to the freely corroding surface potential (E_{corr}). This is of significance in a drinking water context, because it allows for minimal perturbation of corroding surfaces that are sensitive to even minor polarization events. Many of the recently developed software packages provide full graphic functions, sophisticated regression techniques for data fitting, and even experimental simulation options based on modeled parameters. Some popular packages are listed along with applications, limitations, and authors in Table 21–6.

EC assessment is flexible and can be adapted to a variety of experimental needs, which is underscored by the number and diversity of polarization cells developed for specific applications. In many instances it has been possible to produce polarization cells that mimic the hydraulics, geometry, and surface conditions of the actual corrosion surface. The object of a polarization cell is to isolate a test specimen in such a way as to measure its response to an electrical perturbation, while at the same time maintaining a contact environment that closely replicates the freely corroding conditions of its in-service exposure. Common elements of the polarization cell include a test electrode with a well-defined surface area constructed from the material of interest; a stable reference electrode against which the electrical response of the test electrode surface is measured; and a counterelectrode that is a part of the forcing circuit to induce electrical change on the test surface. Several cell designs are presented in Figure 21–13.

Metals Release Metals release tests (sometimes referred to as metal uptake tests) are designed to measure the accumulation of corrosion products in water flowing through a plumbing system or distribution network. Commonly, metal release measurements support a pipe loop corrosion-control demonstration study where the intent is to simulate a residential plumbing system and evaluate the metals concentrations experienced at the consumer's tap under different corrosion-control strategies.

Assessment methodologies that attempt to correlate the underlying metal corrosion rate with the release of corrosion products must recognize the potential for corrosion scale interference. On any metal plumbing surface the newly oxidized metal must pass through a corrosion scale (or passivation film) before release to the flowstream. The oxide form taken by the metal, the mass of corrosion scale formed on the plumbing surface, and the form in which the metal is released from the scale are functions of water quality, solubility, complexation, and hydrodynamic factors unique to the test system. Residence time within the scale may be substantial, and the scale itself may act as a reservoir of the corrosion products. These factors combined make metals release measurements on scaled surfaces highly variable, and sometimes an insensitive indicator of the underlying rate of corrosion.

The AWWA Research Foundation has developed guidelines for constructing and operating a single-pass pipe loop tester representing a facsimile of a residential plumbing network (AWWARF 1990).[46] It provides a surrogate means of evaluating the effectiveness of corrosion-control strategies on residential plumbing systems before systemwide implementation of a treatment program. The pipe rack consists of a PVC manifold supporting multiple piping material flow paths in parallel (see Fig. 21–14). Each flow path contains an approximate 60-ft length of the piping material of interest. The length and inner diameter of the piping material is selected to ensure that the total

TABLE 21–6. **Electrochemical Corrosion Data Analysis Software**

Software Package	Application	Comments	References
Corfit	Capable of determining Tafel values and corrosion current densities from data obtained in the non-Tafel region of typical polarization curves	It is the original and remains the most frequently used routine for polarization data interpretation. Observed data is fitted to the electrokinetic model using a least squares method. The usefulness of the approach is restricted by the limited number of data points the fitting routine can handle. Program written in FORTRAN.	40
Polcorr	Similar application range as CORFIT	Uses an improved linearized least square method of data fitting. Capable of handling up to 51 data points. Designed to run on VAX systems.	41
Polfit	Capable of estimating anodic and cathodic Tafel slopes, corrosion current density, and polarization resistance, and of establishing the confidence limits for each of these parameters	Can provide point-by-point ohmic drop compensation (IR correction) for all observed data points. Provides full graphic representation functions. Capable of simulation modeling of hypothetical polarization parameters. Written in FORTRAN 4.1.	42, 43
Baukamp EIS software	Designed to aid the interpretation of electrochemical impedance spectra. Capable of developing electrical model of corroding surface consisting of distinct electrical building blocks.	Utilizes systems approach. Limited to EIS applications.	44
Gamry software package	Capable of all standard DC polarization data interpretation; full graphics representation; real-time data collection and interpretation features	Software provides instrument control, data logging, and analytical functions for Gamry polarization instruments. Capable of handling over 2,000 data points simultaneously. Operates in Windows environment.	45

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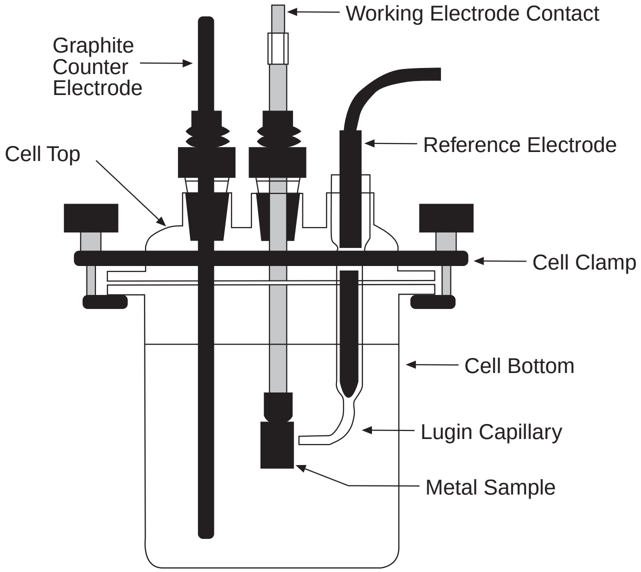
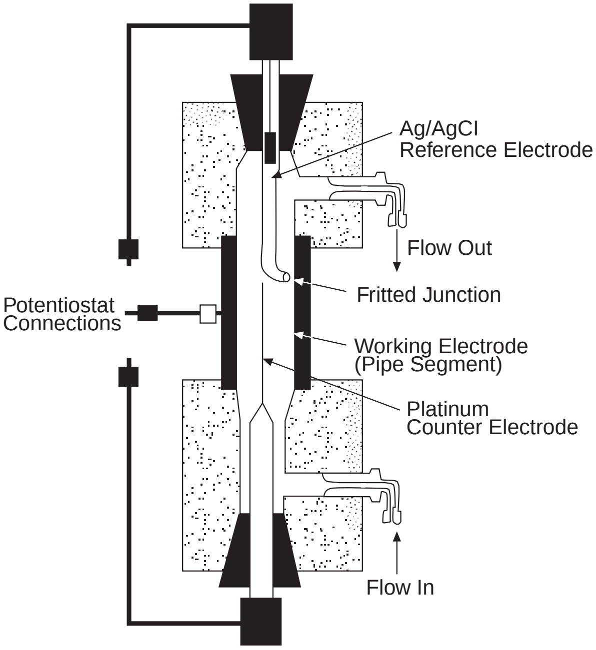


Standard Green Cell



Pipe Section Flow Cell

Fig. 21–13. Electrochemical polarization cells used for corrosion rate measurements (Reprinted from *Internal Corrosion of Water Systems,* 2d ed., by permission. Copyright © 1996, American Water Works Association.)

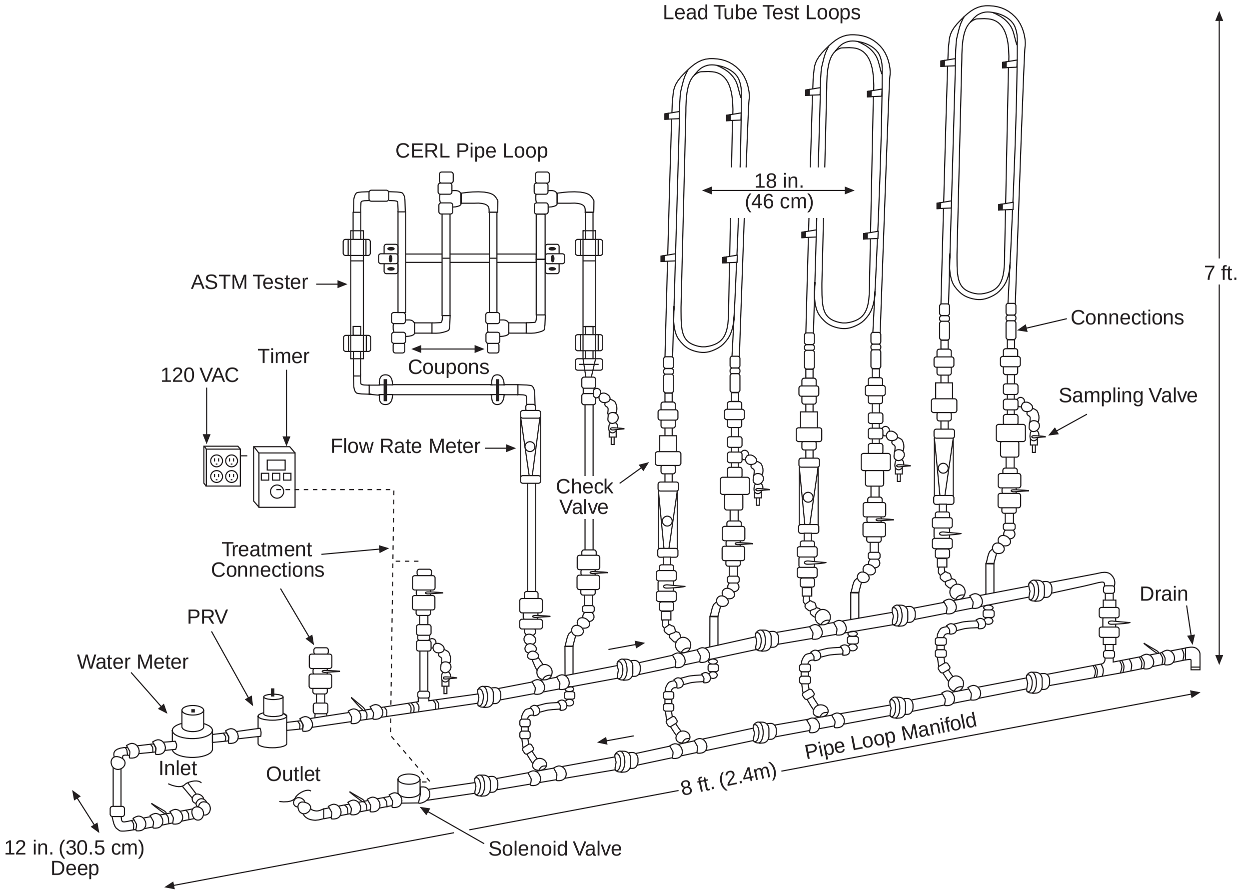


Fig. 21–14. AWWA-RF single-pass pipe loop tester (Reprinted from *Internal Corrosion of Water Systems*, 2d ed., by permission. Copyright © 1996, American Water Works Association.)

water volume held in each path exceeds one liter. Flow rate, temperature, pressure, and water quality of the flowstreams are controlled. Interrupts are programmed into the flow regime to simulate short and extended stagnation periods (similar to daily household operation).

The pipe rack may be configured with three replicate flow paths of the same material, or it may contain flow paths of different materials, including lead tubing, solderless copper tubing, or Pb/Sn soldered (jointed) copper tubing. The soldered copper tubing flow path, if present, contains approximately 70 soldered joints, each exposing between 1 and 3 cm^2 of Pb/Sn solder to the interior flowstream. The solder exposures in contact with the copper tubing constitute a series of galvanic cells. The bimetallic solder alloy is anodic relative to the copper surround, and hence its corrosion rate is accelerated by galvanic action, which results in a higher release rate for lead. The Pb/Sn jointed copper tubing is considered to be the flow path most representative of residential plumbing materials and is recommended for residential corrosion impact assessment.

The metals release sampling protocol calls for standing water samples to be drawn from each of the flow paths following a minimum stagnation period of 8 hours. The respective metals concentrations are used as an indicator of likely standing tap water metals concentrations, which in turn are used as a gauge of water corrosiveness. Metals release measurements are generally made on a weekly basis. The loops require a substantial conditioning period in order to achieve stable metal release rates. Total exposure cycles may run from 3 to 12 months.

The experiences of six utilities using the AWWARF pipe loop protocol have been assessed by Kirmeyer et al.,[47] who found the pipe loop protocol a useful scientific tool for demonstrating the impacts of operational changes and treatment modifications on metals levels in the distribution system. Several important observations relative to the efficacy of the pipe loop approach were reported:

- A high degree of variability exists in the metals release data. Statistical evaluation of pipe rack lead and copper levels shows them to be non-normally distributed (the closest distribution approximation is log-normal). Nonparametric statistical techniques are necessary to make meaningful predictions of metals release trends resulting from changes in water quality.
- Metals levels in the pipe rack respond quickly to changes in incoming water quality caused by treatment interruptions or modifications.
- The principal drawback to the pipe loop protocol is the extended conditioning period required of the test system before meaningful data could be obtained—often in excess of six months.

MITIGATING CORROSION

There are several ways to overcome the difficulties of internal corrosion of plumbing and piping systems. First, and most important, is careful attention to the properties and performance of all materials specified for water contact. A variety of plastics, specialty alloys, and synthetic materials are available that will give reasonable service life for any potable water without significant loss of capacity, excessive failures, release of corrosion products, or other damages commonly associated with corrosion of conventional metal systems. However, water distribution systems are old, complex, and represent a costly investment for any utility. The protection of the current system with its many different metal types is usually the most important corrosion problem facing a utility. The strategy most commonly employed to control corrosion is to change the quality of the distributed water either through pH and alkalinity adjustment, use of chemical corrosion inhibitors, or the blending of different waters.

The corrosiveness of the water passing through the distribution system and residential plumbing can be due to the natural water quality, to treatment processes that have altered the natural water quality, or to any of a number of different chemicals added for treatment purposes. Some of the chemical factors that are important to distribution system corrosion are listed in Table 21–7. Many of these are closely related, and a change in one often changes another. An example of this is the interrelation between carbon dioxide, dissolved inorganic carbon (DIC), and alkalinity—three closely related parameters that are profoundly influenced by each other, and in turn are both influenced by and influence the pH of the water.

In practice, a distribution system consists of many different materials in contact with a distributed water that varies somewhat in quality from one part of the distribution system to another. Because of this, the nature of a corrosion problem is likely to vary across a system, and it is never possible to achieve a control strategy that is optimum for all materials. Most corrosion-control programs are an attempt at compromise. They try to sustain a minimally suitable water quality for each pipe material without exacerbating corrosion on any particular material type. Table 21–8 presents some of the different corrosion concerns associated with the common distribution system materials.

There is no universal consensus on how best to control corrosion on each of the different material types. Because corrosion is often influenced by more than one chemical parameter, a corrosion control strategy that works in one water quality may not be entirely effective in a different water. While a consensus is not possible, some general guidelines on a range of desirable chemical parameters to control corrosion on the different distribution piping and residential plumbing systems have been developed; they are presented in Table 21–9.

Corrosion Inhibitors

Because of the serious economic consequences of corrosion in many industrial settings (oil refineries, manufacturing, etc.), hundreds of different chemical formulations have been developed to inhibit corrosion for site- and material-specific applications. These include compounds such as chromates, molybdates, and phosphonates, all of which can be extremely effective on a variety of metal types under adverse chemical conditions. There are, in fact, few examples of corrosion that cannot be controlled chemically. The more typical issue is what inhibitors are appropriate for the particular environment in which the corrosion is occurring. This is obviously the concern in a drinking water application, where because of public health concerns, the number of chemical additives that can be used is limited to a few inorganic chemical formulations.

In the United States there are dozens of vendors marketing products designated as corrosion inhibitors for drinking water systems. Because of toxicity concerns, all of these products are based on a few phosphate and silicate formulations, and blends

TABLE 21–7. Chemical Factors Influencing Corrosion and Corrosion Control

Factor	Effect
pH	Low pH may increase corrosion rate; high pH may protect pipes and decrease corrosion rates or could cause dezincification of brasses or soft blue hydroxide deposits in copper pipe.
Alkalinity, dissolved inorganic carbon (DIC)	May help form protective coating; helps control pH changes. Low to moderate alkalinity reduces corrosion of most materials. High alkalinities increase corrosion of copper, lead, and zinc.
Dissolved oxygen (DO)	Increases rate of many corrosion reactions.
Chlorine residual	Increases metallic corrosion, particularly for copper, iron, and steel. Reduces microbial-induced corrosion.
Total dissolved solids (TDS)	High TDS tends to increase conductivity and corrosion rate.
Calcium hardness	Calcium may precipitate as $CaCO_3$ and may provide protection and reduce corrosion rates, but may cause turbidity or excessive scaling.
Chloride, sulfate	High levels increase corrosion of iron, copper, and galvanized steel.
Hydrogen sulfide	Increases corrosion rates.
Ammonia	May increase solubility of some metals, such as copper and lead.
Polyphosphates	May reduce tuberculation of iron and steel and provide smooth pipe interior. Also, may increase iron and steel corrosion at low dosages, and may attack and soften cement linings and asbestos-cement pipe. May also increase the solubility of lead and copper and prevent $CaCO_3$ formation and deposition. Sequesters ferrous iron and minimizes occurrence of red water. May revert to orthophosphate and help form protective films.
Orthophosphate	May form protective films.
Silicate	Sequesters metal ions and forms protective films. Silicate solutions increase pH and alkalinity.
Natural color, organic matter	May decrease corrosion by coating pipe surfaces. Some organics can complex metals and accelerate corrosion or metal release.
Iron or manganese	May react on interior of asbestos-cement pipe to form protective coating. May precipitate film on interior of metallic pipes, which may be protective or may cause pitting in copper.
Zinc	May react on interior of asbestos-cement pipe to form protective coating. May react on interior of pipe to form protective coating. May tend to precipitate at cathodic sites and form protective deposits.
Copper	Causes pitting in galvanized pipe.
Magnesium (and possibly other trace metals)	May inhibit the precipitation of calcite form of $CaCO_3$ on pipe surfaces and favor the deposition of the more soluble aragonite form of $CaCO_3$.

Source: Reference 48.

TABLE 21–8. Corrosion and Water Quality Problems Caused by Materials in Contact with Drinking Water

Material	Corrosion Type	Tap Water Quality Deterioration
Cement Based		
Asbestos-cement*		
Concrete	Uniform corrosion	Calcium dissolution. Increased pH values (up to 12). For asbestos-cement pipes, in unstable waters, pH increases and asbestos fibers can be found in the water. Surface roughening, strength reduction, and pipe failure.
Cement mortar†		
Iron		
Cast	Uniform corrosion	Rust tubercles (blockage of pipe)
Ductile	Graphitization	Iron and suspended particles release
	Pitting under uprotective scale	
Steel	Pitting	Rust tubercles (blockage of pipe)
		Iron and suspended particles release
Galvanized Steel	General pitting corrosion	Excessive zinc, lead, cadmium, iron release, and blockage of pipe
Copper	Uniform corrosion	Copper release
	Localized attack	
	Cold-water pitting (type I)	Pipe perforation and subsequent leakage from pipes
	Hot-water pitting (type II)	
	Other types of localized attack	
	Microbial-induced corrosion	Leakage from pipes and sporadic blue deposit release. Rupture of pipes and fittings and consequently leakage.
	Corrosion fatigue	
		Leakage from pipes
Lead		
Lead pipe	Uniform corrosion	Lead release
Lead-tin solder		Lead and cadmium release
Brass	Erosion and impingement attack	Penetration failures of piping
	Dezincification	Blockage of pipes and fittings
	Stress corrosion	Lead and zinc release
Plastic	Unknown	Taste and odor
	Degraded by sunlight and microorganisms?	

Source: Modified from Reference 49.

*No internal lining (e.g., tar).

†Used as internal lining of iron and steel materials.

TABLE 21–9. General Water Quality Conditions to Avoid Internal Corrosion in Various Pipe and House Installation Materials

	Water Quality Parameters											
Type of Material	Corrosion Type	pH	Alkalinity, meq/L	Calcium, mg/L	SO_4, mg/L	Chlorine, mg/L	$\frac{HCO_3}{SO_4 + Cl}$,‡ $\frac{meq/L}{mg/L}$	$\frac{HCO_3}{SO_4}$, $\frac{meq/L}{mg/L}$	O_2, mg/L	Organic Materials	Aggressive CO_2, mg/L	Remarks
Cement-Based Pipe Asbestos cement Concrete Cement mortar	Uniform	>7	>0.3*	>10	<200						<5	The most important parameter is aggressive CO_2.
Iron Cast Ductile Steel	Uniform	>7	>0.2 to 0.5*	Desirable			≥1.5			Important†		
	Pitting								>2			Tends to occur under deposits.
Galvanized Steel							>1.0					After some months, see iron pipes.
Copper	Uniform	>7	Can be low							Might act as an inhibitor	As low as possible	Gives high copper release to the water.
	Cold-water pitting type (I)				As low as possible	As low as possible		>0.8				Type I can be avoided by use of qualified pipe and soldering.
	Hot-water pitting type (II)	>7	1 to 2									
	Other types of localized pitting											
	Microbial-induced corrosion											Chlorine residual >0.4 mg/L.
	Corrosion fatigue											Velocity >1.1 m/s.
	Erosion corrosion											
Lead												Should be replaced in low-alkalinity waters.
Lead pipe		>8										
Lead-tin solder		>8										Orthophosphate dosing at high alkalinity. Flush before drinking.

Source: Modified from Reference 49.

*Scandinavian experience with low-alkalinity waters suggests higher alkalinity, >0.5 meq/L; † high-molecular-weight acidic groups; ‡ the inverse of this ratio is also known as the Larson Index (see Table 21–1).

thereof. There are essentially no other forms of corrosion inhibitors suitable for use in potable water systems. The purity of phosphate and silicate inhibitor formulations is certified by the National Sanitation Foundation (NSF).

Although a thorough understanding of phosphate and silicate inhibitory mechanisms is not possible at this time, it is generally thought that inhibition brought about by both these groups of compounds is the result of passivating film that suppresses the electrochemical corrosion reactions at anodic corrosion sites. Thus, these inhibitors are often referred to as *anodic inhibitors.* Along with electrochemical inhibition there may be the deposition of zinc, iron, or calcium compounds that further suppress corrosion reactions.

Phosphate and silicate compounds have been used in drinking water systems as antiscalants, sequestering agents, and as corrosion inhibitors for well over 50 years. Many of the phosphate and silicate formulations that are now sold as inhibitors specifically for lead and copper surfaces were originally developed for purposes of iron and manganese sequestration, as antiscalants for filter beds, and as a means of dissolving tuberculation on unlined cast iron distribution mains. A discussion of the various phosphate and silicate formulation used in potable water systems follows.

Phosphates The principal types of phosphates used as corrosion control inhibitors are orthophosphates, molecularly dehydrated polyphosphates (often referred to as condensed phosphates), and bimetallic (zinc-containing) phosphates.

Typical orthophosphates are phosphoric acid (H_3PO_4), monobasic sodium phosphate (NaH_2PO_4), dibasic sodium phosphate (Na_2HPO_4), and tribasic sodium phosphate (Na_3PO_4). Sodium orthophosphate compounds, also called crystalline phosphates, are used extensively in North America and Europe as both individual products and in proprietary blends with condensed phosphates. There are several AWWA standards for the purchase of monobasic and dibasic sodium phosphate (AWWA Standard B504-88; AWWA Standard B505-88).

Orthophosphates normally have an inhibiting effect if the corrosion products of the corroding metal can form insoluble phosphate compounds. This will generally improve the protective scale quality by increasing its impermeability and adherence. Pure orthophosphates have been found effective at reducing corrosion on both lead and copper surfaces. Hence, their use has increased dramatically since enactment of the Lead and Copper Rule in 1991. One of the most important determinants of efficacy for application of orthophosphates is pH. Orthophosphates are effective at slightly alkaline conditions, but strong evidence suggests they are of little value at pHs below 7.0. The orthophosphates, however, are relatively insensitive to dissolved solids concentrations. Typical dosage range for orthophosphate is 0.5 to 2.0 mg/L (as PO4), making orthophosphates one of the most economical of corrosion inhibitors.

There are environmental and aesthetic concerns associated with the use of phosphate corrosion inhibitors in general, and orthophosphates in particular. Phosphorus is frequently the limiting nutrient for algal growth in freshwater, and may produce algal blooms in open water reservoirs and in waters receiving wastewater discharges. The orthophosphate form is most effective in stimulating algal growth. However, all of the polyphosphates degrade to orthophosphates with elevated temperatures and sufficient contact time. In short, there is always a potential for increasing the orthophosphate concentration whenever phosphate based inhibitors are used. As a practical consideration, dosing phosphate inhibitors at concentrations as low as possible is appropriate. Communication with and approval by the local water quality regulatory agency is recommended during initiation of a corrosion-inhibitor program of any kind in a municipal water supply.

Dehydrated (Condensed) Polyphosphates This group of phosphate compounds represents a veritable soup of long- and short-chain polymerized molecules, even including some cyclic compounds. The most common compounds in this group are sodium tripolyphosphate ($Na_5P_3O_{10}$), sodium hexametaphosphate (polyphosphate mixture) $(NaPO_3)_6$, and other sodium and potassium polyphosphates. All polyphosphates are formed from the combination of soda ash, caustic soda, or potassium hydroxide with phosphoric acid. Many different formulations are possible; differences stemming largely from heating and crystallization conditions during manufacture. Although a variety of vendor claims are made about the homogeneity and stability of their product, analysis of commercial polyphosphate formulations has shown that these products contain many different phosphate groups, and that the composition will change during long-term storage. The AWWA standard for purchase of sodium tripolyphosphate and sodium hexametaphosphate is AWWA Standard B503–89.

Polymerized forms of phosphoric acid are strong metal-complexing agents and are capable of solubilizing metal oxides and actually dissolve some forms of corrosion scale. Polyphosphates prevent formation of slightly soluble scales of calcium carbonate and calcium sulfate, and sequester and stabilize iron and manganese to prevent red water. The reduction in scale formation can be attributed to the adsorption of the polyphosphate on crystal faces, thus arresting the growth of the crystal. Increased corrosion and metal release rates may occur because of the complexing and sequestering properties of polyphosphates, leading to a higher solubility of metal salts or to the formation of less protective layers.

There is no evidence that pure polyphosphates decrease the solubility of the native corrosion scales that form on lead and copper surfaces. Hence, their effectiveness as corrosion inhibitors on these surfaces is in doubt. In most utility studies, the evidence suggests that orthophosphate, not polyphosphate, is the active form of the corrosion inhibitor for lead and copper. There have been reported successes using inhibitor formulations composed of polyphosphates, but often these formulations are a combination of ortho- and polyphosphates. There is little evidence to suggest that pure polyphosphates play a role in corrosion inhibition. If there is effective metal corrosion protection, it may be due to reversion to the orthophosphate form.

The effectiveness of polyphosphates as inhibitors of iron and steel corrosion has been observed, but studies have not demonstrated conclusively how and why the inhibitors work. Divalent ions, calcium in particular, seem to be needed with the polyphosphates for effective inhibition of steel. The ratio of calcium ion concentration to polyphosphate concentration should be at least 0.2:1, and preferably 0.5:1 or more. Formation of many different minerals in deposits or protective scales is possible during the corrosion of iron, and undoubtedly is affected by the inhibitor formulations.

Bimetallic (Zinc-Containing) Phosphates The bimetallic phosphates combine zinc in concentrations of 5 to 25 percent (by weight) with ortho- or polyphosphates. They were developed and first used as corrosion inhibitors about 1950.[50] These are generally proprietary formulations, but are available from many producers. To maintain the zinc in solution, they are usually provided in a sulfuric or hydrochloric acid solutions and require stainless steel or plastic tanks, pumps, and valves. Many suppliers offer different formulations of bimetallic ortho- or polyphosphate, which differ with respect to the zinc-to-phosphate ratio and solubility. It has been claimed that bimetallic phosphates perform better than individual phosphates at lower dosages, particularly in more mineralized and harder waters. It has also been postulated that zinc forms carbonate compounds and block the cathodic corrosion cell sites, while phosphate forms compounds at the anodic sites. The higher zinc concentrations are thought to reduce the time to establish corrosion control by rapid film formation at the cathode.

Zinc-based compounds have historically been used to alleviate corrosion problems on iron surfaces and are often used in connection with red water–related corrosion problems. Although many utilities use a combination of zinc sulfate and phosphoric acid in their lead and copper programs, the mechanism of zinc action on lead and copper surfaces has never been quantified. Most rigorous corrosion-control pilot programs that have compared zinc-orthophosphate compounds versus simple orthophosphates have shown no significant benefit from zinc additives at operational dosage levels.

The zinc used in the bimetallic corrosion inhibitors is an environmental concern because it contributes to the overall zinc load on wastewater treatment facilities. The prescribed discharge limitation of zinc into the wastewater collection systems of many municipal jurisdictions is often in the range of 0.1 to 1 mg/L in order to prevent excessive zinc concentration in the effluent and to protect the aerobic biological processes of secondary wastewater treatment plants. The zinc will also concentrate in the wastewater sludges and may limit land disposal application. For these reasons, maintaining the zinc concentration at less than 0.25 mg/L in the water distribution system is ordinarily an objective. Bimetallic polyphosphate or zinc orthophosphate solutions are usually effective in the range of 0.1 to 0.3 mg/L.

Phosphate Blends The majority of commercial drinking water corrosion inhibitors contain a blend of orthophosphate and polyphosphate compounds. The predominant polyphosphate component in most blends is either tripolyphosphate, hexametaphosphate, or both. In many cases, mixtures of ortho- and polyphosphates are used to combine corrosion protection with hardness stabilization or red-water suppression.

The fraction of orthophosphate typically ranges from 5 to 40 percent. High proportions of orthophosphate provide more corrosion protection, while high proportions of polyphosphate enhance sequestering. The dosage requirements are primarily dependent on the pH and the calcium concentration in the water. Initiation of corrosion control with blended polyphosphates is often slow; low dosages are used to avoid soft scale formation and discolored water. Positive corrosion control may require three to six months of dosing.

Although extraordinary claims of effectiveness relative to lead and copper corrosion control have been made for many blended phosphate formulations, the preponderance of the evidence suggests that simple orthophosphate is the active constituent and its effectiveness is independent of its source.

Silicates Silicate corrosion inhibitors come in both dry chemical and liquid silicate solutions. In dry form, the silicates are Na_2SiO_3, Na_6SiO_7, and $Na_2Si_3O_7$, with variable waters of hydration. They are produced by fusing sodium carbonate with silica sand. Under heat and pressure, soluble silicates are produced, and these vary in their proportions of sodium oxide (Na_2O) and silica (SiO_2). Both dry solid and soluble sodium

TABLE 21–10. Sodium Silicates Commonly Used As Corrosion Inhibitors

	Chemical Properties				
Type	Na_2O, %	SiO_2, %	Na_2O:SiO_2 Ratio	Silicate Solids, %	Density, g/cm^3
Neutral silicate "N" liquid*	8.9	28.7	1:3.22	37.6	1.38
Disilicate "D" liquid*	14.5	29.0	1:2	43.5	1.53
Metasilicate, granular	20.5	28.7	1:1	58.2	Solid

* PQ Corporation (Philadelphia, Pa.) trade names.

silicates have been used as corrosion inhibitors and sequestering chemicals since the 1930s, and the more common types used in drinking water are listed in Table 21–10.

The "N" soluble silicates with Na_2O:SiO_2 ratios of 1:3.22 are the most commonly used in water with a pH range of 7 to 9 for corrosion inhibition of cold- or hot-water systems up to 180°F (82°C). The more alkaline "D" soluble silicates are used when the pH of the water is 7 or less. AWWA Standard B404–92 is for "N" liquid sodium silicate. The dry granular form is frequently used in pot feeders or "point of use devices" on individual hot-water systems. It is also blended with phosphates and available in many proprietary compounds.

Soft waters of low pH and high oxygen content are most likely to be treated by silicates, although successful use of silicates as corrosion inhibitors has been demonstrated with a relatively wide variety of water qualities. Protection is achieved at silicate dosages varying from 4 to 30 mg/L, with higher dosages used for waters with higher concentrations of hardness, chloride, and dissolved solids, as well as at higher temperatures.[51] In practice, the maximum concentration of silicates (as SiO_2) after dosage is usually limited to 40 mg/L. Due to metal ion–sequestering properties, silicates can also reduce corollary red-water problems, more so than pH or pH and alkalinity adjustment alone.

Relative to lead and copper surfaces, the corrosion protection offered by silicate formulations appears to be caused mainly by pH increase and film formation. There is little evidence to suggest that the silicate component has any inhibitory impact on either lead or copper surfaces. It can readily be shown that the caustic soda component of the silicate formulations increases pH, and hence influences corrosion. However, this same pH adjustment can be achieved at lower cost through the simple addition of a pure caustic soda.

GENERAL CHEMICAL INFORMATION

Several of the corrosion inhibitors have potential problems with regard to handling, storage, or use that should be recognized. Most of the glassy polyphosphates, the crystalline sodium silicates, dry quicklime, sodium hydroxide, and sodium carbonate are very hygroscopic. Unless stored and handled in a low-moisture environment, these chemicals will absorb water and cake, which creates handling and feeding problems.

Bulk storage and handling of dry chemicals is often chosen for larger water systems, whereas bag packaging is chosen for low-use situations and for most of the poly-

TABLE 21–11. Chemicals Used for Corrosion Control of Drinking Water

Chemical Name, Formula, and Common Name	Common Forms	Commercial Strength	Bulk Density, lb/ft^3	Solubility, g/100 g H_2O
Calcium carbonate $CaCO_3$	White or gray; granular or powder	About 95%	100–115	0.0014 at 25°C
Calcium hydroxide $Ca(OH)_2$ High calcium hydrated lime, slaked lime	White powder	72–74% as CaO	25–35	0.19 at 0°C 0.15 at 30°C 0.08 at 100°C
Calcium oxide CaO High-calcium quicklime, burnt lime, unslaked lime	White; lump, pebble, granules, or pulverized	93–98%	55–60	Converts to $Ca(OH)_2$ in solution
Half-burnt dolonite $CaCO_3 \cdot MgO$	White, gray, or tan; granules or powder	About 52% $CaCO_3$ 40% MgO	25–35	0.18
Sodium bicarbonate $NaHCO_3$	White powder	99%	135	6.9 at 0°C 16.4 at 100°C
Sodium carbonate NaOH Caustic soda	Anhydrous; white solid, beads, flakes, or liquid	98.9% NaOH if solid, but 20–50% NaOH in solution form	133	42 at 0°C 347 at 100°C
Liquid sodium silicate $Na_2O \cdot nSiO_2$ Water glass	Opaque, viscous liquid	38–42%		Complete
Zinc-orthophosphates	Liquid	Variable		Complete
Sodium hexametaphosphates	Glassy plates, flakes, lumps, beads, or granular	>80% PO_4	80	Complete
Sodium tripolyphosphates	Powder or granular	>75% PO_4	69	Complete
Monobasic sodiumphosphates	White crystalline solid (granular or powder)	>77.7% PO_4	72	Complete
Dibasic sodiumphosphates	White crystalline solid (granular or powder)	>64.3% PO_4	69	Complete

Source: Adapted from Reference 52.

Note: To obtain bulk density in kg/m^3, multiply lb/ft^3 value by 16.02.

TABLE 21–12. AWWA Standards for Corrosion Inhibitors

Chemical Covered	AWWA Standard No.
Soda ash	B201
Quicklime and hydrated lime	B202
Liquid sodium silicate	B404
Caustic soda	B501
Sodium hexametaphosphate	B502
Sodium tripolyphosphate	B503
Carbon dioxide	B510
Calcium carbonate	Not covered
Half-burnt dolomite	Not covered
Monosodium phosphate, anhydrous	B504
Disodium phosphate, anhydrous	B505

phosphate inhibitors used in potable water systems. The polyphosphates may be produced and purchased as plates, flakes, lumps, beads, and fine or coarse granular or powdered forms. The material is soluble in 50°F (10°C) water at one part to four parts water, and the pH of a 1 percent solution is in the range of 5.8 to 7.3. Hexametaphosphate is available in crystalline form for dry feeder or solution feeding. It is also available in a slowly soluble vitreous form for use in pot feeders in small water systems.

The chemicals most frequently used to control corrosion are listed in Table 21–11 along with application and solubility data.

The AWWA has established standards to define the allowable impurities for most of the common corrosion-control additives (Table 21–12).

TABLE 21–13. Comparative Costs (1994) of Corrosion Inhibitors

Type	Dosage Rate, mg/L	Inhibitor Cost, $/lb	Treatment Cost, $/mil gal
Lime	10–30	0.02	2.00–6.00
Caustic soda	10–30	0.20	10.00–15.00
Soda ash	10–30	0.12	16.70–50.00
Sodium hexametaphosphate	1–4 (PO_4)	0.90 (PO_4)	7.50–30.00
Bimetallic phosphate	0.5–2 (PO_4)	1.50 (PO_4)	6.25–25.00
Zinc orthophosphate	0.1–0.5 (Zn)	2.25 (Zn)	2.00–25.00
Sodium silicate	4–10 (SiO_2)	0.30 (SiO_2)	10.00–25.00
Carbon dioxide	5–10	0.05	2.00–4.00
Phosphoric acid	0.5–3 (P)	0.60 (PO_4)	3.00–18.00
Monosodium phosphate	0.5–3 (P)	1.20 (PO_4)	6.00–36.00
Ortho-polyphosphate blend	0.2–1 (PO_4)	2.50 (PO_4)	4.00–20.00

Source: The above data were taken from a comprehensive guide on determining corrosion costs, corrosion-control chemical costs, and benefit–cost analysis in water distribution systems; see Reference 53.

Note: To obtain $/kg, multiply $/lb by 2.20. To obtain $/m^3, multiply $/mil gal by 0.000264.

Chemical Costs and Dosages

The chemical costs associated with corrosion control are usually moderate or low, and generally the benefits will far outweigh the costs. Table 21–13 presents comparative costs of different corrosion inhibitors and pH control chemicals on a weight and dosage basis.

The pass-through cost to the residential consumer is usually minimal. If the chemical cost is in the range of \$15 to \$20 per million gallons treated, this translates to a per-consumer cost in the range of \$0.75 to \$1.25 per year.

The choice of corrosion inhibitor should be determined by relative effectiveness as related to the price. Evaluation of corrosion-inhibitor effectiveness, as tested by pilot-plant studies, is often important because the level of corrosion reduction possible can vary substantially and should be quantified in terms of percent reduction of corrosion and extension of useful life. A present-worth analysis of water-system corrosion costs with and without inhibitor additions provides a rational basis for choice.

REFERENCES

1. Ryder, R. A., ‘‘Effects of Disinfectant Residuals and Phosphate Inhibitors on Corrosion of Water Distribution Facilities,’’ *AWWA Distribution System Symposium Proceedings,* 41, 1990.
2. USEPA, ‘‘Drinking Water Regulations; National Primary Drinking Water Coolers That Are Not Lead Free,’’ *Federal Register,* Vol. 54, No. 67, 1989.
3. USEPA, ‘‘Drinking Water Regulations; National Primary Drinking Water Regulations for Lead and Copper,’’ 40 CFR, Parts 141 and 142, 1990.
4. Eureau, European Community Drinking Water Standards, ‘‘Propositions D’Amenagement De La Directive Eau Potable,’’ 80/778/CEE, Bruxelles, 1991.
5. OMS, ‘‘Health Based Guidelines for Chemicals in Drinking Water,’’ Geneva, 1992.
6. Environmental Protection Agency, *Lead and Copper Rule Guidance Manual,* Vol. I, 1991.
7. Langelier, W. F., ‘‘The Analytical Control of Anti-Corrosion Water Treatment,’’ *JAWWA,* Vol. 28, No. 10, p. 1500, 1936.
8. Rossu, J. R., and Merrill, D. T., ‘‘An Evaluation of the Calcium Carbonate Saturation Indexes,’’ *JAWWA,* Vol. 75, No. 2, p. 95, 1983.
9. Riddick, T. M., ‘‘The Mechanism of Corrosion of Water Pipes,’’ *Water Works and Sewerage,* Vol. 91, Reference and Data, p. R-133–138, 1994.
10. Larson, T. E., ‘‘Corrosion by Domestic Waters,’’ State of Illinois Department of Registration and Education, Illinois State Water Survey, Bulletin 59, 1975.
11. Feigenbaum, C., Gal-Or, L., and Yahalom, J., ‘‘Scale Protection Criteria in Natural Waters,’’ *Corrosion,* Vol. 34, No. 4, p. 133, 1978.
12. Langelier, W., ‘‘The Analytical Control of Anti-Corrosion Water Treatment,’’ *JAWWA,* 28, p. 1500, 1936.
13. Stumm, W., ‘‘Investigation of the Corrosive Behavior of Waters,’’ *J. Sanitary Eng. Div.,* Proc. ASCE, 86:27, 1960.
14. Kuch, A., ‘‘Untersuchungen zum Mechanismus der Aufeisenung in Trinkwasserverteilungs-systemem,’’ dissertation, Universitat Karlsruhe, 1984.
15. Singley, E., Psigian, R., Ahmadi, A., Pisigian, P., and Lee, T., ‘‘Corrosion and Calcium Carbonate Saturation Index in Water Distribution Systems,’’ EPA Grant No. Report 805400-01-1, Cincinnati, OH, 1985.

16. ASTM D2688-83, Method B, ‘‘Standard Test Methods for the Corrosivity of Water in the Absence of Heat Transfer (weight loss protocol),’’ Philadelphia, 1983a.
17. ASTM G1-81, ‘‘Recommended Practice for Preparing, Cleaning and Evaluating Corrosion Test Specimens,’’ASTM, Philadelphia, 1981.
18. ASTM G46-76, ‘‘Recommended Practice for the Examination and Evaluation of Pitting Type Corrosion,’’ ASTM, Philadelphia, 1976.
19. ASTM D2688-83, ‘‘Method C, Standard Test Methods for the Corrosivity of Water in the Absence of Heat Transfer (machined nipple test),’’ Philadelphia, 1983b.
20. Reiber, S. H., Ferguson, J. F., and Benjamin, M. M., ‘‘An Improved Method for Corrosion Rate Determination by Weight Loss,’’ *JAWWA,* 1988.
21. Williams, S. M., Ainsworth, R. G., and Elvidge, A. F., ‘‘A Method of Assessing the Corrosivity of Waters Towards Iron,’’ Water Research Centre Publication, Swindon, UK, 1984.
22. Temkar, P., ‘‘Development of a Pipe Loop System for determining Effectiveness of Corrosion Control Chemicals in Potable Water Systems,’’ USA-CERL Technical Report N-88/12, 1988.
23. Temkar, P., Schloze, R. J., Maloney, S. W., and Neff, C. H., ‘‘Pipe Loop System for Evaluating Effects of Water Quality Control Chemicals in Water Distribution Systems,’’ Proc, 15th Annual AWWA WQTC, Baltimore, MD, 1987.
24. Reiber, S. H., ‘‘The Electrochemistry of Copper Plumbing Surfaces,’’ *JAWWA,* Vol. 81, p. 114.
25. Reiber, S. H., ‘‘Galvanic Stimulation of Lead-Tin Solder Corrosion,’’ *JAWWA,* Vol. 83, No. 7, p. 97.
26. Rohrback, G. H., ‘‘Determination of Corrosion Rates in a Variety of Different Processes by Means of the Corrosometer,’’ *Proc. of Sixth Regional Conf. of NACE,* Long Beach, CA, 1956.
27. Cooper, G. L., ‘‘Proper Electrical Resistance Corrosion Probe Span Selection,’’ *Electrochemical Techniques for Corrosion Engineering* (R. Baboian, ed.), NACE, Houston, 1986.
28. Dean, S. W., France, W. D., and Ketcham, S.J., ‘‘Electrochemical Methods of Testing,’’ in *Handbook on Corrosion Testing and Evaluation,* John Wiley & Sons, New York, 1971.
29. Dean, S. W., *Electrochemical Techniques for Corrosion Testing* (R. Baboian, ed.), NACE, Houston, 1977.
30. ASTM G 59, ‘‘Standard Practice for Conducting Potentiodynamic Polarization Resistance Measurements,’’ ASTM Philadelphia, 1959.
31. Mansfeld, F., ‘‘Polarization Resistance Measurements—Experimental Procedure and Evaluation of Test Data,’’ in *Electrochemical Techniques for Corrosion* (R. Baboian, ed.), NACE, Houston, 1977.
32. Silverman, D. C., ‘‘Primer on the AC Impedance Technique,’’ in *Electrochemical Techniques for Corrosion* (R. Baboian, ed.), NACE, Houston, 1986.
33. Kendig, M. W., and Mansfeld, F., ‘‘Evaluation of Protective Coatings with Impedance Measurements,’’ *Proc. of the Ninth Int. Congress on Metals Corrosion,* Toronto, 1984.
34. EG&G, Princeton Applied Research, ‘‘Basics of AC Impedance Measurements,’’ Application Note, Princeton Applied Research Inc., Princeton, NJ, 1989.
35. Hladky, K., and Dawson, J. L., ‘‘The Measurement of Corrosion Using Electrochemical Noise,’’ 1981.
36. Dawson, J. L., and Uruchurtu, J., ‘‘Noise Analysis of Pure Aluminum under Different Pitting Conditions,’’ *Electrochemical Methods in Corrosion Research* (M. Dupart, ed.), Ranstech Publications, Switzerland, 1986.
37. Wagner, C., and Traud, W., *Z. Elektrochem* 44:391, 1938.
38. Stern, M., and Geary, A. L., *Journal of Electrochemistry Soc.,* 104:56, 1957.
39. Butler, J. A., ‘‘Kinetic Theory of Reversible Oxidation Potentials at Inert Electrodes,’’ *Trans. Farady Soc.,* 19:729, 1924.

40. Mansfeld, F., *Corrosion,* 29:397, 1973.
41. Gerchakov, S. M., Udey, L. R., and Mansfeld, F., *Corrosion* 37:(696), 1981.
42. Shih, H., and Mansfeld, F., ‘‘Software for Quantitative Analysis of Polarization Curves,’’ Application Note, Korros Data Systems Inc., Los Altos, CA, 1991.
43. Jensen, M., and Britz, D., ‘‘A Linearized Least-Squares Method of Calculations of Corrosion Parameters,’’ *Corrosion* 46:111, 1990.
44. EG&G Princeton Applied Research, ‘‘Electrochemical Impedance Data Interpretation,’’ Application Note, Princeton Applied Research Inc., Princeton, NJ, 1991.
45. Gamry Corp., ‘‘Software Package for Corrosion Analysis,’’ Gamry Corp., Langhorne, PA, 1992.
46. AWWA Research Foundation, ‘‘Lead Control Strategies,’’ AWWARF, Denver, 1990.
47. Kirmeyer, J. K., Sandvig, A. M., Pierson, G. L., and Neff, C. A., *Development of a Pipe Loop Protocol for Lead Control,* AWWA Research Foundation, Denver, 1993.
48. Schock, M. R., ‘‘Response of Lead Solubility to Dissolved Carbonate in Drinking Water,’’ *JAWWA,* Vol. 72, No. 12, pp. 695–704.
49. Vik, E. A., and Hedberg, T., ‘‘Corrosion and Corrosion Control in Drinking Water Systems,’’ in *Proceedings of a Corrosion Workshop and Seminar in Oslo, Norway, March 19–21, 1990,* Houston, TX, National Assocation of Corrosion Engineers (NACE), 1991.
50. Kleber, J. R., ‘‘Use of Bimetallic Glassy Phosphates for Corrosion Control,’’ *JAWWA,* 576: 783, 1965.
51. Lane, R. W., Larson, T. E., Schilsky, and S. W., ‘‘The Effect of pH on the Silicate Treatment of Hot Water in Galvanized Piping,’’ *JAWWA,* 69(8):457, 1977.
52. Parsons, W. A., ‘‘Chemical Treatment of Sewage and Industrial Wastes,’’ Washington D.C., National Lime Assoc., 1965.
53. EES and Kennedy/Jenks/Chilton, *The Economics of Internal Corrosion Control,* AWWA Research Foundation, Denver, CO, 1989.

CHAPTER 22

Water Quality Control in Distribution Systems

INTRODUCTION

If we look at the raw water supply (e.g., a river, reservoir, or aquifer) as the source of raw material, and the treatment plant as the manufacturing facility, then the distribution system is the product delivery system. In the same context, drinking water would be considered a perishable product with a ''shelf life'' defined as the time required to transport water from the end of treatment (manufacturing) to the customer's tap.[1] Consumers demand that the delivery system, particularly with products they consume, be error-free. Microbial contamination can have enormous impacts on public health and on the reputation of the company or organization whose product became unsafe. A waterborne disease outbreak due to a failure in the distribution system will have severe impacts on a water utility when the result includes ill customers, a boil-water order, and a violation of federal law. Other consequences include damaged reputation, lost public confidence, and, perhaps, fines or penalties imposed by a regulatory authority.

The water quality aspects of distribution systems are increasingly important to regulators and utilities, as evidenced by several recent regulatory changes (see Chapter 1, ''Criteria and Standards for Potable Water Quality,'' for details). The Total Coliform Rule established, for the first time, a nationwide regulation requiring every community to have an approved plan for analyzing bacteriological quality at various points in the distribution system and requires public notification whenever these bacteriological standards for are not met. The Lead and Copper Rule requires that samples be collected in the distribution system to monitor concentrations of lead and copper. The Disinfectant/Disinfection By-Product Rule requires measurements of disinfectant residuals and DBPs in the distribution system. No longer is it considered adequate to simply monitor the quality of the treated water as it leaves the water treatment plant.

As research has expanded our knowledge of distribution system water quality and consumers have demanded higher-quality drinking water, the number of issues involving water quality has increased. Traditional distribution-related water quality concerns such as hardness, corrosion, cross-connection control, prevention of waterborne disease, and maintenance of disinfectant residual have been supplemented with emerging concerns such as biofilms, bacterial regrowth, maximum disinfectant residual levels (MRDLs), the dynamics of DBPs in the distribution system, and distribution of groundwater supplies being disinfected for the first time. While in the past it may have been considered that water quality issues were addressed at the treatment plant, biological and chemical reactions in the distribution system are now recognized as important factors in determining the water quality at the customer's tap.

This chapter presents water quality in distribution systems in terms of the historic and current water quality concerns, emerging issues, and mitigation of water quality problems.

TRADITIONAL CONCERNS

Hardness

Hardness of a water is a source characteristic and is not altered significantly in the distribution system. However, hard water affects distribution system components, particularly by encrusting service pipes, water heaters, and plumbing fixtures in the consumer's home and business.

Corrosion

Water quality in a water distribution system can be affected by the chemical interactions between the water and the pipe that occur in the form of corrosion. The relationship between corrosion and water quality is complex; it is discussed in detail in Chapter 21, ''Distribution System Corrosion: Monitoring and Control.'' Corrosion scales, which are accumulations of oxidized metal combined with minerals, such as calcite and bacteria, form on the inside lining of the distribution system pipes. Corrosion at the surface of the pipe can be reduced (passivation) when a sufficiently thick coating of oxidized corrosion products has formed.[2] Disruption of the passive layer by changing the chemical characteristics of the water (e.g., pH, oxidation-reduction potential) can degrade water quality in the distribution system. Release of bacteria that inhabit the scales and colored water events can be caused by disruption of the passivation layer. Corrosion of cement-mortar lined pipe can lead to pH values as high as 12.[3]

Prior to 1991, water utilities addressed corrosion primarily because of economic costs associated with pipe failures, plumbing fixture damage, etc. The leaching of lead and copper into water due to corrosion prompted EPA to adopt the Lead and Copper Rule to regulate corrosion based on health concerns, as discussed in Chapter 1, ''Criteria and Standards for Potable Water Quality.''

Colored Water

Colored water can appear in the distribution system as a result of the oxidizing effect a disinfectant has on either metals in the water supply or on the metal accumulations in the corrosion scales. Corrosion of copper pipe can lead to blue water problems. Generally, colored water episodes are not a health threat, but, rather, an aesthetic problem related to the color and the particulates that often form during the event.

Waterborne Disease

A primary historical concern has been to ensure that the water passing through the distribution system is protected from pathogenic organisms or other contaminants that can be introduced into the distribution system through leaks or cross-connections. Within the past 40 years, several bacterial agents associated with documented waterborne outbreaks have appeared in the United States. Enteropathogenic *E. coli* was first identified to be the etiological agent responsible for waterborne outbreaks in the 1960s. There have been several outbreaks of pneumonia caused by *Legionella pneumophila* attributed to distribution systems.[4] Recently, there have been numerous documented waterborne-disease outbreaks that have been caused by *E. coli, Giardia lamblia, Cryptosporidium parvum,* or *Legionella pneumophila.* Waterborne diseases caused by pathogenic organisms are discussed in more detail in Chapter 4, ''Microbiology of Drinking Water.''

Residual Disinfectant

Maintaining a residual disinfectant in the distribution system has been a traditional concern in the United States. Secondary disinfectants (those disinfectants added for the specific purpose of maintaining a residual in the distribution system) commonly used in the distribution system are shown in Table 22–1. Residual concentrations of these chemicals range from 0.5 mg/L to more that 5.0 mg/L, depending on the chemical, the historic usage, and the source water and treatment system.

EMERGING CONCERNS

Maximum Residual Disinfectant Levels

As discussed in Chapter 1, ''Criteria and Standards for Potable Drinking Water,'' the Stage 1 D/DBPR[5] regulates the public health risks associated with DBPs and disinfectant chemicals in drinking water. The rule establishes Maximum Residual Disinfectant Levels (MRDLs) and Maximum Residual Disinfectant Level Goals (MRDLGs), creating an emerging concern because disinfectant residuals were previously not limited. For systems that have used historically high levels of disinfectants in their systems to create a biologically stable system, disruption of the stability could occur when the level of disinfectant is reduced to meet the MRDL.

TABLE 22–1. Disinfectants Used in Water Distribution Systems

Secondary (Distribution) Disinfectant	Applicability
Free chlorine	Most commonly used disinfectant (64% of systems in U.S.)
Monochloramine	Increasing use due to its relatively good disinfection byproduct control; second most commonly used disinfectant
Chlorine dioxide	Rarely used as a secondary disinfectant
Mixed oxidants	Recent approach using the synergistic impacts on effectiveness by combining free chlorine, chlorine dioxide and ozone
Others—ozone, ultraviolet radiation	Inappropriate for distribution systems since they do not provide a lasting residual after dosing

Biofilms

Biofilms are the accumulation of microbial cells at the pipe surface. A high bacterial population can be associated with an increased possibility of waterborne disease, taste and odor problems, corrosion, and the need for a higher disinfectant residual. The biofilm also includes inorganic matter. There are three sources of bacteria that can form biofilms: outside contamination from a main break or cross-connection; failure of the treatment barrier, allowing viable bacteria to pass into the distribution system; and bacteria that may have been partially inactivated or injured during the treatment process becoming viable in the distribution system.

Occurrence of a biofilm begins when the microbial cell being carried in the liquid medium becomes entrapped or adsorbed at the surface of the pipe. Metabolism can occur if an energy source (e.g., assimilable organic carbon (AOC) and other nutrients) and other essential nutrients are available. With metabolism comes growth of a population that forms the biofilm.[6] Biofilm microbial populations can include bacteria (including nonfecal coliforms and heterotrophic plate count (HPC) bacteria), *Legionella* bacteria, fungi, and more complex organisms such as larvae, nematodes, and Crustacea. There can be events during which bacteria shed from the pipe surface to release coliforms that are within biofilms. Such events may be a more common cause of coliform detection than fecal source contamination.[6,7] Following a contamination event, viruses and parasites (*Cryptosporidium*) can be present.

Emerging concerns surrounding biofilms in the United States center around two recent national regulatory issues: the Total Coliform Rule (TCR) and the Stage 1 D/DPB Rule. To comply with these rules, a utility may need to change disinfectants or limit its historic dosage of secondary disinfection to meet the disinfectant MRDL or DBP limits. Changes in the type or amount of primary disinfectant can impact biological stability. For example, a change to ozone as primary disinfectant, without mitigating the more biologically degradable organic compounds formed by ozonation, could increase the nutrients (AOC) available for creation of biofilms in the system. Increased nutrients can lead to increased bacterial growth.

Bacterial Regrowth

Regrowth of bacteria in the distribution system can cause a TCR violation. Although the fundamental intent of the TCR is to protect against pathogenic microbial contamination, presumably from an outside source such as a pipeline break or cross-connection, the TCR addresses the occurrence of all coliforms (see Chapter 1, "Criteria and Standards for Potable Water Quality," for details). An acute violation of the TCR occurs when a repeat sample is fecal coliform-positive or *E. coli*–positive or if a fecal coliform-positive or *E. coli*–positive original sample is followed by a total coliform-positive. As a result of this lack of source discrimination, a violation of the TCR can be triggered by nonfecal coliforms from regrowth.[8] The operational and database needs for a PWS to successfully obtain a variance under the TCR are so demanding that control of regrowth is critical.[9]

Disinfection By-products

Utilities may need to change primary and secondary disinfectants in order to meet regulatory limits on disinfection by-products. If a new secondary disinfectant is contemplated, the major considerations for implementation are DBP formation potential and distribution system retention time. Generally, concentrations of trihalomethanes (THMs) increase with increasing residence time in the distribution system, although this may not be true for all systems or all DBPs.[10] Free chlorine continues to form DBPs until precursors have all reacted with the chlorine or the chlorine residual is diminished. Long retention times with chlorine dioxide residuals can cause elevated levels of chlorite ion and chlorate ion, both by-products of chlorine dioxide. Chloramines, which are weaker disinfectants, do not form appreciable DBPs and are often used as secondary disinfectants to reduce DBP formation when residence time is long. An emerging issue for utilities is maintaining a disinfectant residual in the distribution system without violating DBP limits.

Disinfection of Groundwater Supplies

Many groundwater supplies in the United States that initiate disinfection either due to a violation of the TCR or the Groundwater Disinfection Rule may find unanticipated consequences of disinfection. Disinfection can change the chemical nature of water in a distribution system, potentially disrupting corrosion scales and reacting with dissolved iron or manganese. Results of these changes can be colored water events, release of coliform and other bacteria, and potential release of toxic contaminants. One utility in the mid-western United States in 1994 initiated disinfection due to a TCR violation and immediately experienced colored water problems. Investigation of the events determined that arsenic, which existed in the groundwater supply only in trace concentrations, was adsorbed onto iron particles that were released when the corrosion scales were disrupted by the newly introduced chlorine. The results were arsenic concentrations roughly 100 times the MCL. No public health problems occurred because the events were periodic and the utility customers generally avoided drinking the colored water.[11]

Nitrification

Nitrification can have various adverse effects on water quality, including a loss of total chlorine and ammonia residuals and an increase in HPC bacteria concentration, an increase in nitrite and nitrate concentrations, and decreases in alkalinity, pH, and dissolved oxygen concentrations.[12] Nitrification is the microbiological oxidation of ammonia to nitrite followed in a second step of oxidation of nitrite to nitrate. Some amount of free ammonia is always present in water containing chloramines. As chloramines degrade, more free ammonia is released. Under the proper environmental conditions, ammonia-oxidizing bacteria (AOB) oxidize ammonia to nitrite. The nitrite can then exert a strong chloramine demand (1 mg/L nitrite has a chlorine demand of 5 mg/L), destroying part of the chloramine residual. Destroying the chloramine residual releases more free ammonia, thus aggravating the cycle of chloramine destruction. The destruction of the chloramine residual and the growth of AOB can stimulate the regrowth of HPC bacteria and coliforms, potentially causing a TCR violation.[13]

A recent study[12] found that two-thirds of the medium to large U.S. water systems that use chloramines experience nitrification to some degree and that one-fourth had moderate to severe nitrification problems in their distribution systems.

CONTROL OF DISTRIBUTION SYSTEM WATER QUALITY PROBLEMS

Hardness

Excessive hardness can be removed by softening processes (see Chapter 14, ''Iron and Manganese Removal''). Treatment at the source is the most effective method if raw-water hardness levels are so high that they affect the entire community. When lower hardness levels affect only industrial or commercial users, treatment is better left to the individual industry rather than undertaking expensive treatment of the entire water supply.

The pH of the water affects the tendency for calcium precipitates to form and accumulate on pipes and in customers' equipment. As presented in Chapter 13, ''Water Softening,'' the pH of the water should be maintained at less than 8.8 after recarbonation to reduce the potential for calcium precipitation within the distribution system.

Corrosion

Corrosion can be caused by:[14]

- Internal electrochemical reactions caused by aggressive water flowing through the pipes
- External corrosion caused by the chemical and electrical conditions of the soil in which the pipe is buried
- Bimetallic corrosion caused when dissimilar metals are connected
- Stray current corrosion caused by uncontrolled DC electrical currents flowing in the soil

The causes and control of corrosion are complex topics, and the reader is referred to Chapter 21, ''Distribution System Corrosion: Monitoring and Control,'' for a detailed discussion.

Colored Water

Compliance with the proposed Ground Water Rule (GWR) should give pause to many groundwater utilities that may have to add a disinfectant for the first time. These disinfectants may oxidize iron and manganese in the raw water, creating a color problem. Chlorine is a powerful oxidant and can substantially change the chemistry of distributed waters, and the surface character of the plumbing materials it contacts. In certain oxidation states, the raw-water constituents may remain soluble and innocuous, while in other oxidation states the material may quickly precipitate, creating nuisance conditions. Metals most problematic in this regard are iron and manganese, which create red and brown precipitates, respectively.

Chlorination can dramatically change the redox potential of a distribution system. An oxidizing environment is considered to have a high redox potential, while a low redox potential is associated with a reducing environment. Free chlorine addition, even at concentrations of less than 1 mg/L (as Cl_2), can shift the redox potential of oxygen-depleted water by up to 500 mV, converting what was a reducing environment to an oxidizing environment. Such a shift would likely change the character (oxidation state) of dissolved metals such as iron or manganese, as well as force a re-equilibration of the corrosion scales on distribution system piping and premise plumbing. Re-equilibration of the corrosion scales may result in the conversion of the existing metal solids to more oxidized forms, which in turn may alter both the physical and chemical character of the scale.

Problems related to iron oxidation are usually immediately apparent upon chlorination, but generally occur at concentrations substantially higher than those associated with manganese (greater than 0.2 mg/L). Oxidation of ferrous iron (Fe^{+2}) to ferric iron (Fe^{3+}) is rapid, producing ferric oxide particulates that are the cause of almost all red water problems. The removal of oxidized iron and manganese by filtration at the water source is the commonly practiced method of removing color related to these two constituents (see Chapter 14, ‘‘Iron and Manganese’’).

Colored water may also be caused by corrosion of iron (red water problems) or copper (blue water problems) piping in the distribution system. Please refer to Chapter 21, ‘‘Distribution System Corrosion: Monitoring and Control,’’ for a discussion of corrosion control.

Bacterial Regrowth in the Distribution System

Bacterial growths tend to be more of a problem in the summer for several reasons:[15]

- Algal blooms in the raw water may increase the amount of biodegradable organics.
- Reaction rates are faster.
- Increased water demand increases the nutrient and disinfectant flux in the distribution system, increases the shear forces on biofilms, and creates shorter residence times.

The amount of biodegradable organic matter in the finished water has a direct effect on the potential for bacterial growth in the distribution system. The use of ozone for disinfection forms readily biodegradable low-molecular-weight compounds when it oxidizes the less-degradable humic and fulvic acids. Systems that have replaced chlorination with ozonation have found that previously biologically stable waters now cause significant biological growths in the distribution system. In these cases or in cases where the raw water contains significant concentrations of biodegradable organics, it may be necessary to incorporate biological treatment in the water treatment process. This is typically done by modifying the conventional filtration process to achieve biodegradation of organics within the filter media. The design criteria for and results of biological filtration are presented in Chapter 12, ‘‘Filtration.’’

Effective system flushing can reduce this problem. Maintenance of disinfectant residuals is also used to limit biological growths in the distribution system. Increasing the level of free chlorine has not always been successful. In some cases, disinfection has been ineffective for growths on iron pipes even at levels of 5 mg/L of free chlorine for several weeks. Free chlorine reacts with electron donors of lower redox potential, such as corrosion products, and chlorine residuals may rapidly decrease. Monochloramine is a weaker oxidant and reacts more slowly with these electron donors. As a result, monochloramine can penetrate deeper into the biofilm before being consumed. Monochloramine therefore is more effective in controlling biofilms than is free chlorine. In extreme cases, pipe sections that do not respond to elevated levels of disinfectant or flushing have been replaced.[16]

Nitrification Control

As noted above, chloramination is more effective than free chlorine for biofilm control. Unfortunately, it increases the potential for nitrification to occur in the distribution system. Nitrification can have a number of adverse effects:[12] decrease in chloramine residual, increase in HPC, increase in nitrite and nitrate concentrations, decrease in alkalinity, pH and dissolved oxygen, inability to meet TCR requirements, and inability to meet nitrite or nitrate concentration limits.

Changes in nitrite and nitrate concentration during nitrification in distribution systems are usually 0.05–0.5 mg/L, although they can be as high as 1 mg/L. Current regulations limit nitrite concentrations to 1 mg/L and nitrates to 10 mg/L as measured at the point of entry to the distribution system and do not take into account potential increases in the distribution system.

Nitrification may affect TCR compliance by causing a decrease in disinfectant residual, which can lead to increased growth of bacteria. Also, the periodic use of free chlorine in an attempt to control nitrification associated with the use of chloramines can lead to increased bacterial growth.

Simply increasing chloramine dosages will not necessarily result in reduced nitrification. AOB have been found to survive and grow in chloramine residuals of 1.2–1.5 mg/L.[12] Nitrification has occurred in distribution systems even with finished water residuals of 4.6 mg/L.[12]

Several methods have been used in attempts to control nitrification.[15] Periodic breakpoint chlorination is probably the most commonly used method once nitrification has started to occur. It is often effective at controlling episodes but not preventing them. Increasing the ratio of chlorine to ammonia-nitrogen will reduce the ammonia available for potential nitrification and may reduce nitrification problems, but may not completely eliminate nitrification because it can occur even with small amounts of ammonia present. Increasing the amount of chloramines added has been found to be more effective when done before the onset of nitrification than after. Improving distribution system conditions by flushing, cleaning, eliminating dead-end mains, and controlling biofilms may also provide long-term reduction of nitrification problems. Low doses of chlorine (0.05 mg/L have been reported to inactivate 3–4 logs of AOB over several hours.[13] Higher concentrations inactivate all of AOB in as little as 30 minutes. Field studies of five cities showed that the presence of chlorite in the distribution system reduced nitrification. Feeding of sodium chlorite at selected times during the year may be effective in controlling nitrification.

Disinfection By-product Control

Studies on the formation of DBPs in water treatment plants suggest that pH, temperature, chlorine concentrations, reaction time, TOC, and bromide concentrations affect the formation of DBPs, as discussed in Chapter 3, ‘‘Organic Compounds in Drinking Water.’’ Field studies of changes in DBPs in distribution systems have produced inconsistent results. THMs and other DBPs have been reported to increase in concentration, decrease in concentration, and remain unchanged with increasing residence time in the distribution system,[10] although it is generally considered that THM concentrations will increase with increasing residence time. The treatment plant is the primary control point for limiting DBP formation. However, care must be exercised that disinfectants applied in the distribution system to maintain residuals and to control biofilms do not cause a violation of DBP limits.

Cross-Connections

A cross-connection occurs when a source of contaminated water, wastewater, drainage, or gas is connected to and introduced into the distribution system. Cross-connections can and have caused illness and death. Pathogenic organisms, as well as chemicals (fertilizers, pesticides, herbicides, boiler chemicals, etc.), have been introduced into water distribution systems and caused public health problems.[17]

An effective cross-connection control program is an essential element of controlling water quality in the distribution system. Most states assign the responsibility for a cross-connection program to the water supplier and have specific regulations and guidelines that specify acceptable programs and mechanical devices that can be used to prevent backflow into the distribution system. Such programs must provide a systematic procedure for locating and removing all cross-connections. An effective cross-connection program typically has the following elements:[17]

- An adequate plumbing and cross-connection ordinance
- An organization or agency with overall responsibility and authority with adequate staff
- Systematic inspection of new and existing installations
- Follow-up procedures to monitor compliance
- Backflow-prevention device standards and standards for inspection and maintenance
- Cross-connection control training
- Public awareness and information program

Blending Different Source Waters

Utilities may find that they need to blend different source waters to meet growing demands, to supplement a depleted supply, or to comply more efficiently with drinking water regulations. Chemical and physical characteristics of different source waters, both surface and groundwater supplies, can be very different. Blending different waters without analyzing and mitigating the potential effects of the resulting blend can lead to distribution system problems. In one example, the City of Tucson, Arizona, was forced to abandon operation of a large new surface water treatment plant when it was found that the blend of the treated water from this new source and the previous raw water supply created severe corrosion problems in the distribution system.

Blending a new water source into the system can disrupt the equilibrium between corrosion scales on the existing plumbing surfaces and the current water quality. These corrosion scales serve to passivate and protect the underlying metal. They also serve as a reservoir of corrosion products (oxidized metals) that can be released when the corrosion scale is forced to equilibrate to a new water quality. Re-equilibration is the chemical process by which corrosion scales (metal oxides) come to a new thermodynamic equilibrium with a changing water quality. This usually involves solubilization of existing mineral scales and may include a change in oxidation state for some of the metal components. It can lead to a physical weakening of corrosion scales, which in turn makes the scale susceptible to hydraulic scour, flow reversals, and water hammer. The potential water quality impacts of these events relate primarily to increased particulate release (turbidity), potential red water episodes, higher coliform and HPC counts, and the potential for increased corrosion on the base metal of the pipe wall.

Because of their volume and friability the corrosion scales of greatest concern are iron-based and the plumbing surfaces of most concern are distribution mains of unlined cast or ductile iron pipe, and residential plumbing comprised of galvanized pipe. In many cases, the galvanized layer will have long since corroded away, leaving a mild steel corrosion surface in homes with this plumbing. This type of residential plumbing has generated the most serious re-equilibration problems, largely because of its proximity to the consumer's tap and the abundance of iron oxide scales that form on the exposed mild steel surfaces. Addition of corrosion inhibitors may be appropriate during the period of re-equilibration.

As a result of weekly or diurnal variations in demand, portions of a system may see a variable water quality that oscillates between that of the different sources. The oscillating water quality may be particularly problematic because of the possibility that the unstable mineral content could prevent the effective equilibration and passivation of corrosion surfaces. As a result, the system may remain at risk for excessive metal release, colored water, and other aesthetic concerns for an extended period of time.

OPERATION AND MAINTENANCE TO IMPROVE WATER QUALITY IN DISTRIBUTION SYSTEMS

Sampling / Monitoring

As discussed in Chapter 1, ‘‘Criteria and Standards for Potable Water Quality,’’ sampling within the distribution system is required to determine compliance with the TCR, lead, and copper rule and D/DBP rule.

The water supplier should also consider sampling and monitoring of the quality of the water at various points in the distribution system beyond the regulatory requirements. This added information can be used to determine the quality of the product it is delivering to its customers and to develop an understanding of the dynamic changes in water quality that occur as water moves through the distribution system. For example, the Denver, Colorado, Water Department has installed remote chemical sensors (RCS) throughout its distribution system.[18] The RCS measure total chlorine, conductivity, pH, temperature, and turbidity. The RCS allowed the department to phase out labor-intensive manual monitoring. The monitoring begins at the point where the water leaves the treatment plant and continues throughout the distribution system to provide information on the water quality dynamics of the distribution system. In one instance, the remote monitoring system alerted the staff to problems with elevated pH caused by relatively low flows in the long, new concrete transmission mains to the new Denver airport. Using the monitoring information, the staff identified the extent of the system affected by this problem and initiated a remedial flushing program.

Maintenance

Maintaining the distribution system to remove sediments, scales, and accumulated nutrients has long been considered an important aesthetic and microbial water quality control function of the water supplier. Periodic flushing of the pipe system extremities by opening fire hydrants is often used for these purposes. This procedure provides regular removal of scales that could become a source of colored-water problems and bacterial contamination. The mechanics of an effective flushing program are made up of several elements.[16,19–22] Good planning and public notification are important. Hospitals and laundries should be warned about impending flushing. Crews should have proper safety equipment, including lights and reflective gear if work is done at night. Hydrants should be flushed from the clean water source outward. In general, a large main should not be flushed from a smaller main, and valves and hydrants should be opened and closed slowly to prevent water hammer. Care must be taken not to reduce pressure below 230 kPa (20 psi) at nearby customers and high local elevations. Flow should be directed in a way that minimizes damage and disruption of traffic. Flushing should not be conducted when the temperature is likely to drop below freezing before the water can run off roads and sidewalks. Flushing should be conducted at times when water is plentiful and reservoirs are full rather than during drought periods.

Hydrants should be opened to generate a velocity suitable for scouring solids from pipes. The velocity should be at least 2 ft/s (0.61 m/s) to suspend sediment and should be no more than 10 ft/s (3.1 m/s) to minimize the potential of water hammer in start-up and shutdown. Most of the scouring is accomplished within the first seconds after maximum velocity is reached and that velocity can then be decreased. Flushing should continue until the water clears up or disinfectant residuals increase.

Flushing can be a source of water quality degradation. For example, the City of Cleveland initiated a flushing program primarily to remove accumulated phosphate from the distribution system.[14] Increases in HPC were observed at several locations following flushing, but HPC levels remained below the recommended maximum of 500 CFU/mL. They noted that flushing provided increased chlorine residual and decreased turbidity as well as reducing the phosphates. They found no benefit to using flushing velocities greater than 2.5 ft/sec. Other cities have found that flushing at 2.5–3.5 ft/sec at least twice per year by opening fire hydrants in the target areas is beneficial.[17] Some systems have found that flushing of dead-end mains must be done on a weekly basis to eliminate customer complaints. Flushing programs may also reduce water meter repairs. More recent management of flushing plans includes utilizing unidirectional flushing where the distribution system is systematically cleaned from source to extremity.

If flushing proves inadequate, air purging or mechanical devices such as swabs or pigs may be necessary. Swabs are polyurethane foam plugs that are somewhat larger than the inside diameter of the pipe. They are forced through the pipe by water pressure. They are effective for slime, soft scale, and loose sediment but are not effective for hardened scales. Pipe-cleaning pigs are stiff, bullet-shaped foam plugs that are forced through the mains by water pressure. They are harder than swabs and can remove harder encrustations. When deposits are especially dense, a rod similar to a sewer-cleaning rod can be used to pull a cutter through the main. The cleaning process may remove encrustations or tuberculation that will then lead to leaks that must be repaired.

WATER QUALITY MODELING IN DISTRIBUTION SYSTEMS

Just as hydraulic computer models have become an important tool for understanding distribution flows, pressures, and performance over time, models that incorporate water quality are increasingly being applied and researched. Early work was limited to steady state models that were useful tools for investigating contaminant movement under constant conditions. Subsequent efforts developed public and private sector models that simulate the movement and transformation of contaminants under temporally varying conditions. These modules include EPANET, CYBERNET, QUALNET, ALCOL, SANCHO, H20NET and Stoner Syner GEE.[16]

Water quality modeling uses mathematical models to calculate various water quality constituents and indicators at different points in the distribution system. By assuming specific operating conditions and establishing model boundaries, water quality parameters can be determined. Potential uses for models include predicting disinfectant decay, disinfection by-product formation, pH changes, and nitrification, among others. In terms of water quality, there are three types of models:

1. Models that calculate changes in nonreactive constituents. These models are particularly useful for applications in which the physical condition involves dilution of the constituent over time. Example applications include determining the change in concentration of total dissolved solids, pesticides, fluoride, and other parameters that do not react chemically or biochemically in the distribution system.

2. Models that calculate changes in disinfectants. These models track the decrease in chlorine concentration following dosing because it reacts with other chemicals (organic and inorganic); it reacts with biofilms; it is depleted in the corrosion process; and the concentration of chlorine changes due to mass transfer between the water and the pipe wall.[23] The EPANET model[23,24] is a public-sector model that has been used for water-quality modeling by many utilities. EPANET uses the Hazen-Williams formula, the Darcy-Weibach formula, or the Chezy-Manning formula to calculate the headloss and also models pumps and valves. To model water quality within distribution systems, the concentration of a particular substance is calculated as it moves through the system from various points of entry (e.g., treatment plants) to water users. This movement is based on: conservation of mass within differential lengths of pipe; complete and instantaneous mixing of the water entering pipe junctions; and appropriate kinetic expressions for the growth or decay of the substance as it flows through pipes and storage facilities. EPANET is useful for modeling both formation of TTHMs and the propagation and maintenance of chlorine residuals. QUALNET, another public-sector model, predicts temporal and spatial distribution of chlorine under unsteady-flow conditions.[25]

3. Models that predict changes in microbiological parameters. These models predict changes in microbiological constituents in the distribution system. Both statistical and deterministic models are used to model microbiological parameters. The former type models, e.g., ALCOL,[26] estimates the risk of total coliforms appearing as the result of temperature, disinfectant residual, BDOC concentration, and the concentration of bacteria in suspension. Deterministic models, e.g., SANCHO,[27,28] determine changes in concentrations of related parameters that are used to predict biological presence in the distribution system. In the case of SANCHO, these parameters are chlorine concentration, BDOC concentration, and the fixed and unattached biomass.

Time of Travel

Time of travel is an important consideration related to water quality changes in a distribution system. Certain water quality problems, such as disinfectant decay and concentration of pesticides, are time-dependent and time travel analysis offers the tool to calculate the time variable. The travel time, T, is determined from the equation,

$$T = L/v, \qquad \text{where } L \text{ is pipe length and } v \text{ is velocity of flow.}$$

The age of water provides information on how long water has existed at specific points in the distribution system. Generally, longer travel times imply greater water quality problems. Normally, water can reach a point in the distribution system from more than one path. Therefore, the age of water typically consists of a distribution of ages. Instead of determining the actual distribution of ages at a particular point, average water age is often used.

Underlying Principles of Water Quality Models

The actual physical system of pipes, pumps, valves, fittings, and storage facilities is modeled as a network of links that are connected at nodes in branched or looped configuration.[16] Links represent pipes, pumps, or valves. Nodes serve as junction, source, consumption, and storage points. A network water-quality model predicts how the concentration of a dissolved substance varies with time throughout the network under a known set of hydraulic conditions and source input patterns. Its governing equations rest on the principles of conservation of mass coupled with reaction kinetics. The following phenomena occurring in the distribution system are represented in a typical water-quality model:[29]

Advective Transport A dissolved substance travels in a pipe with the same average velocity as the carrier fluid, while at the same time reacting (either growing or decaying) at some rate. Longitudinal dispersion is usually not an important transport mechanism (intermixing of mass between adjacent parcels of water traveling down a pipe). Advective transport within a pipe can be represented with the following equation:

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + r(C_i) \tag{22–1}$$

where C_i = concentration (M/L^3) in pipe i as a function of distance x and time t, u = flow velocity (L/T) in pipe i, and $r(C_i)$ = rate of reaction (M/L^3/T) as a function of concentration.

Mixing at Pipe Junctions At pipe junctions receiving inflow from two or more pipes, mixing is assumed to be complete and instantaneous. The concentration of a substance in water leaving the junction is simply the flow-weighted sum of the concentrations from the inflowing pipes. For a specific node k, the concentration leaving is calculated as:

$$C_{k,\text{out}} = \frac{\Sigma\, Q_{\text{in}} C_{\text{in}}}{\Sigma\, Q_{\text{in}}} \tag{22–2}$$

where Q_{in} is the flow into node k from various sources (including pipes and other external sources) and C_{in} the concentration in each source feeding mode k.

Mixing in Storage Facilities Most models assume that the contents of storage facilities (tanks and reservoirs) are mixed completely. The concentration in the facility is a blend of the current contents and any entering water. At the same time, the internal concentration could change because of reactions. The following equation expresses these phenomena:

$$\begin{aligned}\frac{\partial(V_sC_s)}{\partial t} &= \text{Accumulation in Storage}\\ &= \text{Mass flow in} - \text{Mass flow out}\\ &\quad + \text{Mass generated in reactions} \qquad (22\text{–}3)\\ \frac{(\partial V_sC_s)}{\partial t} &= \Sigma\, Q_{\text{in}}C_{\text{in}} - C_s\, \Sigma\, Q_{\text{in}} + r(C_s)\cdot V_s\end{aligned}$$

where V_s = volume in storage at time t, C_s = concentration within the storage facility, Q_{in} = flow into storage from all sources, and $r(C_s)$ = the rate of reaction.

Bulk Flow Reactions While a substance moves down a pipe or resides in storage, it can react with constituents in the water column. The rate of reaction generally can be described as:

$$r = kC^n \qquad (22\text{–}4)$$

where k = a reaction constant and n = the reaction order. Some examples of different reaction rate expressions are $r = -kC$ for chlorine decay (first-order decay), $r = k(C^* - C)$ for THM formation (first-order growth, where C^* = maximum THM formation possible), $r = 1$ for water age (zero-order growth), and $r = 0$ for conservative materials (e.g., fluoride).

Pipe Wall Reactions While flowing through pipes, dissolved substances can be transported to the pipe wall and react with materials such as corrosion products or biofilm. The amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the wall influences the overall rate of this reaction. The surface area per unit volume, which for a pipe equals 2 divided by the radius, determines the former factor. The latter factor can be represented by a mass-transfer coefficient, the value of which depends on the molecular diffusivity of the reactive species and on the Reynolds number of the flow.[29] For first-order kinetics, the rate of a pipe wall reaction can be expressed as:

$$r = \frac{2k_wk_fC}{R(k_w + k_f)} \qquad (22\text{–}5)$$

where k_w = wall reaction rate constant (L/T), k_f = mass transfer coefficient (L/T), and R = pipe radius (L). If a first-order reaction with rate constant k_b also is occurring in the bulk flow, then an overall rate constant $k(T^{-1})$ that incorporates both the bulk and wall reactions can be written as

$$k = k_b + \frac{2k_w k_f}{R(k_w + k_f)} \tag{22–6}$$

Even if k_b and k_w where the same throughout a system, the apparent rate k could still vary from one pipe to the next because of variations in pipe size and flow rate.

System of Equations When applied to a network as a whole, Equations 22–1, 22–2, and 22–3 represent a coupled set of differential/algebraic equations with time-varying coefficients that must be solved for C_i in each pipe i and C_s in storage facility s. Externally imposed conditions are:

- Initial conditions that specify C_i in each pipe i and C_s in each storage facility s at time zero
- Boundary conditions that specify values for external flows and concentrations at each node k which has external mass inputs
- Hydraulic conditions that specify the volume V_s in each storage facility s and the flow Q_i in each link i at all times t

Dynamic Model Solutions

Although steady-state models are much simpler to set up and solve, their restrictive assumptions limit their applicability. The development of dynamic models has greatly advanced the utility of distribution system water quality models. Dynamic models account for changes in flows through pipes and storage facilities occurring over an extended period of system operation and how they affect water quality. These models provide a more realistic picture of system behavior than steady-state models.

Solution methods use one of two approaches for spatial and temporal considerations:

Spatial

- *Eulerian approaches* divide the pipe network into a series of fixed interconnected control volumes and record changes at the boundaries or within these volumes as water flows through them.
- *Lagrangian models* track changes in a series of discrete parcels of water as they travel through the pipe network.

Temporal

- *Time-driven simulations* update the state of the network at fixed time intervals.
- *Event-driven simulations* update the state of the system only at times when a change actually occurs, such as when a new parcel of water reaches the end of a pipe and mixes with water from other connecting pipes.

Hydraulic models are used to determine the flow direction and velocity of flow in each pipe at specific intervals over an extended period. These intervals are referred to as hydraulic time steps and are typically 1 hour for most applications. Within a hydraulic time step, the velocity within each pipe remains constant. Constituent transport and reaction proceed at smaller intervals of time known as the water-quality time step.

Adjustments are made at the start of a new hydraulic time step to account for possible changes in flow velocity and direction. Different solution methods include:

Finite Difference Method (FDM) FDM is an Eulerian approach that approximates the derivatives in Equation 22–1 with their finite difference equivalents along a fixed grid of points in time and space.[25]

Discrete Volume Method (DVM) DVM is an Eulerian approach that divides each pipe into a series of equally-sized, completely-mixed volume segments.[29,30] At each successive water-quality time step, the concentration within each volume segment is first reacted and then transferred to the adjacent downstream segment. When the adjacent segment is a junction node, the mass and flow entering the node is added to any mass and flow already received from other pipes. After these reaction/transport steps are completed for all pipes, the resulting mixture concentration at each junction node is computed and released into the first segments of pipes with flow leaving the node.

Time-Driven Method (TDM) This Lagrangian method tracks the concentration and size of a series of nonoverlapping segments of water that fill each link of the network.[31] As time progresses, the size of the most upstream segment in a link increases as water enters the link, while an equal loss in size of the most downstream segment occurs as water leaves the link. The size of the segments between the most upstream and downstream segments remains unchanged. This sequence of steps is repeated until the time when a new hydraulic condition occurs. The network is then resegmented to reflect changes in pipe travel times, mass is reapportioned from the old segmentation to the new one, and the computations are continued.

Event-Driven Method (EDM) EDM is a Lagrangian method that is similar to TDM, except that rather than updating the entire network at fixed time steps, individual link/node conditions are updated only at times when the leading segment in a link completely disappears through its downstream node.[32]

Data Requirements

Hydraulic Data A water-quality model uses the flow solution of a hydraulic model as input. Dynamic models use a time history of flow in each pipe and of volume changes in each storage facility. These quantities are determined by making an extended-period hydraulic analysis of the system being studied. A good hydraulic understanding of a network is essential for computing accurate water-quality results. A poor hydraulic model will lead to a poorly performing water-quality model.

Water-Quality Data Dynamic models require a set of initial water-quality conditions. There are two basic approaches for establishing these conditions. One is to use the results from a field survey. This approach is often used when calibrating the model to field observations. Sites in the model corresponding to sampled sites have their initial quality set to the measured value. Initial conditions for other locations can be estimated by interpolating between the measured values. It is important to get good estimates of quality conditions within storage facilities, which can be slow to change because of the usually slow replacement rate of water in storage. This approach cannot be used when modeling the age of water because there is no way to measure this parameter directly.

The other approach is to start the model simulation with arbitrary initial values and run it for a sufficiently long period under a repeating hydraulic loading pattern until the system's water-quality behavior settles into a periodic pattern. Results from the last period are taken to represent the system's response to the imposed hydraulic loading.

In addition to initial conditions, the quality of all external inflows into the system must be known. These data can be obtained from existing records when simulating existing operations or could be set to specific values when investigating operational changes.

Reaction-Rate Data The specific reaction-rate data needed depends on the constituent being modeled. It is essential that these data be developed on a site-specific basis, because reaction rates can differ by orders of magnitude for different water sources, treatment methods, and pipeline conditions.

First-order rate constants for chlorine decay in the bulk flow can be estimated by performing a bottle test in the laboratory. Water samples are stored in several amber bottles and kept at constant temperature. At several periods of time, a bottle is selected and analyzed for free chlorine. At the end of the test, the natural logarithms of the measured chlorine values are plotted against time. The rate constant is the slope of the straight line through these points. There is currently no similar direct test to estimate wall-reaction rate constants. Instead, calibration against measured field data is used.

A bottle test can also be used to estimate first-order growth rates for THMs. The test should be run long enough so that the THM concentration plateaus out to a constant level. This value becomes the estimate of the maximum potential of THM formation. A plot is then made of the natural logarithm of the difference in the formation potential and measured THM level versus time. The slope of the line through these points is the growth-rate constant.

Water-quality models depend on hydraulic models to provide information on flows and velocities in pipes. An acceptably calibrated hydraulic model is a requirement. Frequently, hydraulic models are calibrated to match pressures measured in the field. Since calibration only for pressure does not assure that flows and velocity are accurately predicted, additional hydraulic calibration may be required when performing water-quality modeling.[33]

Although most water-quality models can be used to represent both conservative and nonconservative substances, the use of conservative substances is more appropriate for calibration of hydraulic models.

The calibration process can be summarized as follows:[16]

1. Conservative tracer is identified for distribution system. The tracer can be a chemical that is added to the flow at an appropriate location or, in the situation where there are multiple sources of water, can be a naturally occurring difference in the water sources, such as hardness. Chemicals that typically are used include fluoride, calcium chloride, sodium chloride, and lithium chloride. Selection of the tracer geneally depends upon government regulations (e.g., some localities will not allow the use of

fluoride), the availability and cost of the chemicals, the methods for adding the chemical to the system, and the measuring or analysis devices.

2. A controlled field experiment is performed in which: the conservative tracer is injected into the system for a prescribed period of time; a conservative substance, such as fluoride, that normally is added is shut off for a prescribed period; or a naturally occurring substance that differs between sources is traced.

3. During the field experiment, the concentration of the tracer is measured at selected sites in the distribution system along with other parameters that are required by a hydraulic model, such as tank water levels, pump operations, flows, and so forth. In addition to the conservative tracer, other water-quality concentrations, such as chlorine residual, can be measured, though these values are not generally used in the calibration process.

4. The model is then run with alternative hydraulic parameter values to determine the model parameters that result in the best representation of the field data. Perhaps traditional pressure and flow measurements are used to perform a first-step calibration. The water-quality model is then used to model the conservative tracer.

5. Good agreement between the predicted and observed tracer concentrations indicates a good calibration of the hydraulic model for the conditions being modeled. Significant deviations between the observed and modeled concentrations indicate that further calibration of the hydraulic model is required.

The concentration of nonconservative substances changes over time as the substances travel through the distribution system because of reactions with other constituents in the water or through interaction with pipe walls. Laboratory and field data are required to establish both the form of the reaction and the reaction coefficients. For transformations that occur in the bulk water, bottle tests can be used to establish the transformation characteristics. For transformations that involve interaction with the distribution system itself (e.g., the pipe walls demand for chlorine residual), in situ field calibration studies are generally required.

Applications

Water quality modeling in distribution systems is being applied for both operations and regulatory compliance. For operations, disinfectant residuals and water age are operating parameters used to characterize water quality in the distribution system. Modeling results can be used for modifying storage tank operations, source contribution analysis, and assessing sampling programs.[34] As a result of the considerable research in THM kinetics in the recent past, modeling THM formation for regulatory compliance with the D/DBP Rule is being applied in actual distribution systems. Modeling of the Cincinnati Water Works system used multivariable nonlinear regression computer software.[35] In this example, system data were obtained for THMs, TOC, pH, time, and temperature. Once the relationships between THMs and the other variables were determined, predictions of DBP formation could be made.

Modeling biofilms in distribution systems offers the opportunity to determine the biological stability of the water. Recent research is providing a better understanding of the biological stability of distribution system water through increased understanding of relationships between AOC and biofilm HPC concentrations, the biofilm characteristics of different pipe materials, and the application of disinfectants.[36]

REFERENCES

1. Kirkmeyer, Gregory J. New Developments in Drinking Water Technology. Presentation at Water Distribution and Our Aging Distribution Systems, Chicago, Illinois, May 7–8, 1996.
2. American Water Works Association Research Foundation (AWWARF), *Internal Corrosion of Water Distribution Systems,* Chapter 1. Principles of Corrosion Of Water Distribution Systems, Vernon L. Snoeink and Ivo Wagner, 1996.
3. Douglas, B. D., Merill, D. T. and Catlin, J. D., ‘‘Water Quality from Corrosion of Cement-Mortar Linings,’’ *JAWWA*, p. 99, July 1996.
4. Lin, Y. E., Vidic, R. D., Stout, J. E., and Yu, V. L., ‘‘*Legionella* in Water Distribution System,’’ *JAWWA,* p. 112, September 1998.
5. USEPA, ‘‘National Primary Drinking Water Regulations; Disinfectants/Disinfection Byproducts Rule. Notice of Data Availability,’’ *Federal Register* No. 3, 1997.
6. Characklis, W. G., South Central Connecticut Regional Water Authority, American Water Works Service Company, and Stoner and Associates, Inc., *Bacterial Regrowth in Distribution Systems*, AWWA, January 1988.
7. Camper, A. K., Jones, W. L., and Hayes, J. T., ‘‘Effect of Growth Conditions and Substrate and Substratum Composition on the Persistence of Coliforms in Mixed-Population Biofilms,’’ *Appl. Environmental Microbiology,* 62:4014–4018, 1996.
8. AWWARF. ‘‘Assessing and Controlling Bacterial Regrowth in Distribution Systems,’’ Mark W. LeChevallier, Betty H. Olson, and Gordon A. Mc.Feters, Co-Principal Investigators, January 1990.
9. Smith, Darrell B., Hess, Alan F., Iwan, Gerald R., ‘‘Distribution System Operation and Maintenance for Compliance with a Total Coliform MCL Variance—A Case Study,’’ 1993 Annual Conference Proceedings: AWWA; Water Quality.
10. Chen, W. J, and Weisel, C. P., ‘‘Halogenated DBP Concentrations in a Distribution System,’’ *JAWWA*, p. 151, April 1998.
11. ‘‘Final Report: Groundwater Disinfection and Arsenic Concentration in the Fremont Distribution System.’’ Dr. Steve Reiber, Glenn Dostal, P.E., Larry Onnen, of HDR Engineering and Jon McCafferty, P.E., Warren Sund, Larry Andreasen of the City of Fremont. Unpublished investigation, November 1997.
12. Wilczak, A., Jacangleo, J. G., Marcinko, J. P., Odell, L. H., Kirmeyer, G. J., and Wolfe, R. L., ‘‘Occurrence of Nitrification in Chloraminated Distribution System,’’ *JAWWA*, p. 74, July 1996.
13. McGuire, M. J., Lieu, N. I., and Pearthree, M. S., ‘‘Using Chlorite Ion to Control Nitrification,’’ *JAWWA,* p. 52, October 1999.
14. Rodgers, M. L., and Pizzi, N. G. ‘‘Distribution Flushing to Improve Corrosion Control and Water Quality,’’ Water Quality Technology Conference, AWWA, 1997.
15. Odell, L. H., Kirmeyer, G. J., Wilczak, A., Jacangelo, J. G., Marcinko, J. P., and Wolfe, R. L., ‘‘Controlling Nitrification in Chloraminated Systems,’’ *JAWWA*, p. 86, July 1996.
16. Mays, L.W., *Water Distribution Systems Handbook,* McGraw-Hill, New York,1999.
17. *Water Transmission and Distribution*, 2nd ed., AWWA, 1996.
18. Orth, R. A., Bradford, R. B. and Lohman, S. R. ‘‘Monitoring Distribution System Water Quality Through the Use of Remote Chemical Sensors,’’ Water Quality Technology Conference, AWWA, 1997.

19. AWWA, 1986b, *Maintenance of Water Distribution System Water Quality*, American Water Works Association, Denver, CO.
20. California-Nevada AWWA, *Distribution Main Flushing and Cleaning*, California-Nevada Section AWWA, 1981.
21. Chadderton, R. A., Christensen, G. L., and Henry-Unrath, P., *Implementation and Optimization of Distribution Flushing Programs*, AWWA RF, Denver, CO, 1992.
22. Chadderton, R. A., Christensen, G. L., and Henry-Unrath, P., ''Planning a Distribution System Flushing Program,'' *JAWWA*, 85:(7)89, 1993.
23. Rossman, L. A., Clark, R. M., and Grayman, W. M., ''Modeling Chorine Residuals in Drinking Water Distribution Systems,'' *Journal of Environmental Engineering*, ASCE, 120:4:803, 1994.
24. Rossman, L. A., EPANET—Users Manual, EPA-600/R-94/057, U.S. Environmental Protection Agency, Risk Reduction Engineering Laboratory, Cincinnati, OH, 1994.
25. Islam, M. R., Chaudhry, M. H., and Clark, R. M., ''Inverse Modeling of Chlorine Concentration in Pipe Networks Under Dynamic Conditions,'' ASCE *Journal of Environmental Engineering*, 123:1003–1040, 1997.
26. Volk, C., and Joret, J. C., ''Parameters Predicitifs de l'Apparition des Coliformes dans les Reseaux de Distribution d'Eau d'Alimentation,'' *Rev. Sci. l'Eau*, 7:131, 1994.
27. Servais, P., et al., ''Modeling Bacterial Dynamics in Distribution Systems. Procedures of 1994,'' AWWA Water Quality Technology Conference, San Francisco.
28. Laurent, P., et al., ''Testing the SANCHO Model on Distribution Systems,'' *JAWWA*, p. 92, July 1997.
29. Rossman, L. A., Boulos, P. F., and Altman, T, ''Discrete Volume-Element Method for Network Water-Quality Models,'' *Journal of Water Resources Planning and Management*, ASCE, 119: 505–517, 1993.
30. Grayman, W. M., Clark, R. M., and Males, R. M., ''Modeling Distribution System Water Quality: Dynamic Approach,'' *Journal of Water Resources Planning and Management*, ASCE, 114:295–312.
31. Liou, C. P., Kroon, J. R., ''Modeling the Propagation of Waterborne Substances in Distribution Networks,'' *JAWWA*, 79(11): 54–58, 1987.
32. Boulos, P. F., Altman, T., Jarrige, P. A., and Collevati, F., ''Discrete Simulation Approach for Network Water Quality Models,'' *Journal of Water Resources Planning and Management*, ASCE, 121:49–60, 1995.
33. Grayman, W. M., ''Use of Tracer Studies and Water Quality Models to Calibrate a Network Hydraulic Model,'' *Essential Hydraulics and Hydrolog*y, Haestad Press Waterbury, CT, 1998.
34. LeCourt, Thomas D., et al., ''Using Water Quality Modeling to Address Distribution System Water Quality Concerns,'' Proceedings AWWA WQTC, 1999.
35. Jiang, Henry H., et al., ''Modeling THM Formation Kinetics and Applications to Distribution System Water Quality Control,'' Proceedings AWWA WQTC, 1999.
36. Ollos, P. J., et al., ''Modeling of Biofilm Accumulation in Drinking Water Distribution Systems,'' Proceedings AWWA WQTC, 1999.

CHAPTER 23

Residuals Management

INTRODUCTION

In the process of removing undesirable constituents from raw water, a variety of waste products, known as residuals, is generated. Among the undesirable constituents are sand and silt, organics in solution, suspended material, ions that cause hardness, total dissolved solids, nitrates, arsenic, radionuclides, bacteria and other organisms, and naturally occurring and synthetic organic matter. The most commonly used treatment processes that remove these materials and subsequently produce a residual are chemical coagulation, lime–soda softening, sedimentation, removal of iron and manganese, taste and odor control, filter backwashing, membrane separation, ion exchange, and granular-activated carbon (GAC) adsorption.

The residuals may be discharged nearly continuously, as from clarifiers, or infrequently, from plain (no sludge collection equipment) settling basins. The residuals may contain clay, silt, sand, carbon, chemical precipitates, bacteria and other organisms and organic substances. The composition of residuals from a treatment plant treating surface water is likely to vary daily, seasonally, and annually, as raw-water quality changes occur. Residuals from one plant may be significantly different from residuals at a nearby plant that uses the same raw-water source, because of differences in treatment technique and chemical types and dosages.

CHARACTERIZATION OF RESIDUALS

Types of Residuals

Sludges Semisolid residuals produced from mechanical water settling or clarification processes (such as screening or presedimentation), in addition to those produced from the clarification of water that has been chemically preconditioned, are typically referred to as sludges.

Coagulant/Polymeric Sludges. Chemical coagulation and subsequent flocculation are widely used water treatment processes for removing clay, silt, dissolved or colloidal organic material, microscopic organisms, and colloidal metallic hydroxides. Aluminum sulfate (alum) is the most widely used coagulant, although iron salts—ferric chloride, ferrous sulfate, and ferric sulfate—also are used as coagulants. Coagulation sludges consist mainly of the hydrous oxide of the coagulant and materials removed from the raw water.

The coagulation sludge characteristics vary with increasing or decreasing proportions of material coagulated from the water. High-turbidity waters from rivers and streams will usually result in sludges that are relatively concentrated and fairly easy to dewater. Low-turbidity water from lakes or reservoirs will produce fewer solids but will often present a difficult sludge processing problem. In general, iron sludges will have a higher solids content than alum sludges, whereas the addition of polymer or lime increases the solids concentration of each.

Softening Sludges. The softening process removes a portion of the calcium and magnesium compounds from raw water to reduce hardness to a predetermined value. Use of lime and soda ash in the softening process results in sludges that are mainly calcium carbonate (80 to 95 percent by weight of solids); other components include magnesium hydroxide, silt, and minor amounts of unreacted lime and organic matter. Softening sludges normally are easy to concentrate and dewater.

Presedimentation Sludge. In many parts of the country, the normal water supplies are muddy rivers, such as the Mississippi. Many river supplies carrying large quantities of suspended solids are presettled, resulting in sludges containing silts, sands, and, if a coagulant is used to aid presedimentation, inorganic precipitates.

Liquid Wastes The most common liquid waste produced at water treatment plants (WTPs) is spent-filter backwash water.

The solids concentration of spent-filter backwash water may vary from 10 to 400 mg/L. Spent-filter backwash water historically has been discharged back to the raw-water source or returned to the head of a WTP to be processed again. Concerns over the recycling of microorganisms, aggravation of taste and odor problems, increase in disinfection by-products, and other issues have greatly reduced the number of WTPs that directly recycle spent-filter backwash without some further treatment.

Another form of liquid waste from filters is filter-to-waste, or rewash, which refers to the wasting of filtered water during operation of a filter immediately after backwashing, during the ripening stage of a clean filter. Microorganisms (e.g., *Giardia, Cryptosporidium,* and viruses) can pass through a freshly backwashed filter.

The filter-to-waste period for ripening a freshly backwashed filter at most WTPs ranges from 15 minutes to an hour. The filter-to-waste flow is generally a fairly clean waste stream. At most WTPs, this flow is equalized and returned to the head end of the plant.[1] Concerns over effects of recycle streams may dictate treatment prior to recycle.

Slow Sand Filter Wastes. Slow sand filters are cleaned by scraping to remove the ‘‘schmutzdecke,’’ a biologically active layer on the top of the filter, which can contain viruses, cysts, and other microorganisms. The sand removed during scraping can contain a fairly active biological population.[2] Some facilities dispose of the removed material by stockpiling it for uses such as winter road sanding, or as soil additives. More commonly, the material is washed and then stored, for later addition back to the filter. The wash water constitutes a residuals stream that may require treatment. Common disposal methods include discharging to a sewer or a receiving watercourse without treatment. Discharge to a receiving watercourse may require a state or an EPA NPDES permit.

A fairly large volume of liquid wastes can be generated during filter-to-waste cycles of slow sand filters, as some waste the filtered water for 24 to 48 hours.[3] The high quality of this water normally allows disposal without treatment.

Ion-Exchange Brine. Regenerant wastes (i.e., brine and rinse water wastes) from ion-exchange (IX) facilities are generated by some softening plants. Well-head treatment facilities, which often include IX facilities for removal of a specific constituent, such as nitrate, also produce brine and rinse water.[4]

The regenerant waste consists of sodium, chloride, hardness ions, and the backwash water and rinse water used. The term *regenerant waste,* or spent brine, frequently refers to the combination of the used regenerant and the slow rinse, the initial portion of the rinse. Spent brine often has a very high concentration of total solids and total dissolved solids (TDS), but contains very few suspended solids. Gradually, IX resins lose their capacity to be regenerated, and upon being replaced, become a solid waste.

The pH of the IX brine from the water-softening process depends on the nature of the resin and the regenerant. Some cation exchange resins are regenerated using concentrated sulfuric or hydrochloric acid and produce a low-pH regenerant waste. Anion exchangers are usually regenerated with a basic material or sodium chloride. Weak basic resins are typically regenerated using sodium carbonate. Strong basic resins are regenerated using sodium hydroxide and produce a high-pH regenerant. Table 23–1 shows the typical ranges of ion concentrations in the IX wastewater.

Membrane Process Wastes. Membrane processes use semipermeable membranes to remove contaminants from a feedwater. They produce a high-quality finished water and a concentrated stream of contaminants that are rejected by the membranes. Reject waters from several membrane processes (e.g., reverse osmosis, nanofiltration, ultrafiltration, microfiltration, and electrodialysis) are becoming more common as MCLs for drinking water are set at lower levels for more organic and inorganic contaminants, and as the cost of membrane processes decreases.

Membrane concentrate has very few process-added chemicals and thus reflects the character of the raw water used. Membrane processes do not produce more pollutant material or mass—they redistribute, or concentrate, those constituents present in the raw water that are rejected by the membrane.

TABLE 23–1. Typical Chemical Constituents of Ion Exchange Wastewater

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

Source: Reference 5. (Reprinted from *Water Treatment Plant Design,* 3d ed., by permission. Copyright © 1998, American Water Works Association.)

The TDS level of the concentrate can vary over a wide range, depending on the TDS level of the feedwater and the membrane system rejection and recovery. Table 23–2 provides ranges for feedwater TDS for the various membrane processes. The amount of TDS in the concentrate depends on the rejection level of the particular membrane system and the process recovery.

Membrane concentrate has the potential to be toxic to aquatic species, in the case of surface discharge, or to vegetation in the case of land applications by irrigation, because constituents in the raw water are concentrated in the membrane process. Where concentrate is potentially toxic, it can be rendered nontoxic by dilution.

Radioactive Wastes The types and quantities of radionuclides in residuals depend on the ability of the WTP to remove specific radionuclides. Table 23–3 lists drinking water treatment processes, the radioactive contaminant that they remove, and the types of residuals produced. Radon is a volatile gas that can be removed from drinking water by air stripping and GAC, neither of which produces a residual for routine disposal. Furthermore, radon has a half-life of approximately 3.5 days, and decays to essentially zero in roughly 28 days. Radon should not be found in any waste stream from a conventional water treatment process, except in the air from an air stripper. Lead-210 will be found on any material that adsorbs radon, however.

Some materials used in drinking water treatment processes, either for direct removal of a contaminant such as GAC and IX resins, or indirect removal of a contaminant such as filter sand in conventional treatment, will adsorb radionuclides. When the time arrives for these materials to be replaced, they will contain the radionuclides adsorbed but not removed from the material during the treatment process. A list of drinking water treatment process materials and the potential radionuclides contained on these materials is provided in Table 23–4.[1]

Quantities of Residuals

Coagulant Sludges

Alum Sludges. Alum sludge is voluminous because of its poor compactibility. Alum forms a gelatinous sludge that will concentrate from 0.5 to 2.0 percent (5,000 to 20,000 mg/L) in sedimentation basins. When added to water, alum ($Al_2(SO_4)_3 \cdot 14H_2O$) forms

TABLE 23–2. Concentration Factors for Different Membrane System Recoveries

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

Source: Reference 6. (Reprinted from *Membrane Concentrate Disposal,* by permission. Copyright © 1993, American Water Works Association and AWWARF.)

TABLE 23–3. Summary of Treatment Processes and the Types of Wastes Produced from the Removal of Radionuclides from Drinking Water

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

Source: Reference 7.

aluminum hydroxide ($Al(OH)_3$). For every pound (kilogram) of alum added, 0.26 pound (0.26 kg) of aluminum hydroxide is formed.

The most commonly used equation for predicting the quantity of alum sludge is:[8]

$$S = (8.34Q)(0.44\mathrm{Al} + \mathrm{SS} + A) \qquad (23\text{–}1)$$

where:

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

Generally, sludges resulting from the treatment of raw waters having high turbidities will thicken to higher concentrations than will sludges from treating low-turbidity waters.

Reported alum dosages and resultant quantities of sludge solids for several water treatment plants are shown in Table 23–5.[9–11]

Iron Sludges. Iron salt coagulants include ferric sulfate ($Fe_2(SO_4)_3$), ferrous sulfate ($FeSO_4 \cdot 7H_2O$), and ferric chloride ($FeCl_3$). The precipitate formed is ferric hydroxide ($Fe(OH)_3$). When one pound (0.45 kg) of ferric sulfate is added to water, 0.54 pound (0.25 kg) of ferric hydroxide is formed. Like alum sludge, ferric hydroxide is hydrophilic and thickens poorly. The amount of sludge formed should be determined from experience or from jar tests conducted on the proposed water supply. The Cornwell et al.'s equation for predicting the quantity of iron coagulant sludge is:[8]

$$S = (8.34)(2.9\mathrm{Fe} + \mathrm{SS} + A) \qquad (23\text{–}2)$$

TABLE 23–4. Water Treatment Process Materials Containing Radionuclides

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

Source: Reference 1.

TABLE 23–5. Coagulation Plant Sludge Quantities

		Average Raw Water Turbidity,	Chemical Dosages, mg/L		Sludge Quantities	
Plants	Raw Water Source	NTU	Alum	Other	lb/MG	kg/Mm^3
Erie County, NY						
Sturgeon Point	Lake Erie	2–25	15	—	100–175	12–21
Monroe County, NY						
Shorement Plant	Lake Ontario	1–10	18	7*	116	14
Rochester, NY	Lake Ontario	1–10	25	17†	210	25
Monroe County, NY						
Eastman Kodak Co.	Lake Ontario	1–10	24	10*	143	17
Denver, CO						
Foothills Plant	South Platte River (Strontia Springs Dam)	6.6	10	—	83	10
Birmingham, AL						
Shades Mountain	Cahaba River	19	NA	—	215	26
Washington, DC						
Patuxent River	Patuxent River	6.9	NA	—	100	12
Potomac River	Potomac River	120	NA	—	630	76

Source: References 9, 10, and 11.

NA = Not available.

*Clay.

†Clay, carbon, starch.

where:

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

Softening Sludges Chemicals used for lime softening include quicklime (CaO), hydrated lime ($Ca(OH)_2$), soda ash (Na_2CO_3), and sodium hydroxide (NaOH). Lime-softening sludges consist mostly of calcium carbonate ($CaCO_3$) and magnesium hydroxide ($Mg(OH)_2$), as controlled by the hardness removal reactions.

In addition to calcium carbonate and magnesium hydroxide, the sludge may include residues resulting from aluminum or iron coagulation of colloidal particles and unreacted lime. The quantity of sludge produced depends on whether lime/soda ash or sodium hydroxide is used as the softening chemical(s) and the total amount of hardness removed. Much more sludge is produced when lime, rather than sodium hydroxide, is used to precipitate carbonate hardness, since the calcium from the lime must also be precipitated in the chemical softening process. The theoretical amount of sludge solids produced is based on the type and quantity of hardness removed and the treatment chemicals applied, as shown in Table 23–6.[12]

If only the sludge resulting from carbonate hardness removal with lime is considered, it is possible to estimate the dry weight of sludge solids produced as:[13]

$$S = 8.336(Q)(2.0\ \mathrm{Ca} + 2.6\ \mathrm{Mg}) \tag{23–3}$$

where:

S = sludge produced, lb/day (kg/d)
Q = plant flow, mgd (m^3/s)
Ca = calcium hardness removed as $CaCO_3$, mg/L
Mg = magnesium hardness removed as $CaCO_3$, mg/L
8.336 = constant for use with English units (86.4 is the constant for use with the metric units shown)

For surface water supplies, use of a coagulant in addition to softening may significantly increase sludge quantities. The following equation used to estimate quantities should be modified to include the coagulant and raw-water suspended solids:[13]

$$S = 8.143(Q)(2.0\ \mathrm{Ca} + 2.6\ \mathrm{Mg} + 0.44\ \mathrm{Al} + 1.9\ \mathrm{Fe} + \mathrm{SS} + A) \tag{23–4}$$

TABLE 23–6. Theoretical Solids Production (lb dry solids/lb hardness removed as $CaCO_3$)

	Carbonate Hardness		Noncarbonate Hardness	
Treatment Chemicals	Calcium	Magnesium	Calcium	Magnesium
Lime and soda ash	2.0	2.6	1.0	1.6
Sodium hydroxide	1.0	0.6	1.0	0.6

Source: Reference 12. (Reprinted from *Journal AWWA,* Vol. 73, No. 11 (November 1981), by permission. Copyright © 1981, American Water Works Association.)

where:

S = sludge produced, lb/day (kg/d)
Q = plant flow, mgd (m^3/s)
Al = alum dose as 17.1 percent Al_2O_3, mg/L
Fe = iron dose as Fe, mg/L
SS = raw-water suspended solids, mg/L
A = additional chemicals such as polymer, clay, or activated carbon, mg/L
8.143 = constant for use with English units (84.4 is the constant for use with the metric units shown)

Table 23–7 shows reported softening sludge quantities for numerous water treatment plants in the United States.

A survey of softening plants in the United States by the AWWA Sludge Disposal Committee analyzed information from 84 plants. The suspended solids (SS) concentration withdrawn from the softening plant sedimentation basins varied widely, as shown in Table 23–8.[12]

TABLE 23–7. Softening Plant Sludge Quantities

	Flow, mgd	Chemical Dosages, lb/MG Lime	Chemical Dosages, lb/MG Soda Ash	Ratio, lb Solids/lb Lime Dose	Reported Sludge, lb/day
Austin, TX	45	750	—	3.8	91,200
Corpus Christi, TX	56	428	—	2.6	46,600
Dallas, TX	37	342	—	2.2	80,000
Des Moines, IA	30	1,830	197	2.8	560,000
El Paso, TX	19	825	145	3.4	—
Fort Wayne, IN	27	1,746	268	2.0	135,000
Grand Rapids, MI	6	1,350	—	2.0	18,000
Kansas City, MO	98	1,410	—	1.8	—
Louisville, KY	110	348	—	4.5	116,000
Minneapolis, MN	74	1,014	284	2.0	138,000
New Orleans, LA	120	637	143	1.6	304,000
Oklahoma City, OK	12	1,045	—	2.5	183,000
Oklahoma City, OK	21	336	—	1.4	174,000
Oklahoma City, OK	15	906	—	2.1	272,000
Omaha, NB	47	705	63	2.0	70,600
Toledo, OH	80	602	24	2.0	168,000
Topeka, KS	16	1,500	250	1.8	—
Wichita, KS	34	900	—	1.5	12,300
Pontiac, MI	10*	2,200	—	2.5	—
Miami, FL	180*	1,800	—	2.2	—
Lansing, MI	20*	2,200	—	2.3	—
Dayton, OH	96*	2,140	—	2.5	—
St. Paul, MN	120*	990	—	2.4	—

Source: References 14 and 15.

* Plant capacity.

TABLE 23–8. Softening Plant Sludge Concentrations Withdrawn from Sedimentation Basins

Suspended Solids Concentration, Percent	Percentage of Plants
<5, avg. 2.4	52
5–10	24
11–15	11
16–25	6
>25	7

Source: Reference 12. (Adapted from *Journal AWWA,* Vol. 73, No. 11 (November 1981), by permission. Copyright © 1981, American Water Works Association.)

The volume of sludge produced averaged 1.87 percent of the average water treatment plant flow, with a standard deviation of 2.1 percent. Ninety percent of the plants were between 0.4 percent and 1.5 percent, with an average of 1.2 percent.[12] An AWWA Committee Report describes softening sludge volumes as ranging from 0.3 to 5 percent of the volume of raw water treated.[9]

Presedimentation Sludge The quantity of sludge removed during presedimentation is a function of the quantity and type of solid material present in the raw-water supply. Quantities should be expected to vary widely between different plants and different times of year. Rivers carry more sediment following rainstorms than during average flow times. Pilot plant level testing is recommended to determine quantities of presedimentation sludge.

Spent-Filter Backwash Water Spent-filter backwash water represents a large volume of liquid with a relatively low solids content. Spent-filter backwash water typically represents 2 to 5 percent of the total water processed.[9]

The solids content of spent backwash can vary widely from plant to plant, depending on the raw-water quality, efficiency of preliminary treatment units, and duration of filter run and backwash cycle. The average suspended solids concentration of spent-filter backwash water typically is between 50 and 400 mg/L.

Spent-filter backwash water solids characteristically are difficult to separate from the liquid. Wash water recovery ponds sized to hold spent backwash water for 24 hours or more may recover up to 80 percent of the solids with the aid of polymers or other coagulant aids. The recovered water is reprocessed through the treatment plant in many water-short areas, or discharged to a surface water.

Slow Sand Filter Wastes

Scraping. Scraping can remove sand to depths of 0.5 to 4 inches.[2,16] Removing 1 inch of sand will generate 2 to 6 ft^3 of material per 1,000 gpd of filter design capacity, based on typical design flow rates between 45 to 150 gpd/sq ft.

Spent backwash water. Because slow sand filters are seldom subjected to any type of backwashing, the majority do not generate this residuals stream.

Filter-to-waste. Filtering-to-waste after cleaning is a highly recommended operating practice for slow sand filters, and is required by some states. High-quality filtered water is typically discharged to waste without treatment. At some locations, however, this residuals stream may be subject to provisions of an NPDES permit. Waste volumes are generally in the range of 200 to 600 gal/hr/100 ft^2 of slow sand filter area. Filtering-to-waste periods are normally of 24–hour to 48–hour duration, but vary between different sites.

Ion Exchange Wastes The total amount of wastewater (spent brine) usually ranges from 1.5 to 10 percent of the amount of water softened, depending on the raw-water hardness and the operation of the IX unit.[5,17] Figure 23–1 shows the expected wastewater volume as a function of raw-water hardness for the case where all other variables are held constant.

Concentrates from Reverse Osmosis, Nanofiltration, and Electrodialysis-Electrodialysis Reversal (ED-EDR) The quantity of membrane concentrate is directly related to the recovery, *R*, of the membrane system. Equation 23–5 can be used to calculate the quantity of concentrate that is generated by the membrane treatment system.[1]

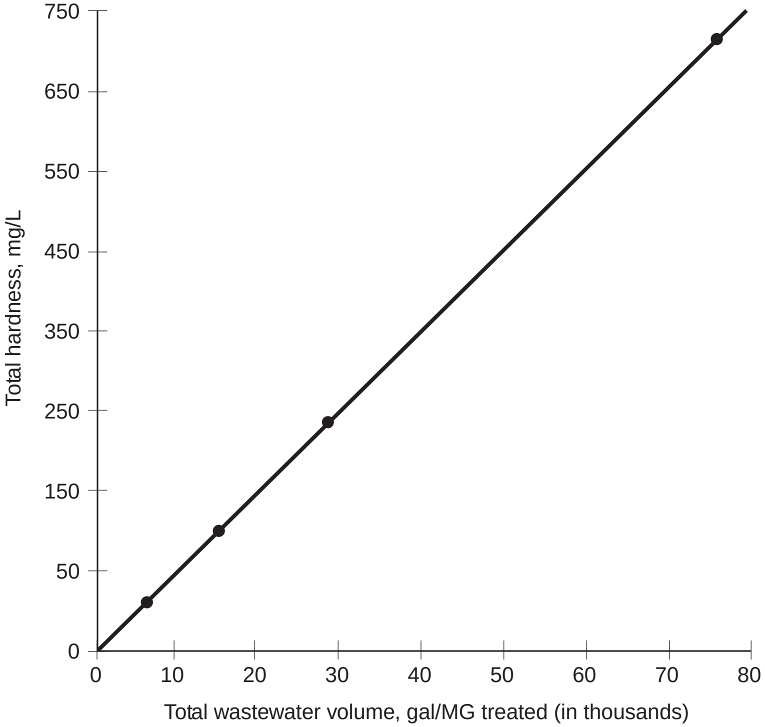


Fig. 23–1. Generation of wastewater volumes from ion exchange (*Source:* Reference 18)

$$Q_c = Q_f\,(1 - R) \qquad (23\text{–}5)$$

where:

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

Q_c can also be expressed in relation to the product volume flow:

$$Q_c = \mathrm{Q}_p(1 - R)/R \qquad (23\text{–}6)$$

where:

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

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

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

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

$$\mathrm{CF} = 1/(1 - R) \qquad (23\text{–}7)$$

where:

R = the fractional system recovery.

The CFs for different recoveries were previously shown in Table 23–2.

For example, if the feedwater has a TDS concentration of 10,000 parts per million (ppm), rejection of the membrane is assumed to be complete, and recovery is 60 percent, then the concentrate would have a TDS level of 25,000 ppm (from 10,000 multiplied by a concentration factor of 2.5). Each constituent of the TDS would similarly be present in the concentrate at 2.5 times the feedwater concentration.[1]

As was discussed in Chapter 15, “Membrane Treatment,” complete rejection of constituents through membranes is not always the case. For species that are not completely rejected, concentration still takes place, but to a lesser extent. A theoretical expression for this situation[19] is:

$$\mathrm{CF} = 1/(1 - R)^r \qquad (23\text{–}8)$$

where:

r = the fractional rejection for the species in question.

Note that for the case where $r = 1$, Equation 23–8 reduces to Equation 23–7.

Use of the 100 percent rejection assumption for reverse osmosis systems will be conservative in overestimating the concentration of the concentrate, and will result in a worst-case scenario. This approach may also be used for EDR systems. The errors should be in the range of 15 percent and under. For NF membrane systems, assumption of complete rejection of all species leads to more significant errors. Rejections of 70 and 90 percent should be used, respectively, for monovalent and multivalent ions.[1]

Characteristics of Sludges

Characteristics of water treatment plant solids affect handling, thickening, dewatering, conveying, and disposal processes that reduce the volume of the sludge to produce a material suitable for disposal or recovery.

Coagulant Sludges

Alum Sludges. In the absence of significant organic pollution of the raw-water, coagulant sludges are essentially biologically inert and have a near-neutral pH. The sludge is generally thixotropic (the plastic nature of the sludge changes with agitation) and gelatinous. However, coagulant sludges from plants treating river water with a fairly high silt content are not as gelatinous as sludge from plants obtaining raw water from clear mountain streams, lakes, or quiescent reservoirs.

Coagulant sludges such as alum sludge may be characterized at varying solids contents, as shown in Table 23–9.

Various raw-water quality parameters and sludge characteristics are shown in Table 23–10 for five water treatment plants in the United States. The Moline, Illinois, water also required a lime dose of 141 mg/L. The Washington, D.C., water was low in color, but had a high suspended solids level, and turbidities ranging as high as 160 NTU. In considering recovery of alum, the aluminum content and dissolved inorganic and organic solids are of importance.[20]

The effects of the type of coagulation mechanism on alum sludge properties are shown in Table 23–11.[21] The data show that the solids content after gravity settling and vacuum dewatering depends on the raw-water turbidity, coagulant dose, and coagulation mechanisms. Because the pH at which coagulation occurs will determine the coagulation mechanism, pH will also affect the residuals solids content.

Another factor influencing the dewaterability of alum sludges is floc density. Settled and dewatered cake solids increase as the suspended solids level in the floc increases.[1]

In a study about the effects of organic matter on floc density, when flocs had more organic matter (as indicated by the amount of TOC removed), their density declined, dewatering rates decreased,and dewatered cake solids decreased (see Fig. 23–2).

TABLE 23–9. Alum Sludge Characteristics

Solids Content, Percent	Sludge Character
0–5	Liquid
8–12	Spongy, semisolid
18–25	Soft clay
40–50	Stiff clay

TABLE 23–10. Raw-Water Quality Parameters and Alum Sludge Characteristics

	Test Locations				
Raw-Water Parameters	Indianapolis, IN	Concord, CA	Tampa, FL	Moline, IL	Washington, DC
Turbidity, NTU	45	42	0.63	71	18
Color, Pt-Co	30	7	100	26	4.2
Average alum dose, mg/L	24	41	100	43	20
Sludge Characteristics					
Initial solids concentration, %	1.7	1.7	1.6	1.7	12.1
Dissolvable inorganic solids, %	26	36	61	7	9
Nondissolvable inorganic solids, %	52	18	6	79	49
Dissolvable organic solids, %	12	18	25	2	26
Nondissolvable organic solids, %	9	28	8	13	16
Total aluminum, mg/L	2,400	2,400	3,500	295	3,750

Source: Reference 20. (Reprinted from *Journal AWWA*, Vol. 71, No. 10 (October 1979), by permission. Copyright © 1979, American Water Works Association.)

Iron Sludges. Data on water treatment plant iron sludges obtained from the St. Louis County Water Company Central Plant No. 3 and the Kingsport (Tennessee) Water Treatment Plant were reported.[23] In both facilities the sludges had been retained in the sedimentation facilities for about 3 months before they were collected for study. Both sludges were diluted with deionized water to total solids concentrations of 2 percent for all studies. Characteristics of the two iron coagulant sludges are shown in Table 23–12. The difference in the iron concentration between the sludges is attributed to the difference in suspended solids levels of the water influent to the iron coagulation basins at the two plants, and the differences in iron doses used to coagulate the solids.

Softening Sludges. Softening sludges are generally white, have no odor, and have a low biochemical oxygen demand (BOD_5) and chemical oxygen demand (COD). The chemical constituents of the sludge vary with the composition of the raw water and the chemicals added. The results of chemical analyses of dry solids from eight water-softening plants are presented in Table 23–13. The precipitates are about 85 to 95 percent calcium carbonate plus 0.4 to 7 percent magnesium oxide. These results are not entirely typical, because many water-softening plants produce residues with a high proportion of magnesium oxide.[12]

Softening sludges should be analyzed periodically for excess lime, and the calcium-to-magnesium (Ca:Mg) ratio should be calculated. Excess lime is an indicator of incomplete reaction in the softening process. If CaO or $Ca(OH)_2$ is present in the solid phase, it is an indication of poor slaking or dissolving, which results in an increase in chemical costs. If the lime does not dissolve prior to incorporation into the sludge, it might remain as $Ca(OH)_2$, if present at a concentration greater than 1,300 mg/L, thus causing poor dewaterability, and ultimately an increase in sludge quantities. Corrective action should be undertaken to eliminate these conditions.[12]

TABLE 23–11. Effect of Coagulation Mechanism on Alum Sludge Properties

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

Source: Reference 21. (Reprinted from *Journal AWWA*, Vol. 79, No. 6 (June 1987), by permission. Copyright © 1987, American Water Works Association.)

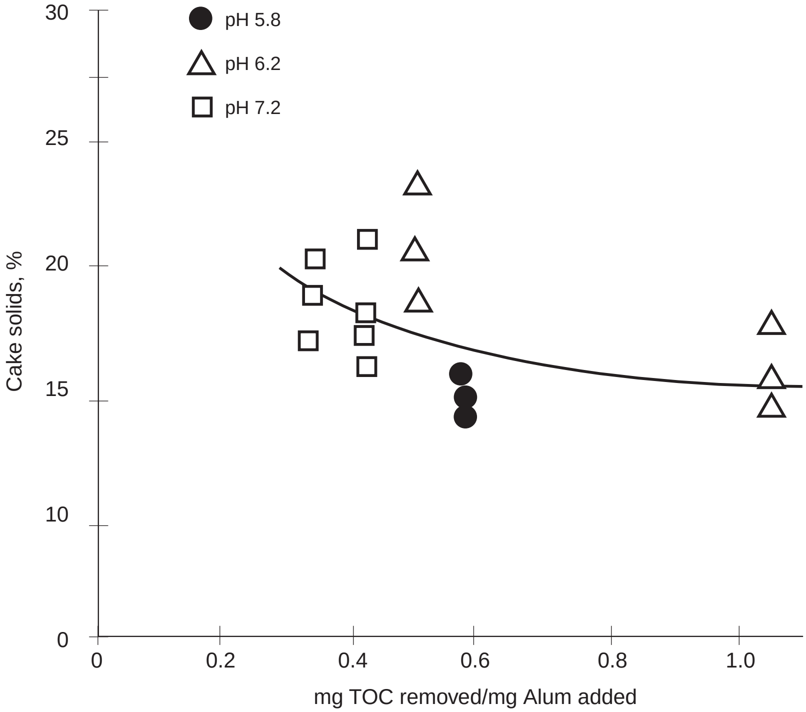


Fig. 23–2. Variations in alum sludge cake solids concentrations as a function of organic content (*Source:* Reference 22) (Reprinted from *Journal AWWA,* Vol. 81, No. 5 (May 1989), by permission. Copyright © 1989, American Water Works Association.)

TABLE 23–12. Iron Coagulant Sludge Characteristics

Sludge Characteristic	St. Louis County	Kingsport
Total solids	20,000 mg/L (2.0%)	20,000 mg/L (2.0%)
Total volatile solids	1,000 mg/L (5% of total solids)	2,800 mg/L (14% of total solids)
Total suspended solids	19,800 mg/L (1.98%)	19,900 mg/L (1.99%)
Volatile suspended solids	1,000 mg/L (5.1% of total suspended solids)	2,800 mg/L (14.1% of total suspended solids)
Iron content	930 mg/L as Fe (4.65% of total solids)	4,120 mg/L as Fe (20.6% of total solids)
pH	8.5	7.4

Source: Reference 23. (Reprinted from *Journal AWWA,* Vol. 70, No. 7 (July 1978), by permission. Copyright © 1978, American Water Works Association.)

TABLE 23–13. Lime Softening Sludge Characteristics for Eight Plants

	Boulder City, NV[24]	Miami, FL[24]	Wright Aero Corp., Cincinnati, OH	St. Paul, MN[24]	Lansing, MI[24]	Wichita, KS[25]	Vandenberg, CA[26]	Columbus, OH[27]
Silica, iron, and aluminum oxides	2.0	1.5	4.4	2.0	—	0.6–2.0	7	3.3–3.6
Calcium carbonate	87.2	93.0	88.1	85.0	80–90	89–98	85	80–85
Calcium hydroxide	—	—	—	—	—	—	1	—
Magnesium oxide	7.0	1.8	2.2	6.2	4–6	0.4–3.5	7	5.2–8.6

The Ca:Mg ratio of a sludge is an indicator of its ability to thicken and dewater. Generally a sludge with a Ca:Mg ratio less than 2 will be difficult to dewater, whereas a sludge with a Ca:Mg ratio greater than 5 will dewater relatively easily. A plot of Ca:Mg molar ratio versus the settled solids and the filter cake solids concentration is shown in Figure 23–3.[28] High-magnesium softening sludges can be considered to be nearly equivalent to mixed coagulant-softening residues because of similar poor dewaterability. Magnesium hydroxide and aluminum hydroxide sludges are difficult to dewater. Studies at Johnson County, Kansas, showed that carbon dioxide could be used to dissolve the magnesium hydroxide present in the sludge, thereby reducing, by a factor of 3, the thickening area required to produce an underflow solids content of 15 percent.[29]

Specific Resistance The dewatering properties of sludges may be characterized by the rate of dewatering (specific resistance) and the extent of dewatering (filter cake solids or settled solids concentration). Specific resistance is a measure of the rate at which a sludge can be dewatered, and its value depends on the size of particles in the filter cake. While initially developed to measure vacuum filter performance, specific resistance also gives a representation of the dewatering rate by a variety of filtration processes, including gravity settling, centrifuge, belt filter press, filter press, and sand bed dewatering.

A residue's specific resistance to filtration is usually measured using a vacuum filter device. This test is described in detail in Cornwell et al.[8] Resistance to filtration depends on the permeability or porosity of the sludge cake. Permeability is a function of particle size and particle deformation (compressibility) when pressure is applied. Specific resistance may be calculated using filtration data with the following formula:[1]

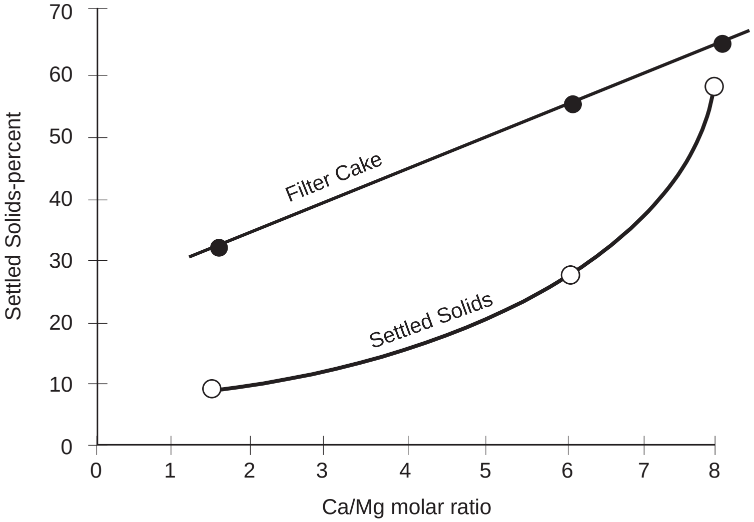


Fig. 23–3. Effect of Ca:Mg ratio on sludge solids concentration for lime sludges (*Source:* Reference 28; reprinted from *Journal AWWA,* Vol. 65, No. 6 (June 1973), by permission. Copyright © 1973, American Water Works Association.)

$$r = \frac{2PA^2b}{\mu c} \tag{23–9}$$

where:

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

Specific resistance data can be used for comparing sludges and for evaluating the effect of polymers on dewatering (see Fig. 23–4). Because the specific resistance of a sludge often depends on the pressure applied during filtration (i.e., the compressibility) and on the mixing applied during chemical addition, resistance values are used typically as an index of dewaterability rather than as well-defined sludge properties. Sludge specific resistances change during storage as a result of chemical aging and biological

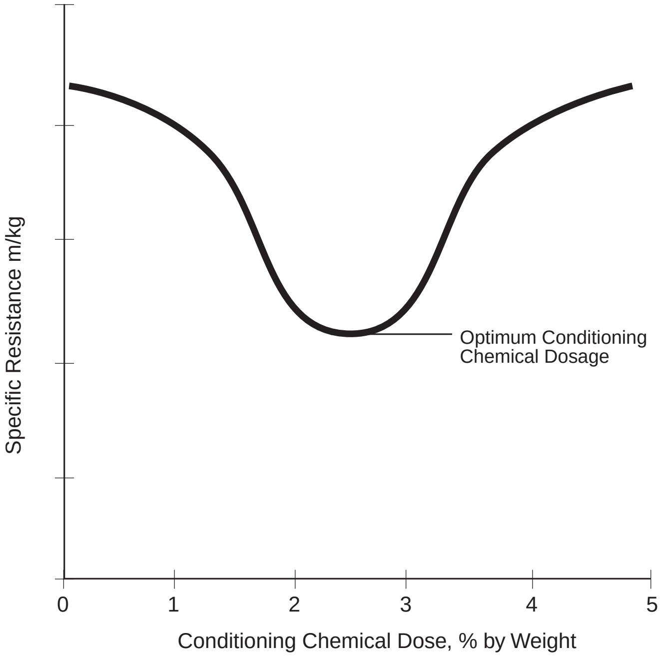


Fig. 23–4. Use of specific resistance to determine optimum chemical dosage (*Source:* Reference 8; reprinted from *Handook of Practice: Water Treatment Plant Waste Management,* by permission. Copyright © 1987, American Water Works Association and AWWARF.)

activity. Specific resistance values are useful in comparing treatment options, conditioning chemicals, or mixing conditions.

Specific resistance of sludges varies widely, as shown in Table 23–14. In general, residuals with specific resistance values of 10×10^{10} m/kg or less are considered to dewater readily, while those with a specific resistance of 100×10^{11} m/kg have poor dewaterability.[1]

Specific resistance for coagulant sludges increases as the pH rises and as the raw-water turbidity decreases. The factors in coagulation that lead to high moisture content in the sludge cakes also cause sludge to dewater slowly. Alum sludges from low turbidity raw waters have both a low rate of dewatering and a low solids content. Chemical conditioners, usually anionic polymers, may be added to sludges to decrease the specific resistance. Higher magnesium concentrations adversely affect specific resistance, as shown in Figure 23–5.[28]

Of particular importance is the solids concentration necessary to produce a sludge that can be easily handled, transported, and disposed of. As a sludge dewaters, it becomes an increasingly viscous fluid, and eventually forms a solid cake. The point at which the slurry becomes a solid is not distinct, and the extent to which a sludge must be dewatered depends on the method of handling. If a sludge is dewatered by a belt filter press or centrifuge and handled by a conveyor belt, then a lower shear stress may be sufficient to permit handling than if the sludge is drained on a drying bed, and is to be removed from the bed by a front-end loader.[30]

TABLE 23–14. Specific Resistance for Various Chemical Sludges

Missouri City	Sludge	Specific Resistance ($\times 10^{10}$ m/kg)
Jefferson City	Lime and iron	2.1
Jefferson City	Lime and iron	4.3
Kansas City	High-magnesium softening sludge	5.5
Boonville	Lime and alum	5.8
Boonville	Excess lime and alum backwash	6.0
Jefferson City	Lime and iron	6.1
Jefferson City	Lime and iron	6.8
Jefferson City	Lime and iron	7.0
Kansas City	Softening	12
Boonville	Excess lime and alum backwash	13
St. Joseph	Cationic flocculant	14
St. Louis	Lime and iron	21
Kansas City	High-magnesium softening sludge	25
St. Louis	Iron	41
Boonville	Lime and alum	53
St. Louis County	Iron backwash	77
St. Louis County	Iron	78
St. Joseph	Cationic-flocculant backwash	80
St. Louis	Iron backwash	122
St. Louis County	Iron	148
Moberly	Alum	164

Source: Reference 28. (Adapted from *Journal AWWA*, Vol. 65, No. 6 (June 1973), by permission. Copyright © 1973, American Water Works Association.)

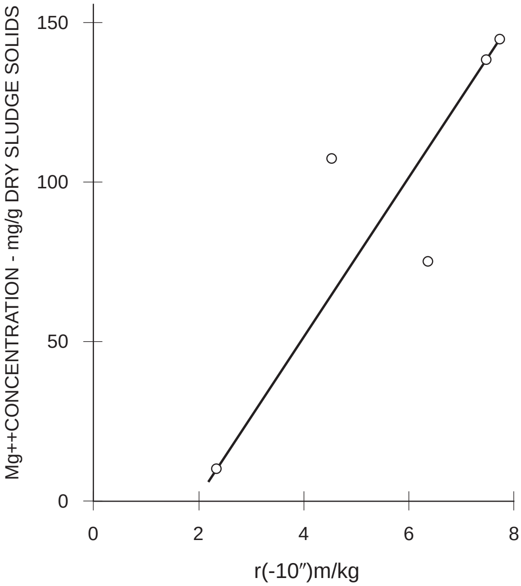


Fig. 23–5. Effect of magnesium concentration on the specific resistance of softening sludges (*Source:* Reference 28. Reprinted from *Journal AWWA,* Vol. 65, No. 6 (June 1973), by permission. Copyright © 1973, American Water Works Association.)

The solids concentration of a dewatered sludge by itself is a poor indicator of its handleability. While an alum sludge may be sufficiently dewatered for handling at 30 to 40 percent solids, a lime sludge dewatered in a lagoon to 50 percent solids may not be handleable with earthmoving equipment. Many utilities have reported that lime sludge cakes in the 50 to 65 percent moisture content range are sticky and difficult to discharge cleanly from dump trucks. The character of lime sludges at varying solids contents is generalized as shown in Table 23–15.

Shear stress is an important parameter in determining the handleability of a sludge. The undrained shear strength of various water treatment residuals, shown in Figure 23–6, varies significantly with the solids content. Figure 23–6 also shows that the sludge settled solids concentration can provide a reasonable estimate of the range of solids concentrations where a sludge makes the transition from a liquid to a handleable solid. This condition is clearly presented in Figure 23–7, where the solids concentration needed to produce a handleable sludge occurs in a shear stress range of 0.02 to 0.05

TABLE 23–15. Lime Sludge Characteristics

Solids Content, %	Sludge Character
0–10	Liquid
25–35	Viscous liquid
40–50	Semisolid, toothpaste consistency
60–70	Crumbly cake

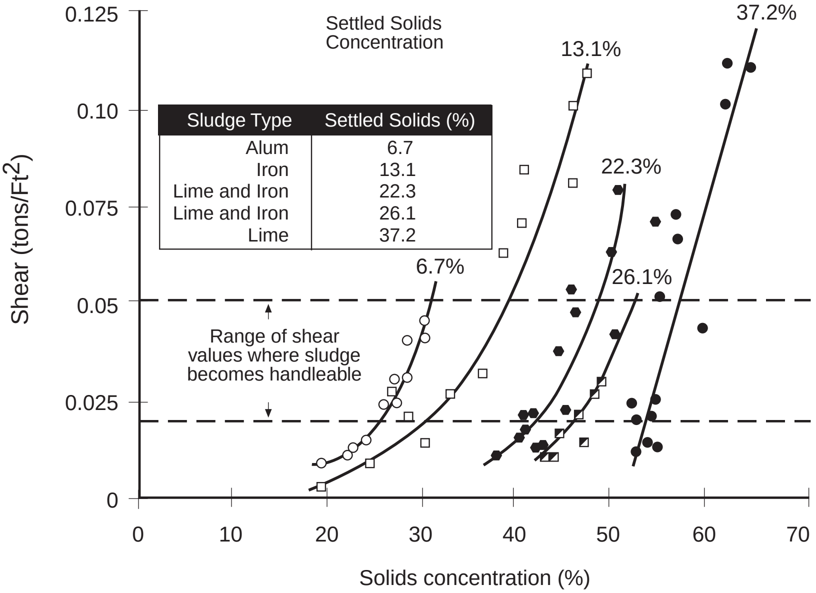


Fig. 23–6. Variation in shear strength with sludge moisture content (*Source:* Reference 28. Reprinted from *Journal AWWA,* Vol. 65, No. 6 (June 1973), by permission. Copyright © 1973, American Water Works Association.)

tons/ft^2. The data in Figures 23–6 and 23–7 also show that alum sludges generally fall in the settled solids range of 7 percent and below. Therefore, solids concentrations of 15 to 20 percent may be sufficient to produce a handleable sludge. In contrast, some softening sludges may require concentrations above 50 percent before they can be handled.

According to the theory of filtration, the resistance of sludges to filtration is a function of particle size of the flocs in the sludge cake. Although there is no precise relationship between particle size and cake resistance, measurements of particle size generally support this theory.

Data presented in Figure 23–8 for various metal hydroxide sludges show the relationship between mean floc size and specific resistance. These data were measured using a HIAC particle counter and indicate that unconditioned alum sludge has a mean floc size of 20 μ or less.[31]

Conditioning chemicals also influence particle size. There is typically an increase in particle size caused by the addition of polymer and an associated decrease in the specific resistance to filtration.

Although the average particle size is the primary factor in determining sludge filtration behavior, two other factors are important. If the particle distribution is bimodal (i.e., if there are two major sizes of particles, small and large), the sludge is susceptible to “blinding,” which is defined as the migration of fines through the cake, resulting in much lower cake permeability near the filtering surface. This occurrence has been documented for certain wastewater sludges but does not appear to be common for WTP residuals.[1]

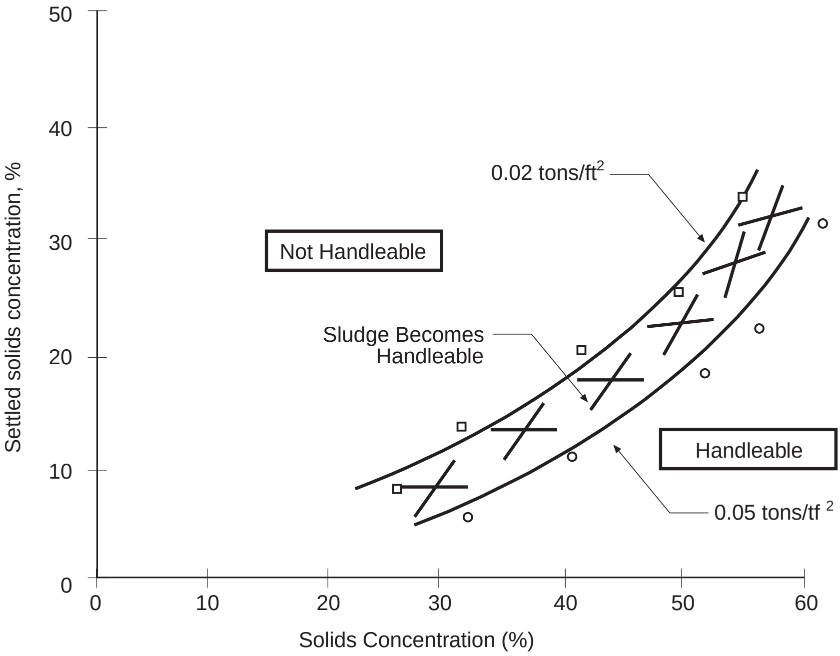


Fig. 23–7. Comparison of sludge settled solids concentration with the solids concentration where a sludge becomes ‘‘handleable’’ (Reprinted from *Handbook of Practice: Water Treatment Plant Waste Management*, by permission. Copyright © 1987, American Water Works Association and AWWARF.)

A more likely problem, especially with alum sludges, is the formation of small particles from the breakup of alum floc due to shear. Alum sludge is very sensitive to shear (Fig. 23–9), and a shear (G) value equal to 500/sec can be attained in filter cakes from the passage of water during vacuum dewatering.[32] Therefore, much of the demand for conditioning chemicals results from the shear associated with the dewatering process and not because alum sludge is composed of small particles. An important function of conditioning chemicals is to make the sludge more resistant to shear.

MINIMIZING RESIDUALS PRODUCTION

Residuals production can be minimized, either by the removal of water to reduce the volume, or by the reduction of the amount of solids present in the sludge, or by a combination of the two. If a useful by-product can be obtained from the residual material, some of the sludge-handling costs can be recovered.

One method of reducing the amount of solids is to reduce the amount of chemicals used for coagulation and/or softening. The quantity of chemical coagulant used can be reduced in some WTPs by substituting organic coagulants (polymers) for inorganic coagulants, either partially or entirely. However, Bishop has cautioned that polymers

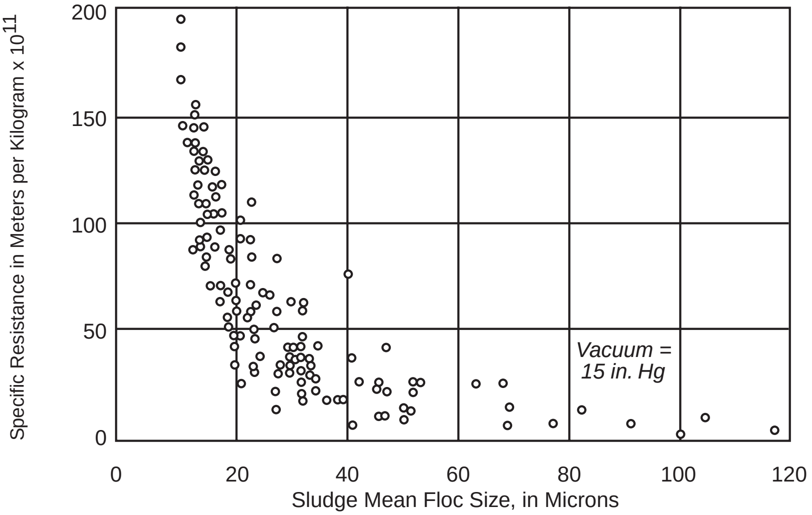


Fig. 23–8. Floc size and resistance of metal hydroxide sludges to dewatering by vacuum filtration (*Source:* Reference 31; reprinted from *Journal of Environmental Engineering,* "Vacuum Filtration of Metal Hydroxide Sludges," 1980. Copyright © American Society of Civil Engineers.)

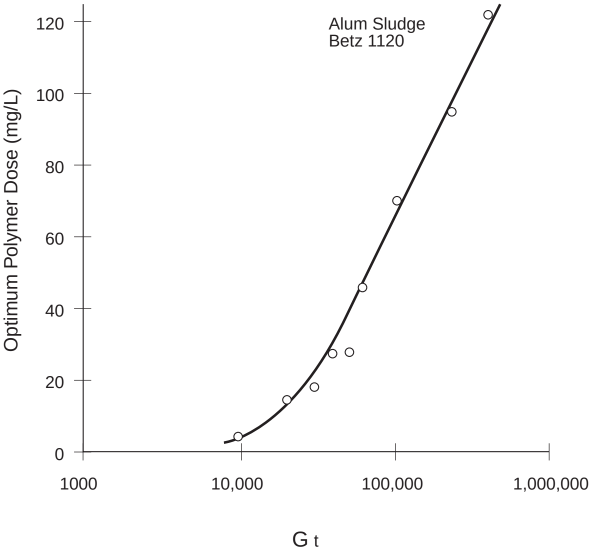


Fig. 23–9. Effect of *Gt* on optimum polymer dose for alum sludge conditioning (*Source:* Reference 32; reprinted from *Journal of Environmental Engineering,* "Mixing Intensity and Polymer Sludge Conditioning," 1984. Copyright © American Society of Civil Engineers.)

are not effective in removing color and can create problems in alum recovery processes.[33]

One method of reducing solids production in a softening WTP is to replace soda ash and some or all of the lime with sodium hydroxide. Based on the theoretical solids production data, solids loads could be reduced by up to 50 percent. Substitution of sodium hydroxide has not been widely practiced, because it generally is more expensive than the lime and soda ash it replaces. However, the high chemical costs for sodium hydroxide are at least partially offset by lower solids production and sludge disposal costs.

Split treatment is another method used to reduce softening sludge quantities when high magnesium hardness removals are required. This method normally is justified because it eliminates lime treatment of the bypassed water and minimizes recarbonation requirements.[34,35] It also minimizes sludge production because the calcium carbonate solids created by recarbonation of excess lime are eliminated.

Operation at a pH less than 10 to 10.5 will selectively remove calcium hardness, leaving magnesium in solution; waste volumes can be reduced, and the dewatering characteristics of the softening residues can be improved. However, this method results in incomplete softening that may not be acceptable, depending on the targeted level of hardness in the finished water.

Optimization of lime feed systems and mixing also can reduce solids loads by maximizing the efficiency of chemical utilization and by minimizing the amount of unreacted lime in the waste solids. Improved mixing in feeders, flash mixers, and flocculation zones reduces excess lime usage. Facilities with well-mixed solids contact clarifiers use only 2 to 3 percent excess lime.[36] Sludge recirculation from the clarifier back to the rapid mixer improves the efficiency of calcium carbonate precipitation, reduces excess lime usage, and can improve dewaterability.[36, 37]

The recycle of spent-filter backwash and clarified water (supernatant, filtrate, or centrate) from the dewatering process will reduce solids loads because this water already has been softened. These process wastewaters represent 3 to 5 percent of the total plant flow; hence, their recycle would reduce solids loads by a similar amount.[12]

Sludge volumes can be minimized prior to subsequent treatment and dewatering by controlling sludge withdrawals from the settling basins to increase the solids content. By increasing softening sludge solids content from 2 to 5 percent, the sludge volume would be reduced by 60 percent.[12] Similarly, alum sludge volumes may also be reduced by controlling sludge withdrawals, although the increase in solids content may be only from 0.5 to 0.75 percent, since alum sludges are much more dilute than calcium carbonate sludges.

Direct filtration has been used in some plants where the raw-water supply is of high quality. This process generally allows lower chemical feed rates than conventional flocculation, settling, and then filtration, and therefore produces lower quantities of residuals. However, direct filtration also removes a barrier (sedimentation) that may be necessary for removal of *Giardia* and *Cryptosporidium* from the raw water.

Another method of minimizing waste solids production is to reevaluate finished-water quality needs, such as reducing the amount of softening. For example, if a plant is removing 150 mg/L of hardness, it could reduce its waste solids load by 16 percent by removing only 125 mg/L. Not only would the sludge quantity be reduced, but chemical usage costs would decline by a similar amount.

WPT sludge quantities can also be reduced by selecting alternative water supplies that result in the least sludge production, and reducing the finished water demand through use of water conservation techniques.

THICKENING AND DEWATERING ALTERNATIVES

Thickening

Thickening, which begins with concentrating the sludge in the bottom of the clarifier, is an effective and inexpensive method and generally the first phase of reducing sludge volume and improving sludge dewatering characteristics. Thickening is most effectively accomplished as a separate operation. Thickening will improve the consistency of feed material for subsequent dewatering units, and in many cases will reduce the size of dewatering equipment. WTP residuals are most commonly concentrated using gravity thickeners, but they can also be concentrated in flotation thickeners or by gravity belt thickeners.

Gravity Thickening Gravity thickeners are generally circular and are usually concrete, although small tanks are sometimes made of steel. They are typically equipped with rake mechanisms. The floors are conically shaped with a slope of between 10 and 20 percent. The slope allows the rake mechanism to move solids to the discharge hopper. Gravity thickeners can be either batch feed or continuous flow. Residuals thickened in gravity thickeners may require conditioning.

Coagulant Sludge Thickening Results. Coagulant sludge, which is usually withdrawn from clarifiers at less than 1 percent solids, can be thickened in gravity thickeners to 2 to 3 percent solids. Aluminum and iron hydroxides may be conditioned with the aid of polymers; but the polymers will have a minimal effect on the ultimate degree of compression. Polymers will affect particle size and zone settling velocity, and will likely improve capture efficiency. Settling curves are shown in Figure 23–10 for 0, 10, and 20 mg/L doses of polymer.[38]

Typical design solids loading rates reported for alum sludge thickening are 4.0 lb/day/ft^2 when sludges are conditioned with polymers. Alum sludges that are mixed

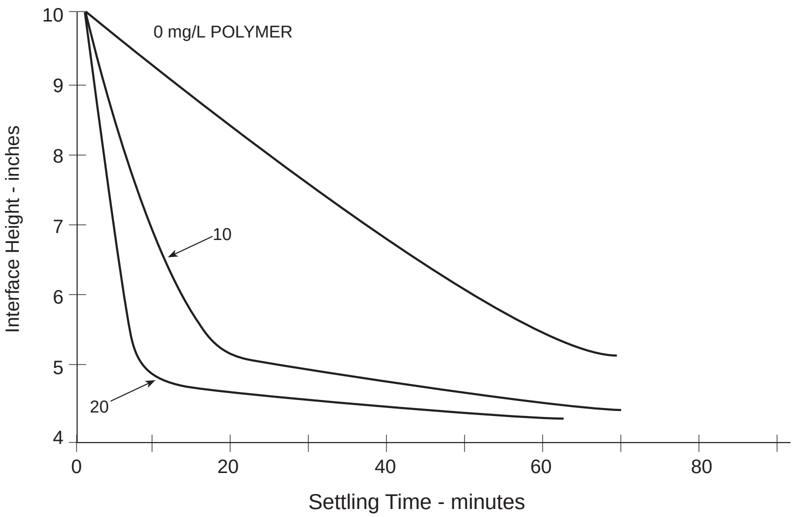


Fig. 23–10. Effects of polymer dosage on settling time (*Source:* Reference 38. Copyright © American Society of Civil Engineers.)

with clay or lime have exhibited thickened concentrations of 3 to 6 percent, and 9 percent, respectively, at higher overflow rates than sludges without clay or lime addition.[10,39]

Lime Sludge Thickening Results. Softening sludge can often be concentrated to greater than 5 percent solids in the clarifier. Solids loadings of 20 to 40 lb of solids/sq ft of thickener surface area/day (100 to 200 kg/m^2/d) are commonly practiced. Table 23–16[24] shows reported thickening results for lime softening sludges.

Thickening requirements increase if the softening sludge includes magnesium hydroxide, turbidity, or coagulants. Bench-scale thickening tests can provide a good estimate of sludge-thickening characteristics and design requirements.

Provisions to recycle the underflow to the thickener feed are sometimes provided to prevent the solids from becoming too thick, causing subsequent handling problems. In the design of a thickener, storage requirements, particularly when a dewatering device is used, must be considered, along with the thickener area required to produce a desired solids underflow.

Flotation Thickening Dissolved air flotation (DAF) thickening is a solids-handling option for residuals concentrates that consist of low-density particles. Potential benefits include lower sensitivities to changes in influent solids concentration and solids feed rate.

Several plants have had success with DAF thickening in concentrating hydroxide sludge to levels between 3 and 4 percent solids.[40,41] Loading rates for hydroxide sludges vary from 0.4 lb/ft^2/hr to 1.0 lb/ft^2/hr for facilities achieving 2 to 4 percent float solids concentration. The hydraulic loading of DAF units is reported at less than 2 gpm/ft^2.[42]

Survey of Thickening Methods A 1991 survey of water treatment plants in the United States, "Water Industry Data Base (WIDB)," was developed by the American Water Works Research Foundation (AWWARF) and the American Water Works Association (AWWA). The survey, which covered 438 utilities and 347 WTPs, is presented in Table 23–17 for thickening.[43]

Dewatering

Either nonmechanical dewatering, such as lagoons or drying beds, or mechanical dewatering, such as centrifuges, belt filter presses, or pressure filtration, can be used for

TABLE 23–16. Gravity Thickening Performance of Lime Softening Sludges

Location	Solids Input (%)	Solids Output (%)
Boca Raton, FL	1–4	28–32
Dayton, OH	2–4	15–25
Lansing, MI	12–16	20–25
Ann Arbor, MI	9	20
Miami, FL	30	40
Cincinnati, OH	5	15

Source: Reference 24. (From *Water and Waste Engineering,* "Softener Sludge Disposal—What's Best?" December 1974.)

TABLE 23–17. Survey of Thickening Methods at Water Treatment Plants in the United States

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

Source: Reference 43. (Adapted from *Proceedings of 1991 AWWA/WEF Joint Residuals Management Conference,* by permission. Copyright © 1991, American Water Works Association.)

WTP residuals. The 1991 WIDB survey also included dewatering methods, as shown in Table 23–18. The 180 dewatering lagoons are the same 180 lagoons reported for thickening in the previous table. The survey also did not differentiate between belt filter presses and filter presses in dewatering.

Lagoons One of the oldest and also most common methods currently used for handling of water treatment plant residuals is lagooning. Lagoons can be used for storage, thickening, dewatering, or drying. At some locations, lagoons have also been used for final disposal of residuals. The operating costs of this technique are low, but the land requirements are high. Because of the space requirement, lagooning may be most attractive for small, isolated plants.

Lagoons are generally built by enclosure of a land area with dikes or berms, or by excavation, with no attempt to maximize drainage with underdrains or by a sand layer. However, it is usually desirable for lagoons to have good drainage. This is best accomplished by constructing them with the bottom of the lagoon at natural ground level; an exception occurs where the existing ground surface is a tight clay soil with poor drainage initially. Preferably, sludge lagoons should not be built in excavated pits, and never with a depth below groundwater level. To allow some dewatering, residuals should have drainage into the subsoil, and the surface should be open to evaporation.[44]

TABLE 23–18. Survey of Dewatering Methods at Water Treatment Plants in the United States

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

Source: Reference 43. (Adapted from *Proceedings of 1991 AWWA/WEF Joint Residuals Management Conference,* by permission. Copyright © 1991, American Water Works Association.)

Lagoons typically are earthen basins with 4- to 12-foot (1.2- to 2.7-m) sidewater depths, covering from 0.5 to 15 acres (0.2 to 6.0 ha), which are equipped with inlet control devices and overflow structures. The best design practice is to place the inlet and outlet structures as far apart as possible. Residuals are added until the lagoon is filled with solids, and then it is removed from service until the solids have dried to the point at which they can be removed for final disposal.

If necessary, the residuals can be pumped for relatively long distances to a lagoon site. The distance between filter plants and lagoons, while adding to the cost of a pipeline, is a minor cost item in total water filtration plant costs. At York, Pennsylvania, residuals are pumped 2 miles to a lagoon, and at Appleton, Wisconsin, the sludge lagoons are 2.1 miles (4.03 km) from the softening plant. The Louisville (Kentucky) Water Company, which clarifies and softens Ohio River water, has three lagoons located 7 miles (13 km) from the major treatment plant.

Alum Sludge Lagoon Operation and Design. Alum sludges have proved difficult to dewater in lagoons to a concentration at which they can be landfilled. Neubauer reported a detailed study of a lagoon receiving alum sludges from a 32-mgd (121-ML/d) plant in Rochester, New York.[45] The 400 x 300–foot (122 × 91–m)-wide lagoon had been in operation for 3 years. Core samples indicated that the solids concentration increased from about 1.7 percent at the sludge interface to a maximum of 14 percent at the lagoon bottom. The average solids concentration of the sludge was 4.3 percent, with a majority of the sludge having less than 10 percent solids concentration. The lagoon did not produce a sludge suitable for landfill disposal without further dewatering.

Other plants have reported removing thickened alum sludge by dragline or clamshell and dumping the sludge in thin layers on the lagoon banks to air-dry; dumping the thickened sludge on land disposal areas or on roadsides; or transporting the thickened sludge to a specially prepared drying bed.[46]

In general, alum sludges do not consolidate under water, but they do dry readily when exposed to air and when drainage through the soil occurs. When lagoons are built above ground, the berms should be from 10 to 15 feet (3 to 5 m) high and far enough from property lines so that, if necessary, their top elevation can be raised. This can be done by removing dried sludge from the interior of the lagoons for use as embankment material. Lagoon berms for larger plants or those with softening should be about 12 feet (4 m) wide at the top to facilitate the use of equipment for lagoon cleaning.[46]

Two or more lagoons should be provided for alternating use, to allow between 6 months and 1 year for decanting, evaporation, and drainage. Such a drying period reduces the sludge to the consistency of the soil at the site.[44]

The State of Kansas Bureau of Water Supply has sludge storage lagoon design criteria requiring water treatment plants to have as a minimum the following storage lagoon capacity:[47]

- Two cells to be provided.
- Clarification sludge: Each cell to be designed for storage of 16 cu ft of sludge/MG (0.12 m^3/ML) of raw water treated during an 18-month period.
- Softening sludge: Each cell to be designed for storage of 85 cu ft of sludge/MG (0.64 m^3/ML) treated during an 18-month period for the first 100 mg/L of total

hardness removed; each cell designed for storage of 82 cu ft/MG (0.62 m^3/ML) treated for additional 100 mg/L of total hardness removed.

- Iron (Fe) and manganese (Mn) removal: Each cell to be designed for storage of 1.0 cu ft (0.028 m^3) of sludge per 1.0 mg/L of Fe and Mn removed during an 18-month period. If potassium permanganate ($KMnO_4$) is used, add to either Fe or Mn (or both) present in raw water 0.35 mg/L Mn for each 1.0 mg/L $KMnO_4$ added.
- Each cell to have a 2-foot (0.62-m) depth available for supernatant above the sludge storage design level.
- Each cell embankment to have a 1.5-foot (0.46-m) freeboard above the clear water zone.
- Facilities for return of water to the WTP from the clear zone to be sized between 5 and 10 percent of the plant design flow rate.
- Cells to be designed and constructed so that they can be mechanically cleaned. Consideration must be given to the movement of heavy equipment such as trucks, draglines, etc. over the top of the embankments.

Lime Sludge Lagoon Operation and Design. Lime sludges are more easily dewatered in lagoons than are alum sludges. The town of Wauseon, Ohio, has successfully used a lagoon system to dewater lime sludge since 1968.[12] The lime lagoons are still operating in 2000. The town operates two lagoons; one provides storage for current sludge discharges while the other is in the drying phase. The sludge in the second lagoon is allowed to freeze in the winter and dry through the summer. In August or September a front-end loader and dump truck remove the dewatered sludge at 40 percent solids concentration. The cleaned lagoon is then placed in operation to accept sludge while the other lagoon begins its drying phase. The sludge in the lagoons typically accumulates to a maximum depth of 30 inches (0.76 m).

Lagooned lime sludges are generally considered to be a poor fill material, and final disposal of the dried material may still present a problem.[46]

Sand Drying Beds Sand drying beds, similar to those used for dewatering wastewater sludges, have also been used successfully for many years for dewatering water treatment plant sludges. Drying beds generally consist of a shallow structure with a 6- to 9-inch (0.15- to 0.22-m) layer of sand over a 12-inch (0.304-m)-deep gravel underdrain system. Sand sizes of about 0.4 mm are typically used with a uniformity coefficient of less than 5. Excessively coarse sands result in too great a loss of solids in the drying bed filtrate. The gravel underdrain system is typically ⅛ to ¼ inch (3.2 to 6.4 mm) size graded gravel overlying drain tiles.

In especially dry climates found in the arid southwestern United States, shallow, shaped earthen basins are used that rely solely upon evaporation to separate solids from the water. These basins are more similar to lagoons than sand drying beds, with the exception that the depth of sludge application is similar to that used for sand drying beds. Sludge is applied in 1- to 3-foot (0.3- to 0.9-m) layers and allowed to dewater. With either drying bed type, sludge storage facilities may be necessary for periods when climatic conditions prevent effective dewatering. The rate at which sludges placed on sand drying beds will dewater depends upon the air temperature and humidity, wind currents, and the viscosity and specific resistance of the sludge.

The sludge dewatering process occurs by two mechanisms:

- Gravity drainage through the sludge cake and sand-filter
- Air drying from the surface of the sludge cake by evaporation

Usually both processes must be functioning for the sludge to reach a condition in which it may be removed from the drying bed for transport to a point of ultimate disposal. The design of drying beds should consider sludge characteristics affecting gravity drainage and air drying rates and the extent to which sludge may penetrate into and through the sand bed during the initial drainage phase. Excessive penetration requires frequent sand replacement and produces unacceptable direct filtrate discharge.[9] Organic polymer conditioning increases compressibility and reduces penetration.

Gravity drainage rates for water treatment sludges vary considerably with the nature of the sludge, the extent of conditioning, and the applied depth. Generally, softening sludges drain rapidly, iron-based coagulant sludges show intermediate drainage properties, and unconditioned alum sludges show relatively poor drainage characteristics. The specific resistance of the sludge correlates well with the gravity drainage rates; thus, physical or chemical conditioning significantly improves drainage characteristics of poorly draining sludges.[9]

Sand Drying Bed Operating Results. Effective organic polymer conditioning substantially decreases the time required for the gravity drainage phase of dewatering. For example, King et al. found that for well-conditioned alum sludge the time can be decreased by 50 to 70 percent.[48] For sludges with a low specific resistance, drainage can be satisfactory at applied depths of 2 to 3 feet (0.6 to 0.9 m). For poorly draining sludges, applied depths of 1 foot (0.3 m) or less are required, unless conditioning agents are used.

Air drying is normally necessary for a drained sludge on a sand drying bed to reach a state in which it can be removed. Although sludge drying rates vary through the depth of sludge, with the top layers drying most rapidly, Novak and Langford found that sufficient moisture is normally lost in air drying to render the entire cake handleable.[49] However, they also note that some sludges, especially those that have not been conditioned, may form a dry surface crust that prevents further evaporation. Novak and Langford believe that the completeness of drying throughout the sludge cake is dependent on the completeness of drainage, particularly for well-conditioned sludges.

Neubauer reports that with a 5 mph (8.1 km/h) wind, temperatures of 69 to 81°F (21 to 27°C), and humidities of 72 to 93 percent, solids concentrations of 20 percent were achieved from an alum sludge in 70 to 100 hours, with 97 percent capture of solids and a solids loading of 0.8 lb/sq ft (3.9 kg/m^2).[45] Use of effective sand sizes of 0.38, 0.50, and 0.66 mm made little difference in total drying time.

The sizing of drying beds should be based on the effective number of uses per year that may be made of each bed and the depth of sludge that can be applied to the bed.

$$A = \frac{V}{7.48(ND)} \qquad (23\text{–}10)$$

where:

A = drying bed area, sq ft (m^2)
7.48 = constant for use with English units (1,000 is the constant for use with the metric units shown)
N = number of times that beds may be used each year
D = depth of sludge to be applied, ft (m)
V = annual volume of sludge for disposal, gal (1)

The number of times that the beds may be used each year depends on the drying time and the time required to remove the solids and prepare the bed for the next application. The bed is usually considered dewatered when the sludge can be removed by earth-moving equipment (such as a front-end loader) and does not retain large quantities of sand. Alum sludges generally attain solids concentrations of 15 to 30 percent, and lime softening sludges attain 50 to 70 percent solids content. Both field tests and a detailed study of the climatic variations are required. The number of bed uses will range from 10 to 20 times per year, depending upon the climate. The usage rate may be increased if polymers are used.

The drying time required will increase with greater sludge depths. Alum sludge at Kirksville, Missouri, required 20 hours per percent solids concentration for an 8-inch (0.2-m) application and 60 hours per percent solids concentration for a 16-inch (0.41-m) application.[49] In order to obtain a dewatered cake on the bed with a thickness suitable for removal with a front-end loader, at least 16 to 24 inches (0.41 to 0.61 m) of sludge should be applied. For example, with a 1 mgd (3.785 M1/d) average treated water quantity, 2,000 lb of sludge/MG (239.6 kg/M1) treated, and 20 bed uses per year, a 2 percent concentration sludge applied at a 16-inch (0.41 m) depth will require:

$$A = \frac{(2{,}000 \text{ lb/MG})/\text{day})(365 \text{ day/yr})}{(0.02)(8.34 \text{ lb/gal}) \left(\dfrac{20 \text{ uses}}{\text{yr}}\right)\left(\dfrac{16 \text{ in.}}{12 \text{ in./ft}}\right) (7.48 \text{ gal/cu ft})} \qquad (23\text{–}11)$$

$$A = \frac{4{,}376{,}000}{(20)(1.33)(7.48)} = 22{,}000 \text{ sq ft } (2{,}044 \text{ m}^2)$$

Sand drying beds have low construction costs if land is readily available. Operating costs can be moderate to high, depending upon the difficulties encountered in sludge removal operations. A sludge that is adequately conditioned and does not penetrate deeply into the sand layer may be fairly easily removed with a front-end loader.

Freezing In climates where freezing temperatures occur frequently, the freezing and thawing of lagooned alum sludges may result in a marked improvement in the dewatering of the sludges. Freezing of waste alum solids causes the water in the gelatinous material to crystallize, and upon thawing, the water does not return to the sludge, but leaves a granular solid of coffee-ground consistency. Artificial freezing has been applied, but the electrical energy costs are prohibitive [\$136/ton (\$150/metric ton) @ \$0.08/kWh].[50]

A 1974 report prepared for the Denver, Colorado, Board of Water Commissioners described a testing program for the proposed Foothills water treatment plant.[11] Pilot ponds were used to show the effect of freezing on 6- to 14-inch (0.15- to 0.36-m) depths of alum sludge. The initial sludge placed in the pilot ponds was taken from an existing sludge lagoon at the Moffat water treatment plant, and had initial solids concentrations ranging from 5.2 to 11.7 percent total solids. The final solids concentrations of samples taken from the pilot ponds after 2 to 5 months during the winter ranged from 10 to 68 percent solids. Eight of the 12 ponds showed final solids concentrations of 26 to 68 percent solids; of these eight ponds, six had final solids concentrations of greater than 44 percent solids.

Important design and operating considerations for shallow freezing and drying ponds were described in the 1974 report based on pilot testing:[11]

- Provide means for uniform application of solids
- Provide means of complete decanting of separated water
- Maximize porosity of pond bottoms to increase drying
- Design to minimize capture of drifting snow, since snow acts as insulation and markedly reduces freezing of solids
- Provide means of snow removal and/or mixing of solids into any snow layer

The results of this testing program were promising enough that a system to allow wintertime freezing of alum sludge was designed into the 125 mgd (473-ML/d) Foothills plant. The alum sludge-handling system consists of six 200 × 400–foot (61 × 122–m) ponds with decant capability, a gravel drainage system, and spray application of sludge in thin layers. A large basin is provided that will allow storage of alum during periods when it is too cold for spray application of sludge.[51]

Tests conducted in New York State indicated that a 0.3 percent solids sludge that was placed in a lagoon in January with a depth of 30 inches (0.76 m) and subjected to natural freezing, dewatered to 35 percent solids by the next August by decanting the liquid.[52] Allowing the sludge remaining after decanting to stand for 1 week in 80°F (27°C) weather then increased the solids content to about 50 percent, suitable for handling and disposal in a landfill.

Centrifugation Centrifugal force created by rotating a liquid at high speeds is used to increase the settling rate of solids. Among the different types of applicable commercial centrifuges are the scroll-discharge, the solid-bowl decanter, the plow-discharge, and the basket-bowl.

Solid-Bowl Centrifuge. The most commonly used centrifuge for dewatering water treatment sludges is the continuously discharging, countercurrent, solid-bowl decanter centrifuge. The principles upon which this machine is based are illustrated in Figure 23–11. The sludge is introduced into the rotating bowl through a stationary feed tube at the center of rotation. The solids are thrown against the wall of the bowl, with the lighter liquid forming a concentric layer inside the solids layer. Inside the bowl is a helical screw conveyor or ‘‘scroll’’ that rotates in the same direction as the bowl but at a slightly different speed. This conveyor moves the solids deposited against the bowl toward the small-diameter end of the bowl. There they are ‘‘plowed’’ up the dewatering ‘‘beach’’ and out of the liquid layer, being discharged from the bowl through suitably located ‘‘ports.’’

Figure 23–12 illustrates the typical construction of a horizontal, solid-bowl machine. Ports in the bowl head act as overflow weirs for discharge of clarified effluent. The location of these ports with respect to the axis is adjustable, and determines the

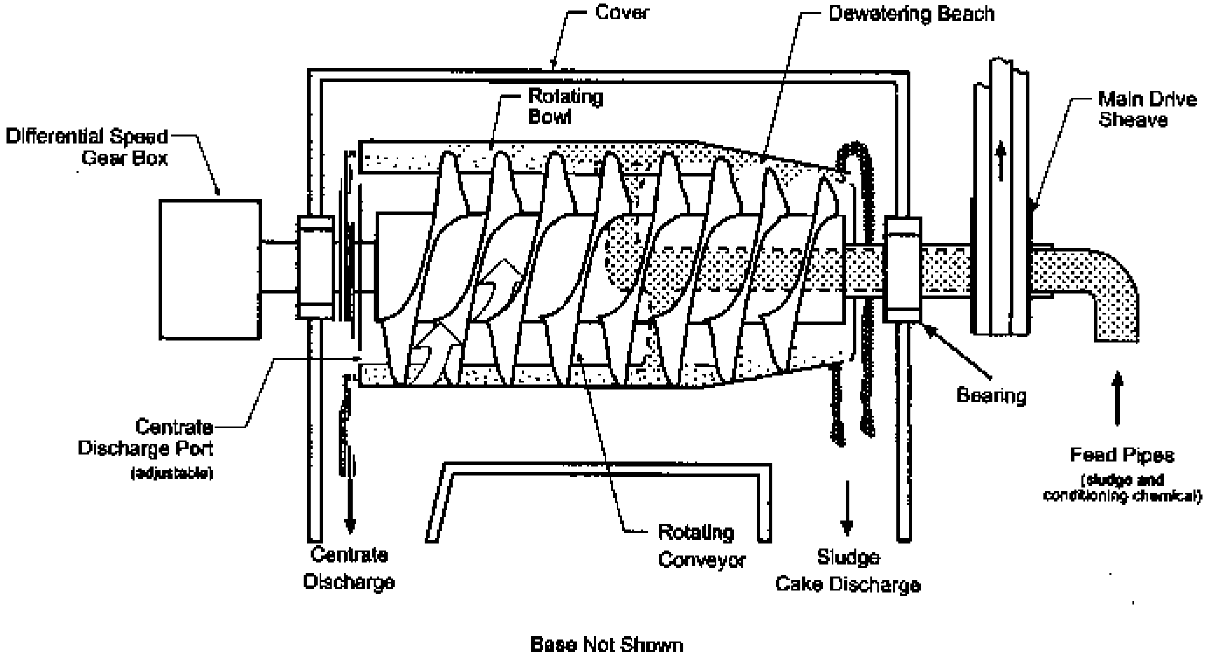


Fig. 23–11. Continuous countercurrent solid-bowl centrifuge

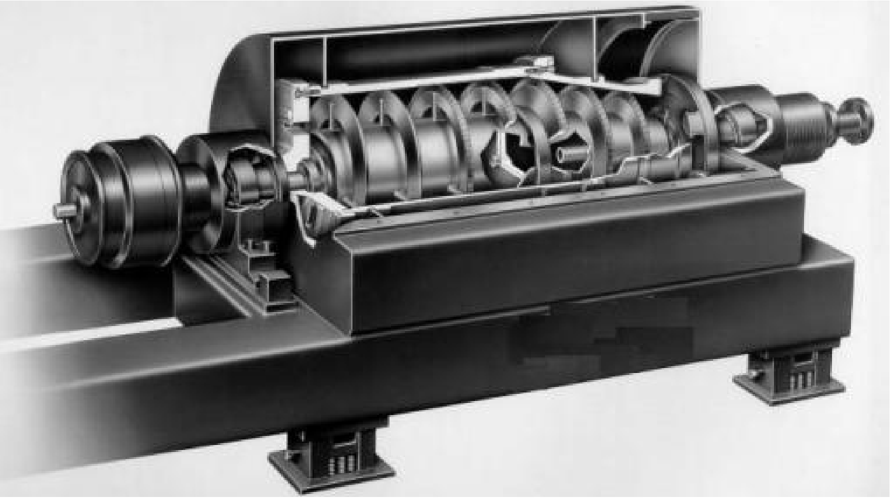


Fig. 23–12. Cross section of a horizontal, solid-bowl centrifuge (Courtesy of Alfa Laval Inc., Warminster, PA)

level of slurry or "pool depth" retained in the bowl. Usually the pool depth is set so that the liquid in the bowl submerges all but a portion of the conical drainage deck. A solid-bowl centrifuge must carry out the dual functions of clarifying the incoming sludge and conveying the solids out of the bowl. Increasing the centrifugal force and lowering the liquid depth in the bowl, for example, theoretically will improve clarification, but in many instances may act to the detriment of the machine by hindering the conveying of solids.

Most solid-bowl machines employ the countercurrent flow of liquid and solids described above and illustrated in Figure 23–11, and are appropriately referred to as "countercurrent" centrifuges. A second variation of the solid-bowl centrifuge is the concurrent model shown in Figure 23–13. In this unit, liquid sludge is introduced at the opposite end of the bowl from the dewatering beach, and sludge solids and liquid flow in the same direction. General construction is similar to the countercurrent design except that the centrate does not flow in a different direction from that of sludge solids. Instead, the centrate is withdrawn by a skimming device or return tube located near the junction of the bowl and the beach. Clarified centrate then flows into channels inside the scroll hub and returns to the feed end of the machine, where it is discharged over adjustable weir plates through discharge ports built into the bowl head.

A significant development for solid-bowl decanter centrifuges was introduced in the mid-1980s: the use of a backdrive to control the speed differential between the scroll and the bowl. The objective of the backdrive is to control the differential by allowing optimum solids residence time in the centrifuge, thereby producing the optimum cake solids content. A backdrive of some type is essential in dewatering alum sludges because of the fine particles present. The backdrive function can be accomplished with a hydraulic pump system, an eddy current brake, a DC variable-speed motor, or a Reeves-type variable-speed motor. The two most common backdrive systems are the hydraulic backdrive and the eddy current brake.

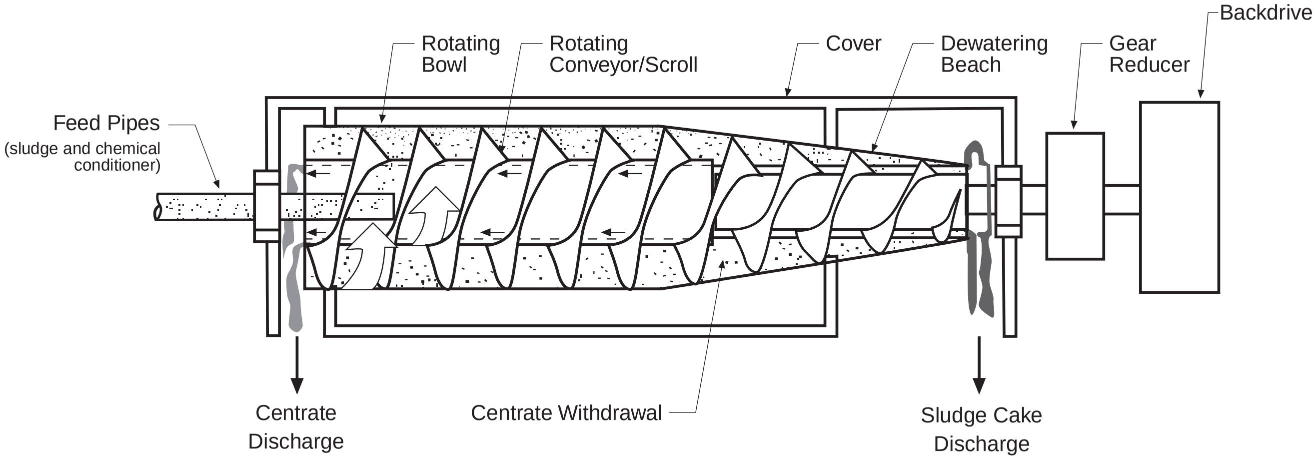


Fig. 23–13. Continuous concurrent solid-bowl centrifuge

Most centrifuge installations have the centrifuge mounted a few feet above the floor, and use a belt conveyor to move dewatered cake away. Other methods of installing a solid-bowl centrifuge are to put the centrifuge on the second floor of a two-story building and drop the dewatered cake into either trucks or a storage hopper on the first level; to mount the centrifuge about a foot off of the floor and drop the cake into a screw conveyor built into the floor; or to let the centrifuge cake drop into an open-throated progressive cavity-type pump for transfer of the cake to a truck, incinerator, or storage.

Centrifuge performance is measured by the percent solids of the sludge cake, the percent solids capture, the overall quality of the centrate, the solids loading rate, and the polymer requirement. The performance of a particular centrifuge unit will vary with the sludge feed rate and the characteristics of the feed sludge, including percent solids and sludge temperature.

Centrifuge performance is also affected by polymer type, the dosage utilized, and its point of introduction. Centrifuge performance on a particular sludge will also vary with bowl and conveyor design, bowl speed, differential speed, and pool volume.

The feed rate to the centrifuge is always a critical factor. The best performance data has been obtained at about 75 to 85 percent of the total solids or hydraulic capacity of the centrifuge, where the lowest polymer consumption is observed and the driest cake is obtained.[9]

Solid-bowl centrifuges have the largest sludge throughput capability for single units of any type of mechanical dewatering equipment. The larger centrifuges are capable of handling 300 to 700 gpm (19 to 44 1/s) per unit.

Solid-Bowl Centrifuge Performance Data. Feed solids concentrations of alum sludges dewatered by centrifuges generally range from 2 to 6 percent solids; however, 0.4 to 1.0 percent alum sludges have been successfully dewatered. Well-controlled feed concentration usually produces polymer savings and good performance.[9]

Overall raw-water characteristics affect the dewatering property of coagulant sludge. For alum sludge generated from processing raw water with a turbidity of 4 to 8 NTU, cake dryness will generally reach 15 to 16 percent, which is considered good performance for a centrifuge.[9] Alum sludge is dewatered at the City of Phoenix, Arizona, Verde Water Treatment Plant, using centrifuges located on a platform above the floor (Fig. 23–14).

This water treatment plant is operated frequently as a peaking plant or an emergency standby plant, but is not operated all the time. The centrifuge produces a cake dryness of 25 to 30 percent solids, from a feed solids concentration of 3 to 4 percent. In fact, the centrifuges could produce a cake that was so dry it was difficult to remove from loadout hoppers, so the polymer dosage was reduced to 5 to 6 lb per ton of solids, to limit cake solids to 25 percent. The alum dose is typically 12 mg/L, and normal raw-water turbidity is 10 to 15 NTU.

Alum sludges containing high raw-water turbidity, clay additives, or lime may be expected to produce higher cake solids concentrations with lower polymer requirements than pure alum sludges.

Lime softening sludge dewaters with relative ease because of its calcium carbonate content. It was reported that a thickened lime sludge could be dewatered in a solid-bowl centrifuge to a cake solids concentration of 55 percent, with 78 to 93 percent solids capture.[53] An improvement in solids capture efficiency (90 to 100 percent solids capture) was produced by increasing the sludge residence time; however, a slightly wetter cake of 45 percent suspended solids resulted.

Fig. 23–14. Solid bowl centrifuges at Verde Water Treatment Plant (Courtesy of City of Phoenix and Alfa Laval Inc., Warminster, PA)

A 1969 summary of lime sludge dewatering operational data from four softening plants using centrifugation showed that the cake solids concentrations were in the range of 55 to 65 percent suspended solids by weight, with a centrate suspended solids concentration of 500 to 10,000 mg/L.[54] These values are comparable to the summary of typical operational data from the centrifugation of lime softening sludges presented in Table 23–19.[55] Another study showed a consistent cake solids concentration of 50 to 60 percent suspended solids with a total suspended solids recovery greater than 90 percent.[27]

The apparent relationship between cake solids concentrations and centrate quality and how each was affected by centrifuge operation can be explained by a plot of

TABLE 23–19. Solid-Bowl Centrifuge Line Softening Sludge Performance Data

Parameter	Typical Range
Feed, percent solids	10–25
Cake solids concentration, percent solids	55–70
Centrate, percent solids	1.0–1.5
Solids recovery, percent	91–96
Centrifugal force, gravities	3,500–4,000
Scroll differential speed, rpm	20–28

Source: Reference 55. (From *Water and Waste Engineering,* "Sludge Handling," May 1975.)

characteristics centrifuge results (Fig. 23–15).[56,57] At very low residence times the only solids captured in the cake are those of high density, such as calcium carbonate. Solids of a lighter, more flocculent nature can pass into the centrate, yielding a high centrate solids concentration and a corresponding low solids capture. As the solids residence time in the centrifuge increases, a greater percentage of the lower density solids is incorporated into the cake (see curve B in figure), often yielding a decreased cake solids concentration. It is only after increased time of centrifugation that these solids are sufficiently compressed to yield a further increase in cake solids concentration.

When lime is added to waters with high magnesium content, large quantities of magnesium hydroxide are precipitated. If lime recalcining is practiced, the resulting magnesium oxide will appear as recycled inert material; so there will be an ever-increasing amount of solids in the system. A centrifuge can be used to classify the sludge into its calcium carbonate and noncalcium carbonate components because of differences in specific gravities. The ability of the centrifuge to provide magnesium classification is a major advantage of centrifuges over belt filter presses or pressure filters when recalcining is practiced.

Belt Press Filtration Belt filter presses employ single or double moving belts to continuously dewater sludges. All belt press filtration processes include three basic operational stages: chemical conditioning of the feed sludge; gravity drainage to a nonfluid consistency; and shear and compression dewatering of the drained sludge.

Figure 23–16 depicts a simplified schematic of a modern belt press.[58] The press uses two endless belts of synthetic fiber that pass around a system of rollers at constant speed and perform the function of conveying, draining, and compressing. Sludge feed is pumped or delivered to the free gravity drainage zone, the low-pressure zone, the higher-pressure zone, and finally to an independent high-pressure section.

Good chemical conditioning is important for successful and consistent performance of the belt filter press. A flocculant (usually an organic polymer) is added to the sludge

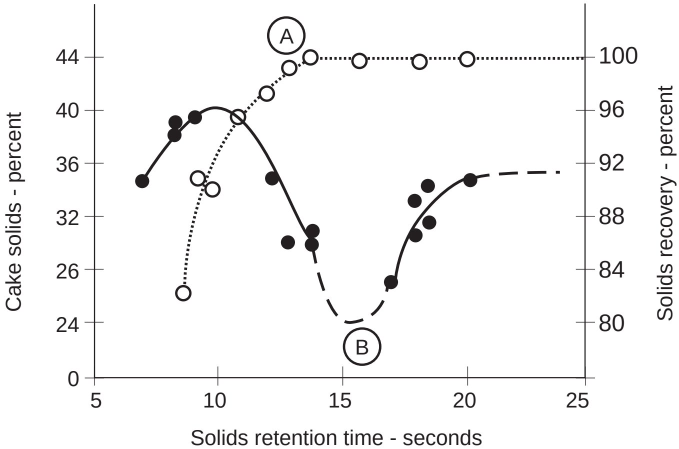


Fig. 23–15. Effect of solids retention time on centrifuge efficiency (*Source:* Reference 12; reprinted from *Journal AWWA,* Vol. 73, No. 11 (November 1981), by permission. Copyright © 1981, American Water Works Association.)

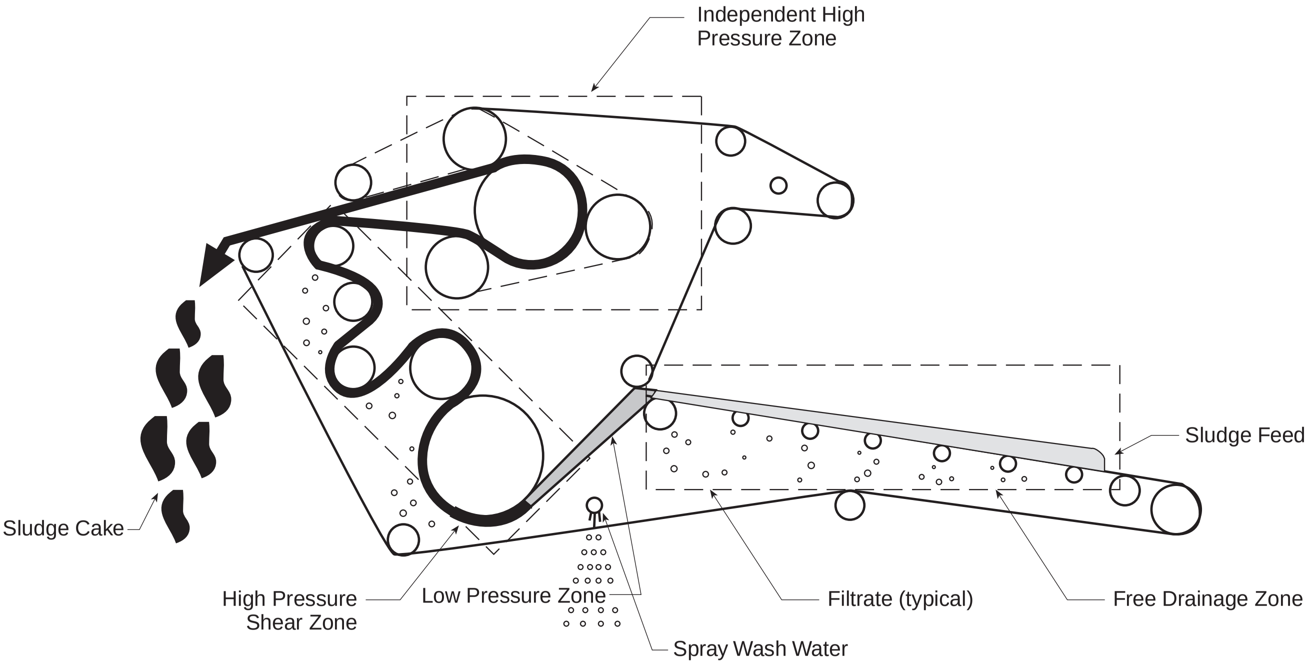


Fig. 23–16. Simplified schematic of a belt filter press

prior to its being fed to the belt press. Free water drains from the conditioned sludge in the free drainage zone of the press.

The sludge then enters a two-belt contact zone, where a second upper belt is gently set on the forming sludge cake. The belts with the captured cake between them pass through rollers of decreasing diameter. This stage subjects the sludge to continuously increasing pressures and shear forces. Pressure can vary widely by design, with the sludge in most presses moving from a low-pressure section to a medium-pressure section. Some presses include a high-pressure section that provides additional dewatering. Progressively, more and more water is expelled throughout the roller section to the end where the cake is discharged. A scraper blade is often employed for each belt at the discharge point to remove the cake from the belts. Two spray-wash belt cleaning stations using high-pressure water are generally provided to keep the belts clean.

Belt press performance is measured by the percent solids of the sludge cake, the percent solids capture, the solids and hydraulic loading rates, and the required polymer dosage. Several machine variables, including belt speed, belt tension, and belt type, influence belt press performance.

Belt speed is an important operational parameter that affects cake solids, polymer dosage, solids recovery, and hydraulic capacity. Low belt speeds result in drier sludge cakes. At a given belt speed, increased polymer dosages result in higher cake solids. With an adequate polymer dose, solids recoveries are improved by lowering belt speeds. Hydraulic capacity increases at higher belt speeds, but the solids capture is reduced.

Belt tension has an effect on cake solids, maximum solids loading, and solids capture. In general, a higher belt tension produces a drier cake but causes a lower solids capture, at a fixed flow rate and polymer dose. A drawback of using higher tension is increased belt wear. For sludges with a large quantity of alum sludge, the belt tension must be reduced to contain the sludge between the belts. The maximum tension that will not cause sludge losses from the sides of the belts should be used.

Belt type is an important factor in determining overall performance. Most belts are woven of polyester filaments, and they are available with weaves of varying coarseness and strength. A belt with one of the coarser and stronger weaves may require high polymer dosages, to obtain adequate solids capture.

Most manufacturers' belt presses can be equipped with sensing devices that may be set to automatically shut off the sludge feed flow in case of underconditioning. Both underconditioned and overconditioned sludges can blind the filter media. In addition, overconditioned sludge tends to drain so rapidly that solids cannot be evenly distributed across the belt. Vanes and distribution weirs included in the gravity drainage section help alleviate the problem of distribution of overconditioned sludge across the belt, and inclusion of a sludge blending tank before the belt press can also reduce this problem.

A belt press installation should be designed for daily washdown by hosing around the press; therefore, drainage and safe walking area around the press are important.

The required flow rate for belt washing water is usually 50 to 100 percent of the flow rate of sludge to the machine, and the pressure is typically 100 psi (690 kPa) or more. Some belt presses recirculate washwater from the filtrate collection system, but usually potable water is used. The combined flow of washwater and filtrate typically contains between 500 to 2,000 mg/L of suspended solids. To minimize the effect on finished drinking water quality by recycle streams, the filtrate should be either discharged to a sanitary sewer or treated before recycle.

Belt Filter Press Performance Data. At the water treatment plant at Somerset, Kentucky, belt filter presses are reported to produce an average cake solids content of 34 percent from a feed solids content of 3.1 to 5.3 percent.[59] Raw water is obtained from a lake and has average turbidities ranging from 16 to 31 NTU. Coagulation is accomplished with 14 to 15 mg/L of alum, 1.0 mg/L of cationic polymer, and 6 to 6.5 mg/L of lime. Polymer dosage is 3.2 lb/ton (1.6 g/kg) of solids.

A demonstration test of a belt filter press was conducted at the Western Pennsylvania Water Company plant in New Castle, Pennsylvania.[60] The raw river water received coagulation treatment with alum, polymer, and lime (when required to raise pH). The feed solids content averaged 3.8 percent, and the sludge cake produced averaged 32 percent solids. Polymer requirements were 2.1 lb/ton (1.1 g/kg) of dry solids. Solids recovery averaged about 90 percent.

A demonstration test conducted on alum sludge at the San Jacinto Water Purification Plant in Houston, Texas, illustrates the difficulty in dewatering some alum sludges.[61] With polymer dosages of 7.3 to 10.1 lb/ton (3.7 to 5.1 g/kg) of solids, cake solids contents of only 13.5 to 15.6 percent were obtained from a feed solids concentration of 2.5 percent. With the addition of 12.5 percent diatomaceous earth, cake solids increased to 19.4 percent solids. With the addition of 23 percent lime for conditioning, cake solids contents of 23 percent were obtained.

Dewatering of an alum sludge at the 15 mgd Cedar Park, Texas Water Treatment Plant occurs on a belt filter press. The raw-water turbidity typically ranges from 1 to 5 NTU, with spikes to 20 NTU. The coagulant dosage is typically 20 mg/L of alum with polymer. The belt press dewaters the sludge from 0.5 to 1.0 percent feed solids to about 16 percent cake solids, with a polymer dose of nearly $20 per ton of dry solids.

The belt press at Cedar Park is contained in a prefabricated metal building and is shown in Figure 23–17. The dewatered alum sludge cake coming off the unit is shown in Figure 23–18.

Typical performance data of belt filter presses on lime softening sludge at three water plants have been summarized.[62] Feed sludge concentrations are about 20 to 25 percent solids, and cake solids concentrations are 60 to 70 percent. The solids recoveries are 90 to 95 percent, and polymer requirements are typically 2 to 3 lb/ton (1 to 1.5 kg/g) of dry solids.

Pressure Filtration The filter press offers advantages for difficult-to-dewater residuals because it is a batch operation where the residual can be kept under pressure on the filter for extended periods of time. In addition, filtrate liquors produced are low in suspended solids content. Since lime softening sludges dewater readily to a dry cake with other devices, and since centrifuges have the ability to classify the magnesium hydroxide from the calcium carbonate, the filter press has not found significant application for softening sludges.

A filter press consists of a number of plates or trays that are held rigidly in a frame to ensure alignment and are pressed together either electromechanically or hydraulically, between a fixed and moving end. Figure 23–19 illustrates a typical filter press layout.

The two types of filter presses that are commonly available to dewater residuals from water treatment plants are the fixed-volume recessed plate filter press and the variable-volume recessed plate filter press, also referred to as the diaphragm filter press. The recessed plate filter press is also called the chamber filter press.

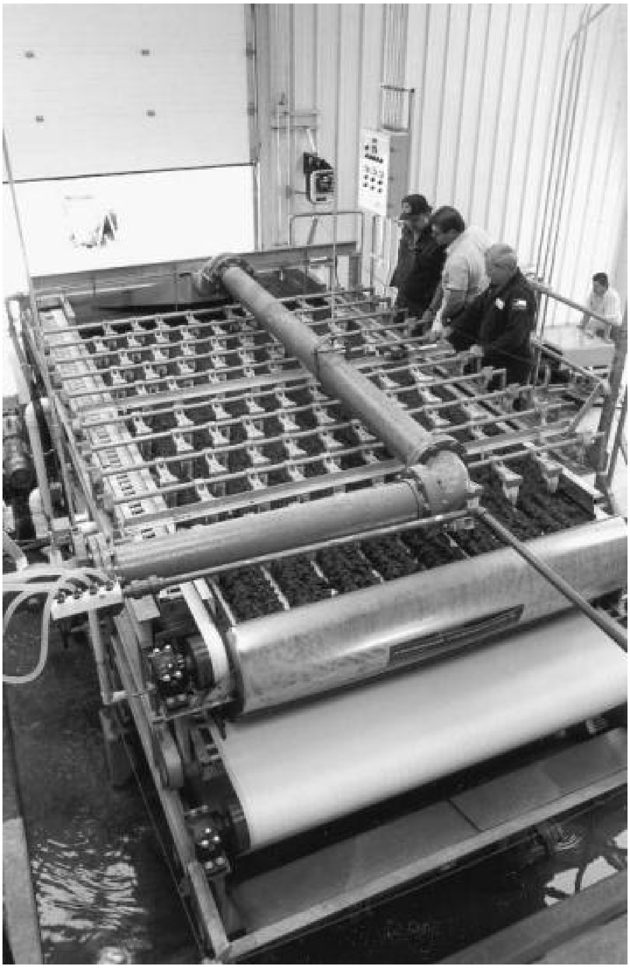


Fig. 23–17. Belt press installation at Cedar Park, Texas, water-treatment plant (Courtesy of Ashbrook-Simon-Hartley)

In the fixed-volume recessed plate filter press, liquid sludge is pumped by high-pressure pumps into a volume between two filter plates. On the face of each individual plate a filter cloth is mounted. As a result of the high pressure that the sludge is under, a substantial portion of the water in the feed sludge passes through the filter cloth and drains from the press. Sludge solids and the remaining water eventually fill the void volume between the filter cloths, so that continued pumping of solids to the press is no longer productive. At this point, pumping is stopped, and the press is opened to release the dewatered sludge cake prior to initiation of a new ‘‘pressing cycle.’’

In the fixed-volume recessed plate press, a cloth filter medium is used on both sides of the filtering volume. As shown in Figure 23–20, sludge is pumped into the volume between the cloth sides, and water is expelled through the medium. Sludge pumping

Fig. 23–18. Alum sludge cake discharging from belt filter press at Cedar Park, Texas (Courtesy of Ashbrook-Simon-Hartley)

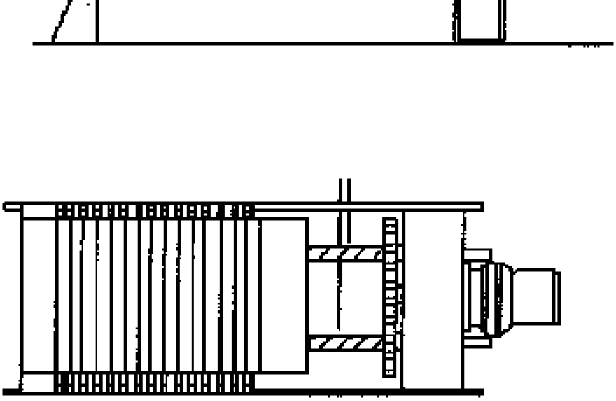


Fig. 23–19. Typical filter press

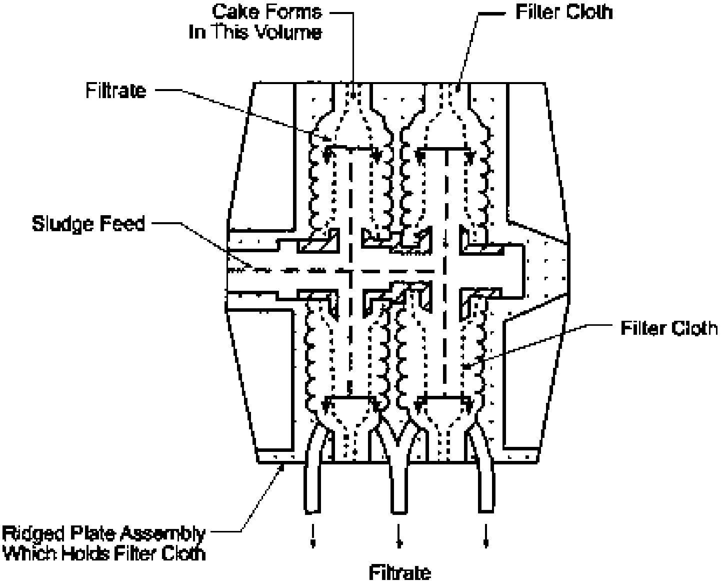


Fig. 23–20. Cross-section of a fixed volume recessed plate filter press assembly

is at relatively high pressures, up to 225 psi (1,550 kPa), and the driving force for movement of water through the cloth is this high pressure. Low-pressure recessed plate presses are also available, operating at about 100 psi (690 kPa). When little or no additional filtrate is being produced, the pumping is stopped, the press is opened, and the sludge cake falls from the press.

In a variable-volume recessed plate, or diaphragm, filter press, sludge is pumped into the press at a low pressure until the volume of the press has been filled with a loosely compacted cake; then sludge pumping is stopped and the diaphragm is inflated for a preset time. For the diaphragm press, although most of the water removal occurs when sludge is being pumped into the press, a significant quantity of water is also removed after the diaphragm is inflated. The filter cloth is washed periodically, by permanent spray nozzles. Figure 23–21 shows the basic configuration of one cell of a diaphragm press and the stages of operation.

The diaphragm press has several advantages over the fixed-volume recessed plate press. First, a drier cake with a relatively uniform moisture content is produced. Second, the diaphragm press has an overall shorter cycle time and therefore a higher production throughput. Two other advantages of the diaphragm press are the lower operation and maintenance requirements for the sludge feed pumps, and the ability to dewater a marginally conditioned sludge to a high solids content. Another advantage of the diaphragm press is that it does not require a precoat.

The principal disadvantage of the diaphragm press is that its initial cost can be two to three times the cost of a fixed-volume recessed plate press. Also, the capacity of the largest diaphragm filter press is generally less than that of the largest fixed-volume recessed plate filter press.

Control of filter presses may be manual, semiautomatic, or fully automatic. In spite of automation, operator attention is often needed during the dump cycle to ensure

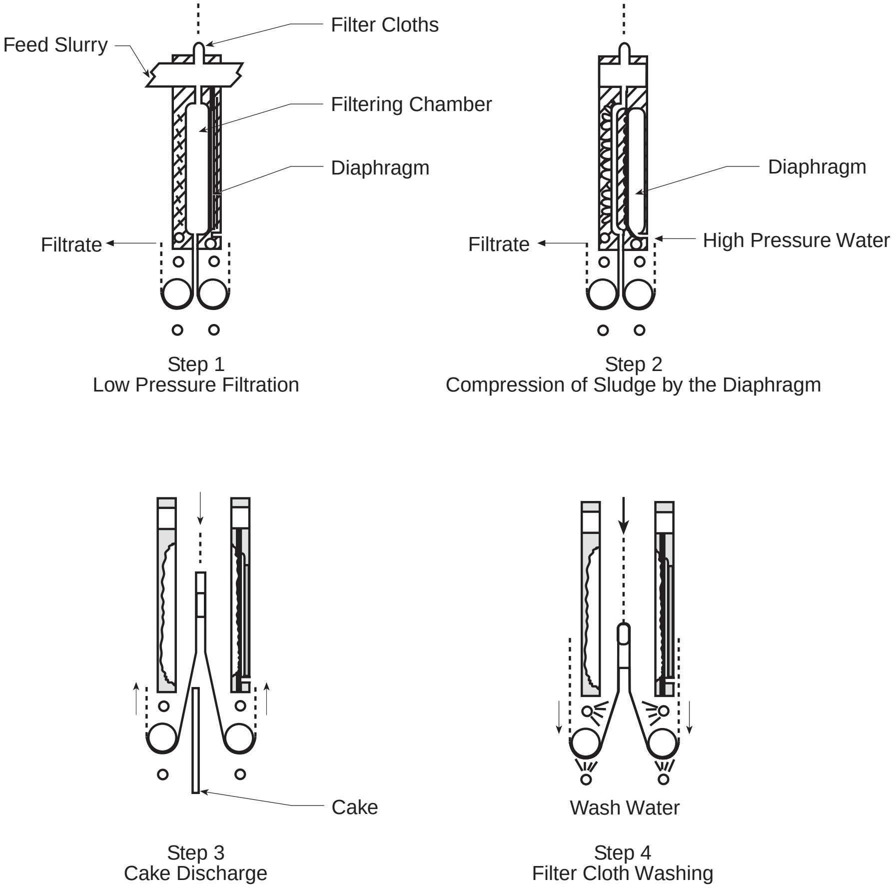


Fig. 23–21. Operational cycle for a variable volume diaphragm filter press

complete separation of the solids from the media of the filter press. The assistance with cake removal typically consists of an operator with a ‘‘rowboat’’ oar scraping, helping pry the cake off the filter cloth (see Fig. 23–22). Process yields can typically be increased 10 to 30 percent by carefully controlling the optimum cycle times with a microcontroller.

Filter presses are normally installed well above floor level so that the cakes can drop out into trailers positioned underneath the presses. Alternatively, conveyors can be installed under the presses to transport the cakes to a dewatered cake storage area.

In order for pressure filtration to be economical, alum sludges must be conditioned to achieve a lower resistance to filtration. Lime is an effective conditioner, and fly ash can be equally effective.[9] The choice of conditioning agent is an economic one that should be investigated for each application.

Lime is typically added to alum sludge until the pH of the slurry is raised to about 11. There must be sufficient residence time of the lime-treated sludge to allow complete reaction of the lime with the sludge. A minimum time requirement is typically 30

Fig. 23–22. Typical method for dewatered cake removal from filter press

minutes. Insufficient residence time may produce premature plating-out of lime on the filter media and the interior of pipelines.[9]

Two-stage conditioning systems were demonstrated to be somewhat more economical than one-stage lime addition. In these systems, only a portion of the lime is added to the incoming untreated sludge, followed by a small polymer addition. When the mixture is then allowed to age, an appreciable supernatant is formed, which is decanted and returned to the head of the plant. After sufficient residence time, the remainder of the lime is added, and the sludge is then ready for filtration. The total lime addition is less than that used in a single-phase system.[9] Each case should be tested to determine the effectiveness of the type of conditioning system.

The capacity of a pressure filter is determined by the number of filter plates, the size of filter plates, and the cake thickness provided for in the filter plate chamber. Filter cake thickness is critical in the design of a pressure filter; cake thicknesses are standardized to 0.98, 1.18, and 1.57 inches (25, 30, and 40 mm) thickness.[9] Filtration tests determine the most economical cake thickness for any given application.

The basic components for a filter press system are shown in Figure 23–23 and include:

- Storage and mixing tanks for chemical reagents
- A storage and conditioning tank, to provide a consistent feed to the filter presses for the duration of the filtration cycle. In this tank, chemical reagents are introduced to improve the filterability characteristics of the sludge. Means for agitation are provided to prevent segregation of particles and also to prevent size degradation and breakdown in the flocculated feed.

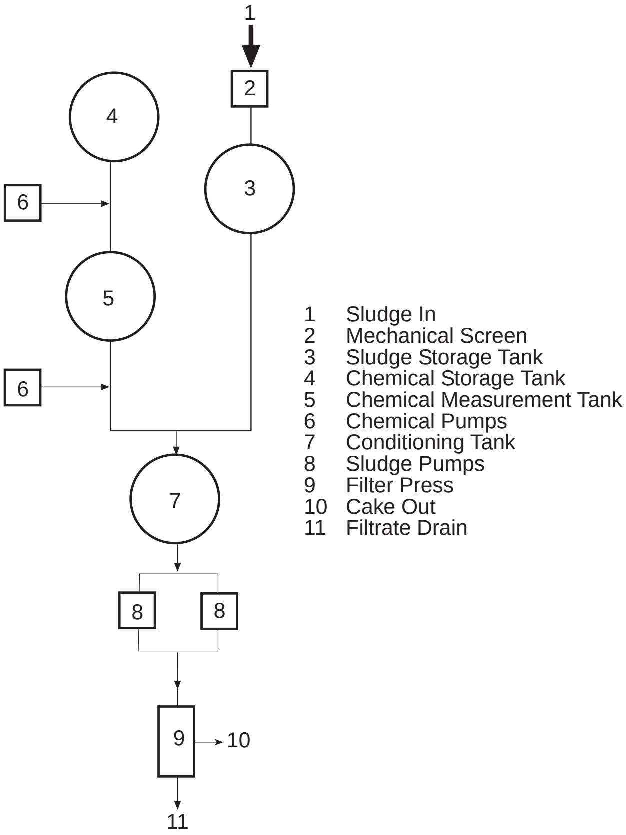


Fig. 23–23. Typical filter press system components

- Feed pumps
- Filter presses with ancillaries, including filter media and filtrate collection trays or launders. The selection of the correct filter media is an important factor in pressure filtration.
- Interconnecting pipework, valves, and air vessels for the sludge feed pump system and filtrate disposal. Air vessels serve to even out pressure variations caused by the action of the feed pumps.
- Filter cloth washing machines and drying racks
- Means for collecting and delivering filter cake to a disposal point in a form that is acceptable for any further processing required

Filter media are almost exclusively of the monofilament type; that is, woven from a single filament rather than a yarn that is twisted together of many fibers. The monofilament media do not blind from swelling of the yarn, a problem with multifilament media in the past.

Any filter medium eventually blinds and must be washed periodically. If sludge conditions cause calcium carbonate to precipitate on the filter cloth, an acid wash system—either an acid soaking system or an acid recirculating system—is used. The latter is preferred because it not only reacts with the calcium carbonate deposits but also offers a surface scrubbing action to wash out loose material. In a soaking system, the filter is filled with acid and allowed to stand overnight. One drawback of this system is the formation of gas pockets in the upper portion of the filter chambers that prevent the acid solution from reaching the upper portion of the filter plates.[9]

Disposal of filtrate produced during pressure filtration typically is a problem because of the chemical characteristics of the material. The conditioned sludge has a pH of about 11.5, which causes a significant fraction of insoluble aluminum hydroxide to be converted to soluble aluminate. In addition, precoat material can contribute potentially significant concentrations of trace metals to the filtrate.

Possible filtrate disposal methods include direct discharge to a waterway, discharge to a sanitary sewer system, and treatment of the filtrate prior to disposal or recycle. Direct discharge to a waterway requires compliance with effluent standards, while discharge to a sanitary sewer requires compliance with local sewer use ordinances. One filtrate disposal method involves neutralization of the filtrate, sedimentation, and returning the filtrate to the head of the sludge treatment process.

Filter press performance is measured by the solids content in the feed sludge, required chemical conditioning dosages, cake solids content, total cycle time, solids capture, and yield, in lb/sq ft/hr (kg/sq m/hr). These performance parameters are all interrelated; for example, as the feed solids content increases, the required chemical dosages and total cycle time usually decrease, while the filter yield, or throughput, usually increases. As the chemical conditioning dosage is increased up to the optimum level, the cake solids content, solids capture, and yield all increase, while the cycle time decreases.

Filter press testing at several Monroe County, New York, water treatment plants and at the Erie County Water Authority's Sturgeon Point plant was conducted on alum sludges.[10] Filter cake concentrations of 40 to 50 percent solids were obtained in laboratory experiments and in a trailer-mounted pilot plant. Filtrate quality was suitable for its inclusion back into the plant influent as raw water at the treatment plants. Lime requirements amounted to approximately 25 percent of the waste solids on a dry weight basis, and the precoat was approximately 2 percent of the waste solids. The pressure filtration cycle time ranged from 90 to 120 minutes.

Operating data from the alum sludge treatment system in Atlanta, Georgia, indicate that filter presses are capable of producing a lime-conditioned filter cake with an average of 46 percent solids. The filtrate contains less than 10 mg/L of suspended solids.[39]

The high costs, the batch operation of filter presses, the complexity of the process, and the large chemical conditioning doses are the major reasons for infrequent use of filter presses for WTP residuals.

Lime Sludge Pelletization Sludge pelletization occurs during the suspended-bed cold-softening water treatment process, used primarily in the southeastern United States. The softening process seems to work best on high-calcium, warm-temperature groundwater.

The detention time in a suspended-bed softening reactor is approximately 8 to 10 minutes. The conical reactor vessel is constructed with sides approximately 80 degrees from horizontal (see Fig. 23–24), and the vessel is charged initially with a 0.20 to 0.25 mm effective size silica catalyst. The high-velocity, upward spiral flow of the raw water suspends the granular catalyst, which is essential for the efficient removal of

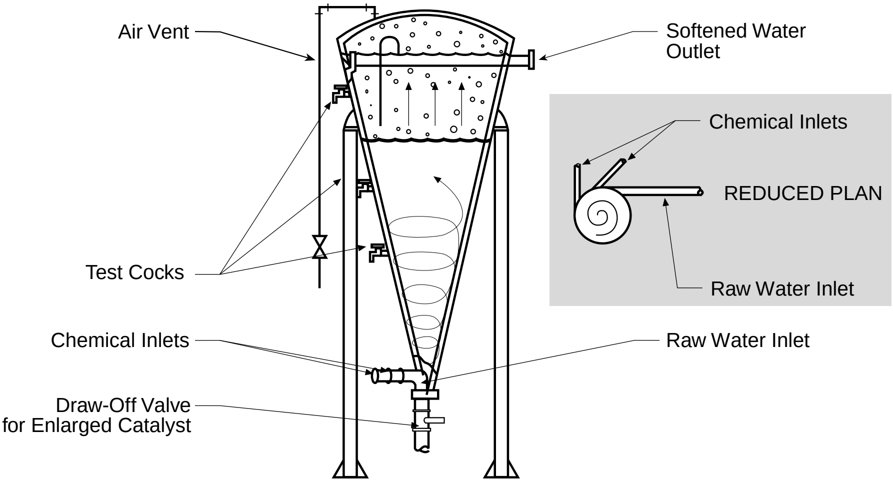


Fig. 23–24. Schematic diagram of a reactor for lime sludge pelletization (Courtesy of USFilter)

hardness. Upward velocity is limited to about 3 ft/min (0.015 m/s) at the top of the cone to prevent carryover of catalyst particles.[12]

Lime is injected into the reactor while the raw-water flow is gradually increased from a low initial rate to design capacity. The lime reacts with calcium bicarbonate and carbon dioxide to form calcium carbonate, which precipitates on the suspended particles.

Claims have been made that the size of the calcium carbonate-coated particles can reach 1.6 mm diameter; however, operating experience has shown that maximum sizes are in the 0.7 to 1.0 mm range.[12]

Theoretically, reactors should be capable of continuous operation. This requires a fine balance between the blowdown of sludge pellets and the addition of new, granular catalyst to maintain a constant-volume bed. In practice, the balance is difficult to accomplish, so the reactors are generally operated in a batch mode.

Treated water turbidity is used as a measure of treatment effectiveness. When high turbidities are experienced, often after about 40 days of operation, the reactor run is terminated. At the end of the run, the contents of the reactor, water, and sludge pellets are discharged into a storage and drainage facility. After drainage, the pellets can be treated as solids.

The pelletized sludge is approximately 60 percent solids by weight as it leaves the reactor. The entrained water can be readily drained, with the resulting product being 90 percent solids by weight. If the weight of the entrained catalyst is accounted for, each 1.0 cu ft (0.028 m^3) of drained sludge contains approximately 105 pounds (48 kg) of calcium carbonate.[12] A comparison of the volumes of conventional and pelletized sludge shows that the volume of pelletized sludge is 10 to 20 times less than the volume of undewatered conventional sludge.

The limitations on this approach are: magnesium content should be less than 85 mg/L as $CaCO_3$; turbidity should be less than 10; and, in cold climates, the reactors must be enclosed in heated structures. Excessive magnesium forms magnesium hydroxide, which does not plate out on the nuclei and will quickly clog downstream filters. Also, upflow rates of about 10 gpm/sq ft (24 m/h) are too high to permit removal of suspended solids, which will also pass on to downstream filters. However, the resulting economies in sludge handling may be great enough for the designer to consider adding the reactor ahead of a conventional clarifier.

Japanese Pellet Flocculation Process. The pellet flocculation process has been used successfully to treat alum sludge in Japan.[33] It involves multistage gravity thickening of the sludge, chemical treatment using sodium silicate and a polymer, and a dewatering process using a large horizontal-rotating drum called a dehydrum. This process is reported to be capable of producing a sludge with 25 to 30 percent solids without the need for mechanical vacuum or pressing equipment. It is also reported that the process appears to be most appropriate for treating large volumes of sludge.[63]

RECOVERY OF COAGULANTS

The recycling and recovery of coagulants has long seemed promising for recovering a resource (i.e., a coagulant) and for minimizing waste by extracting aluminum or iron coagulants from the waste stream. Recalcination of spent lime is a proven technology at many locations.

Alum Recovery

Alum Recovery by Acidification Aluminum recovery from sludges produced in potable water coagulation plants has been studied by researchers over the last 30 to 40 years. The traditional scheme for alum recovery consists of thickening sludge from settling basins and filter backwashing, reducing the pH of the sludge by acid addition, and separation of the dissolved aluminum (in the form of aluminum sulfate) by decanting it from the residual solids. The recovery of alum by acidification with sulfuric acid is shown in the following equation.[64]

$$2Al(OH)_3 + 3H_2SO_4 \Leftrightarrow Al_2(SO_4)_3 + 6H_2O \qquad (23\text{–}12)$$

As indicated by the above equation, about 1.9 g of sulfuric acid is required for each gram of sludge treated. This assumes, generally, that aluminum recovery in excess of 80 percent can be expected at a pH at or below 2.5.[65] Critical design and operation factors include extraction pH and acid contact time. Extraction pH is typically in the range of 1.8 to 3.0. Acid contact time of 10 to 20 minutes seems reasonable based on full-scale operations data and laboratory testing.[66] Another critical component is the amount of metal coming from the raw water compared with the coagulant. For example, the aluminum from raw-water solids is more difficult to dissolve than that associated with an alum coagulant.

The impact of coagulant reuse on treatment plant operation and the resulting finished water quality must be carefully considered. Even though the acidification process dissolves the coagulant, concentrations of coagulant impurities and raw-water contaminants, such as iron, manganese, chromium, and other metals, may also be dissolved and recycled to the head of the plant. While these contaminants may again be removed from the water supply during coagulation, settling, and filtration, the net result is a concentration of contaminants. Especially important is the recycling of organic material, which may increase disinfection by-product formation potential.

For some time, the acidification of alum sludge was considered beneficial prior to pressure filtration at many water treatment plants in Japan. Fifteen Japanese water treatment plants, five in the Tokyo area, used an acid alum recovery process of pressure filtration. Concern over the possible recycling and concentrating of heavy metals has halted this procedure.

Recovery by Liquid-Liquid Extraction An alternative method of alum recovery was investigated at Michigan State University.[64,67–69] Basically, the method uses organic solvents for the extraction, by liquid ion exchange, and of high-purity concentrated alum from sludge.

In general, solvent extraction (also called liquid-liquid extraction) is the separation of the constituents of a liquid solution by contact with an immiscible liquid. The operation is dependent on the differential solubilities of the individual species in the two liquid phases. For water treatment applications, the solvent from which the extraction is made is water, in which the other solvent must be both insoluble and immiscible.

A special type of liquid–liquid extraction is termed *liquid ion exchange* because of its similarity to resin ion exchange. In liquid ion exchange, a small quantity of an organic-soluble chemical called the *extractant* is dissolved in a second organic liquid called the *diluent*. The mixture is often referred to as the *organic phase* or the *solvent*. The diluent may be a material such as kerosene or a similar hydrocarbon. During the extraction operation, the extractant reacts chemically with the desired metal in the aqueous phase, forming a metal-extractant complex that is soluble in the diluent.[67]

Based upon testing of the liquid ion exchange process for alum recovery on continuous-flow laboratory equipment, the following conclusions were reached:[69]

- The process will recover more than 90 percent of the aluminum from influent aluminum hydroxide sludge.
- The recovered alum is of the same quality and concentration as commercial liquid alum, or higher.
- The extractant is highly selective for aluminum over potential heavy metal contaminants (copper, cadmium, manganese, zinc, iron, and chromium).
- It may be feasible to operate a system at zero cost or with a net operating cost credit at plants using a high alum feed rate; that is, the operation and maintenance costs and the amortization of the capital cost may be offset by the value of the recovered alum. At plants using lower feed rates, the value of the alum recovered will help offset the annual operating costs.

A demonstration plant for the testing of the modified liquid ion exchange alum recovery process was built at Tampa, Florida.[70] The City of Tampa's Hillsborough River water treatment plant is a 65-mgd (246-ML/d) conventional coagulation plant that treats a highly colored raw water. At the time of this project, the Tampa WTP utilized alum coagulation with sodium silicate addition as a settling aid (polymer later replaced the sodium silicate). The process was able to recover more than 90 percent of the aluminum in the sludge. The recovered alum had the same characteristics as commercial alum. The sludge reduction accompanying aluminum recovery was as predicted by direct sludge acidification. The economic key was to obtain an aluminum concentration greater than 1,200 mg/L in the sludge.

Current Use of Alum Recovery Few full-scale alum coagulant recovery facilities are currently active.[66] It is essential that pilot or laboratory scale operations be performed when considering the feasibility of implementing alum coagulant recovery.

Iron Coagulant Recovery

The recovery of iron coagulants involves acidification of ferric hydroxide and a recovery technique very similar to that described for the acidic alum recovery process. The pH of the iron sludge is lowered by acid addition to a range where the solubility of ferric iron is significantly increased, and the iron is released back into solution. The principle of this method is shown in the simplified relationship:

$$2Fe(OH)_3 + 3H_2SO_4 \rightarrow Fe_2(SO_4)_3 + 6H_2O \tag{23–13}$$

The pH must be reduced to 1.5 to 2.0 to attain 60 to 70 percent recoveries of iron.[23]

This method can be improved by adding a reducing agent to the sludge before acid addition, to convert the form of the iron from the precipitated ferric ion (Fe^{+3}) to the more soluble ferrous ion (Fe^{+2}). Sodium sulfide (Na_2S) has been found to be an effective reducing agent. Recoveries of as much as 60 percent of the sludge iron were achieved at a pH of 3.0 when the reducing agent was used. The recovery processes also had a marked effect on the settling characteristics of the residual sludges. For both residual sludge samples the solids settled to about 20 percent of the original volume in a period of less than 30 minutes, and the sludge solids concentrations increased from the original 2 percent total solids before recovery, to 7 to 9 percent total solids in the settled sludge. In addition, the weight of sludge solids requiring ultimate disposal was reduced.

The presence of a sulfide residual in the recovered iron solution adversely affects the coagulation performance of recovered iron. The problem can be remedied by adding potassium permanganate to the recovered solution before reuse. Sulfate buildup may be a problem with repeated iron recycling. Another alternative for removing sulfide is to strip it as H_2S by aerating the recovered iron solution at the low recovery pH. It is necessary to control the H_2S release in the off-gas. Iron coagulant recovery is not commonly practiced.[1]

Recalcination of Lime Softening Sludge

Lime recovery by recalcination has been widely used for years. Recalcination has the potential to recover substantially more lime than used in the original lime–soda water-softening process, while at the same time producing carbon dioxide for use in recarbonation, and greatly decreasing the volume of sludge requiring ultimate disposal. As with other recovery processes, careful consideration must be given to the expected quality of the recovered product and to including processes to remove impurities. Centrifuges can be used to separate some impurities.

Quicklime (CaO) can be produced from lime softening residues, after purification and dewatering of the calcium carbonate, through use of a drying process. The basic step in the recalcination process is the burning of softening sludges at a temperature of 1,850°F (1,010°C). The reactions during recalcination are:

$$CaCO_3 \rightarrow CaO + CO_2 \qquad (23\text{–}14)$$

$$Mg(OH)_2 \rightarrow MgO + H_2O \qquad (23\text{–}15)$$

Since the lime–soda process produces approximately two parts of lime for every part of calcium carbonate applied, it is theoretically possible to recover up to twice as much lime as originally used. In practice, the yield is somewhat reduced by side reactions involving impurities and inefficiencies associated with preliminary dewatering, as well as capture of the recalcined lime.[12]

Among the factors that lead to decreased lime recovery, the effects of magnesium, silica, and typical surface water suspended solids must be considered. Of these materials, the magnesium content of the raw water is the most important. Softening sludges can be recarbonated to redissolve magnesium selectively. Magnesium oxide recovered from recalcination will not slake, but will pass through the softening process as the oxide. It has been noted that one part of silica will combine with six parts of calcium oxide to form an inert complex that will not slake.[27] A significant loss of lime can occur if high concentrations of silica are evident.

Calcium carbonate is a high-density solid, whereas the hydroxides of metals such as magnesium, iron, and aluminum are more light and flocculent by nature. These three metal hydroxides are undesirable contaminants in a lime recalcination process.

Lime recalcining plants generally consist of the following components:[12, 15]

- Sludge thickening from an initial 3 to 10 percent solids, to 18 to 30 percent
- Recarbonation using stack gases, 15 to 27 percent CO_2, to redissolve magnesium hydroxide selectively
- Dewatering, usually by centrifuges, to 45 to 65 percent dry solids by weight
- Flash dryers and cyclone separators using hot off-gases from the recalciner
- Recalcining furnace

Available furnace types include the rotary kiln, the flash calciner, the fluidized-bed calciner, and the multiple-hearth calciner. Features of some existing lime recalcining plants are summarized in Table 23–20.[15]

Recalcining yields a calcium oxide product of approximately 90 to 93 percent purity at a fuel rate of about 8.5 to 12 bil Btu/ton (8,967 to 12,660 bil J/907 kg).[24] High energy use apparently has limited use of the recalcination process.

ULTIMATE SOLIDS DISPOSAL

Surveys of Disposal Methods

Water treatment plant sludges historically have been discharged either directly or indirectly into a surface water. In 1953, 92 percent of 1,600 coagulation and softening plants surveyed disposed of their sludges in streams or lakes.[71] A 1969 survey of 80 primarily large plants showed that the disposal of water treatment plant wastes in surface waters had decreased to 39 percent for softening plants and 49 percent for

Image Not Available

coagulation plants.[71] Spent-filter backwash waters were discharged to streams and lakes by 83 percent of the plants responding in 1953 and 49 percent in 1969.

Results on water treatment plant waste disposal methods from a 1979 survey of 75 alum coagulation plants and a 1981 survey of 100 softening plants are shown in Table 23–21.[12,66] The percentage of softening plants discharging sludge to rivers or lakes had decreased to 13 percent by 1981, while 20 percent of alum coagulation plants still practiced this method of sludge disposal in 1979. Substantial numbers of water treatment plants continue to discharge sludge and spent-filter backwash water to surface waters; however, it is obvious that this practice is steadily being restricted.

Disposal Options

There are eight basic sludge disposal options that can be used by water treatment plants:

1. Discharge to waterway
2. Discharge to sanitary sewers
3. Codisposal with wastewater biosolids
4. Lagooning with ultimate disposal of the residue
5. Mechanical dewatering with landfilling of residue
6. Coagulant recovery
7. Land application
8. Use for building or fill materials

Discharge to Waterway The oldest disposal method is to discharge sludges to the nearest available waterway—stream, pond, lake, or ocean—but its use is decreasing under the pressure of state regulatory agencies and federal laws.

TABLE 23–21. Methods for Disposal of Water Treatment Plant Waste

	Percent of Plants Using Indicated Disposal Method	
	Softening* Sludge	Coagulation† Sludge
Sludge lagoon	34‡	43
Sanitary sewer	8	27
River of lake	13	20
Recalcination	5	—
Direct land application	5	—
Other	—	10

*Reference 12.

†Reference 66.

‡Fifty-six percent of softening plants surveyed had sludge lagoons, 60 percent of which were considered ‘‘permanent lagoons’’; thus, 34 percent of plants used sludge lagoons for disposal.

The following conclusions about discharge of WTP residuals to waterways came from the 1996 USEPA *Handbook on Management of Water Treatment Residuals:*[1]

> Because of the potential toxicity to aquatic organisms, utilities that discharge WTP residuals to soft waters (with hardness less than 50 mg $CaCO_3/L$) should consider a different residuals disposal method. In addition, receiving waters with a pH less than 6 should be avoided. Discharge to these waters could result in the increased solubility of metals and increased toxic effects.

Discharge to Sanitary Sewers In 1968 8.3 percent of WTPs discharged solids to sanitary sewers.[71] A 1979 survey of alum users found that 27 percent of these plants discharged sludge to the sanitary sewer,[66] so it is increasing, but a 1981 survey of softening plants found that only 8 percent of these plants discharged softening sludges to sanitary sewers.[12] Softening plants produce greater solids quantities, which are a larger load on the wastewater treatment plant process—a fact that may account for their more limited use of this method.

This technique of sludge disposal transfers the solids-handling problem from the water treatment plant to the wastewater treatment plant (WWTP). However, inclusion of the necessary capabilities in the solids-handling facilities of the WWTP may result in an overall cost savings, by consolidating the equipment and reducing the number of personnel required for total solids handling. Many wastewater utilities are concerned that the water treatment plant solids will adversely affect their treatment processes. However, these same chemicals are used extensively in waste treatment to remove phosphorus, and no adverse effects result. A number of factors must be evaluated if this approach receives serious consideration for a given application.

A major consideration is the ability of the wastewater collection system and WWTP to accept the increased hydraulic and solids load imposed by the addition of the WTP wastes. The direct discharge of spent-filter backwash water into the sewer system, for example, could cause a hydraulic overload of the collection system, or a hydraulic surge large enough to cause the WWTP clarifier performance to deteriorate. Hydraulic surge storage at the WTP, with gradual release, may be needed if the volume of water plant waste is large in proportion to the wastewater flows. Release during low wastewater flow periods (midnight to 6:00 A.M.) may be desirable. Another aspect to consider is that the sewer receiving the WTP sludges must be of adequate capacity and should provide velocities adequate to prevent deposition of the WTP sludge in the sewer. Studies at Detroit report that a velocity of 2.5 ft/sec (0.76 m/s) is adequate to prevent settling of the sludges in the sewer.[46]

The bulk of the solids from the WTP sludges will be removed in the WWTP primary clarifier. The solids-handling system at the WWTP must be capable of handling the additional solids load. It will be a rare and fortunate circumstance for an existing WWTP solids handling system to be able to handle the unplanned addition of WTP sludges if the water plant and wastewater plant are of comparable size. Dallas, Texas, in an attempt to dispose of waste lime sludge and to use it effectively for wastewater treatment, found that it greatly improved the removal efficiencies of both BOD and suspended solids.[72]

Culp and Wilson in 1979 studied the effect of adding alum sludge to an activated sludge wastewater treatment facility and reported no significant benefit or detriment to the treatment process or the anaerobic digester.[73] There was an increase in sludge-handling quantities in proportion to the increased water treatment solids.

WTPs that discharge their residuals to a sanitary sewer or a WWTP are usually regulated by local industrial user permits.

Codisposal Lime sludge may be of some value in disposing of another waste because high-pH liquid or semisolid waste can neutralize acidic waste. At one water treatment plant in Europe, lime softening is practiced in conjunction with hydrogen-ion-exchange softening; the lime softening waste is used to neutralize the acidic rinse water from regeneration of the resin. Lime sludge could be used in a codisposal plan for many reasons:[3]

- Elevation of pH
- Bulking agent
- Neutralization of acid wastes to bring them within National Pollutant Discharge Elimination System (NPDES) permit limits
- Assistance in pretreatment of industrial wastes
- Incineration to produce high alkaline ash

Lagoons A detailed description of lagoon dewatering appears earlier in this chapter. Mechanical dewatering is expensive, especially for small plants. The most viable alternative for small plants may be lagooning. Larger plants where large tracts of inexpensive land can be obtained have also used lagooning. However, in many instances storage of dilute or concentrated water treatment plant solids in lagoons is considered the ultimate disposal. In effect, this is a postponement of the inevitable ultimate disposal requirement.

Landfill Options Landfilling typically falls into two categories: codisposal and monofilling. This discussion assumes that the WTP residuals to be managed have not been classified as hazardous wastes. When a WTP's residuals do exhibit hazardous characteristics, they must be handled as Resource Conservation and Recovery Act (RCRA) Subtitle C wastes. Typically, WTP residuals will be classified as nonhazardous solid waste, and be covered under the requirements of the RCRA Subtitle D.[1] Subtitle D regulations (40 CFR Parts 257 and 258) apply to landfills that receive only WTP residuals (monofills), as well as landfills that accept solid waste other than household waste (e.g., industrial waste).

Municipal solid waste landfills (MSWLFs) are covered by the criteria of Part 258. If a utility disposes of its drinking water residuals in a monofill, then Part 258 criteria do not apply. If, however, the WTP residuals are codisposed of with municipal solid waste, including household waste, the requirements established for MSWLFs apply.[1]

Many state landfill requirements are more restrictive than federal criteria. Major landfill considerations include:

- Siting

  Restrictions on siting a landfill in or near wetlands, airports, floodplains, fault areas, seismic impact zones, and unstable areas are typically established.
- Monofill Method

  The major sludge monofilling methods are trench filling and area filling. Sludge solids content, sludge stability, site hydrogeology (location of ground water and bedrock), ground slope, and land availability help determine the monofilling method that is selected.

- Environmental Considerations

  The potential for leaching of metal components and other contaminants in the residuals into groundwater supplies must be addressed.

- Lining

  Technology-based standards require a composite liner system be installed that consists of an upper geomembrane liner and a lower compacted-soil liner to control the escape of leachate into groundwater.

- Land Application

  The options for land application of WTP residuals include agricultural use, silvicultural application, application for reclamation to disturbed and marginal lands, and disposal to dedicated lands.[74,75] The application of WTP residuals may beneficially modify soil properties while recycling residual components. The possible disadvantages of land application of WTP residuals include an increase in the concentration of metals in the soil and possibly in groundwater; adsorption of soil phosphorus by water treatment residuals, decreasing the productivity of the soil; excessive application of nitrogen, resulting in the transport of nitrate to ground water; and possible effects caused by the application of poorly crystallized solids of aluminum.[76] A simplified planning procedure for land application is presented in Figure 23–25.[1]

Use as Building or Fill Material Alum sludge has been suggested for use as a plasticizer in the ceramics industry, as a constituent of refractory bricks, and as a road-stabilizing agent. In Atlanta, dewatered alum sludge from the Hemphill alum sludge treatment facility is transported to a residential building site, where it is used as fill.[1] Sludge cake is spread and compacted by a bulldozer to depths as great as 6 feet (1.8 m). No problems have been reported with driving loaded trucks over the compacted sludge cake.

Liquid Residual Alternatives

There are several potential sources of liquid waste streams from a conventional water treatment plant.

Major Waste Streams

- Spent-filter backwash water
- Filter-to-waste water

Other Waste Streams

- Presedimentation basin blowdown sludge
- Sludge thickener supernatant
- Sludge lagoon overflow
- Filtrate from plate and frame filter presses
- Pressate from belt filter presses
- Centrate from centrifuges
- Leachate from sand drying beds

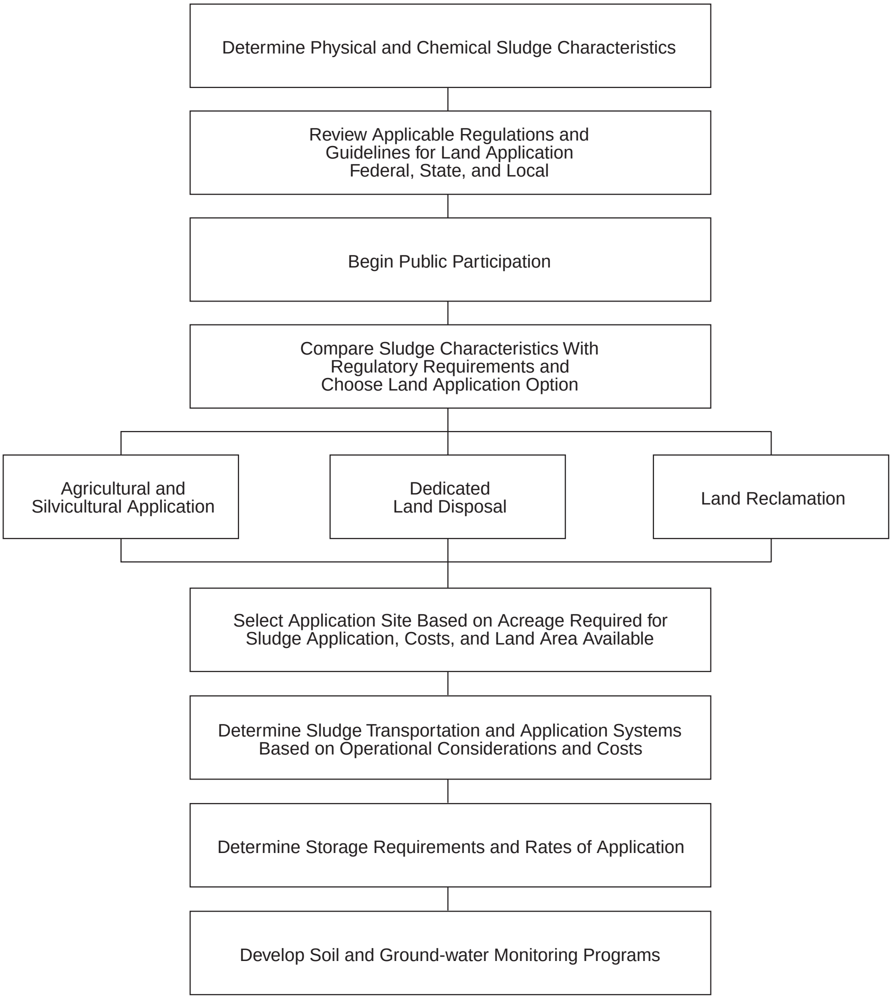


Fig. 23–25. Simplified planning procedure for land application of WTP residuals

The volume of these streams may constitute 3–10 percent of the plant production, a potentially significant amount of recoverable water. Discharge to a surface water, a common practice in the past, is often prohibited under current pollution regulations. Discharge to a sewer system or recycling of these streams is often the only viable alternative.

Discharge to the Sanitary Sewer System Discharge to the sanitary sewer system is a common practice for disposal of both sedimentation basin underflow and spent-filter backwash water. The primary factor in determining the acceptability of this practice is the concurrence of the utility responsible for wastewater collection, treatment, and disposal. Wastewater utilities with an effective industrial pretreatment program may have limits on the quantity of flow of solids that can be added to a sewer. The discharge of water treatment wastes to the sanitary sewer system may substantially increase the hydraulic and/or solids loading on wastewater collection and treatment facilities. If the wastewater system capacity is adequate, and if the practice does not negatively impact the wastewater utility's sludge disposal practices, the wastewater utility may be willing to accept the water treatment wastes. Conditions for such acceptance often include stipulations on the timing of the discharges, the maximum discharge rate, monitoring of waste characteristics, etc. In some instances, the discharge of water treatment wastes to the sanitary sewer system may benefit both utilities.

Recycle Concerns Recycling of the liquid waste streams creates concerns about the effects on the quality of the finished water. The primary contaminants of concern in the liquid waste streams are:

- *Giardia* cysts
- *Cryptosporidium* oocysts
- Turbidity
- Disinfection by-products (DBPs), including total trihalomethanes (TTHMs)
- DBP precursors, including TTHM formation potential (TTHMFP)
- Total organic carbon (TOC)
- Assimilable organic carbon (AOC)
- Taste and odor-causing compounds
- Manganese and iron

Other contaminants may also be a concern, depending on the raw-water constituents and the treatment processes used.

The introduction of these contaminants back into the main treatment train may upset process performance, requiring adjustments to the process to achieve a desired finished water quality. The threat of waste streams to treatment facility performance can also be hydraulic in nature. If a treatment plant cannot easily adjust the raw-water flow to compensate for the start of a recycle stream, then the plant flow will increase. This increase in flow can affect several treatment operations, including chemical feed rates, process loading rates, the hydraulic profile, and downstream pumping facilities. As most plants operate at their best under steady conditions, changes in plant operations can diminish treatment performance. Storage of the recycle flows is often provided so that these flows can be introduced into the main treatment flow at a controlled rate.

Equalization of waste streams is necessary for at least two reasons: to avoid abrupt changes in the flow rate and water quality at the inlet to the main treatment plant, and to minimize the size required and allow consistent operation of any waste stream treatment facility. Typically, the waste stream flows by gravity to the equalization basin. Waste stream pumping is then required, either before or after the waste stream treatment process(es), for the flow to reach the head of the plant.

Treatment To address the concerns associated with recycle flows, they are often introduced at the head of the treatment process or receive separate treatment before being introduced into the main flow of the plant.

Typical locations for reintroducing recycle streams to the treatment train are shown in Figure 23–26. The most common location is at the head of the plant, such as the inlet to a presedimentation basin or rapid-mix basin. In some cases, recycle streams

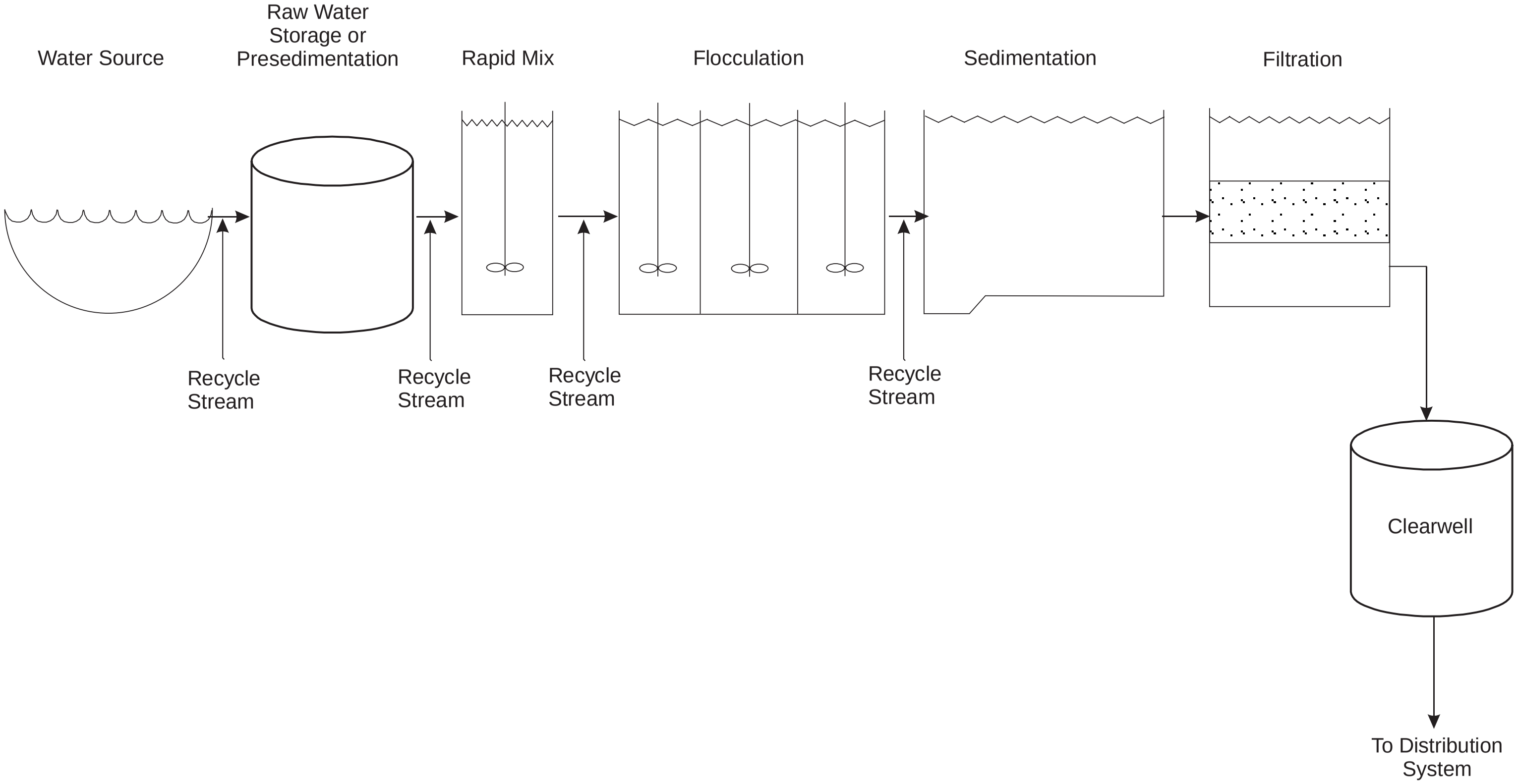


Fig. 23–26. Potential locations for return of recycle flows

have been added at the inlet of the flocculation basins; this has normally been done with untreated spent-filter backwash water, because solids in the recycle are thought to improve flocculation and sedimentation of the main process flow.

Because of concerns about potential effects on finished water quality, separate treatment of the liquid waste streams may be required before they are recycled. In California, the state's Safe Drinking Water Act[77,78] required that new treatment plants have solids removal for spent-filter backwash water recycle streams, although no specific treatment methods are specified.

Treatment options for recycled flows include:

- Lagooning, with or without chemical addition
- Batch sedimentation, with or without chemical addition
- Continuous flow conventional or high-rate sedimentation, with or without chemical addition and flocculation
- Dissolved air flotation
- Granular bed filtration
- Microfiltration

The recycled flows may also be disinfected following the above processes and before introduction into the main treatment process.

Information presented by Cornwell and Lee[79] indicates that very low overflow rates must be used in settling spent-filter backwash water without chemical addition. For 70–80 percent removal of *Giardia*- and *Cryptosporidium*-size particles, without chemical pretreatment, overflow rates of less than 0.05 gpm/ft^2 (0.1 m/h) were required. The addition of a nonionic polymer showed improvement to 90 percent removal at overflow rates of 0.2 to 0.3 gpm/ft^2 (0.5 to 0.8 m/h). At another plant, virtually no removal of *Giardia*-size particles was detected at 1.0 gpm/ft^2 (2.5 m/h), but the addition of less than 1 mg/L of nonionic polymer resulted in greater than 90 percent removal of the same-size particles; the removal of larger particles, such as flow containing microbial contaminants, must also be considered when examining pathogen removal.

More recent work[80] achieved different results in treating spent-filter backwash water by sedimentation. No significant improvement in the removal of total suspended solids (TSS) was shown with polymer addition. Some improvement in 2- to 10-micron (μ) particle removal was achieved by adding polymer. However, the greatest level of particle removal (about 90 percent) was achieved through a combination of pH adjustment and the addition of both ferric chloride and polymer, suggesting that particle charge is an important factor in settling spent-filter backwash water.

In addition to the reduction of particulates, turbidity, and pathogens, waste stream treatment may also be necessary to prevent the recycle of DBP precursors, TOC, and AOC. Cornwell and Lee[79] examined the removal and recycle of these contaminants at full-scale plants. Although waste streams had high levels of TTHMFP and TOC prior to settling, the settled waste streams had TTHMFP and TOC levels near those found in the raw water. The TTHM precursors and TOC were apparently associated with solids in the waste streams and were removed by sedimentation.

At two WTPs, high AOC levels were found in the untreated waste streams. Contrary to what was seen with TTHMFP and TOC, little AOC removal was achieved with settling. The end result is that high levels of AOC were recycled to the head of the plants. The AOC levels of finished water at one plant were not affected by the recycle, but were increased substantially during recycle at the other plant.

Results of a study on a full-scale plant using both tube and plate settlers have been reported.[81] The tube settlers were installed in an existing circular clarifier and the plate settlers were installed in a new circular basin. The spent-filter backwash water was pumped to the clarifiers from an equalization basin. No separate flocculation facilities were provided.

Both clarifiers consistently achieved greater than 90 percent reductions in turbidity and 2- to 5-μ particles with the addition of 0.7 mg/L anionic polymer. Treated turbidities were in the range of 2.0-3.6 NTU. Loading rates of 0.20–0.38 gpm/ft^2 (0.49–0.93 m/h) were tested with little variation in performance.

THMs and TTHMFP were also measured in the untreated and treated backwash waters. Total THMs were about 40 μg/L in the untreated water, and were not significantly affected by treatment. Total THMFP, however, was reduced by 45 to 55 percent, to approximately 1,000 μg/L. Little difference between the performance of the tube and plate settlers was shown.

Results from studies at two full-scale WTPs in metropolitan Phoenix, Arizona, the Verde and Mesa plants, have been reported.[82] Both plants have plate settling facilities that include polymer feed, rapid mix, flocculation, and plate settler. At the Verde plant, a combination of spent-filter backwash water, centrate, and gravity thickener overflow is treated; the Mesa plant treats only spent-filter backwash water. Facilities at both plants are operated continuously for 6 to 8 hours per day.

Results from the Verde plant showed consistent treated turbidities of less than 25 NTU with the addition of 0.4 mg/L polymer and loading rates of up to 0.39 gpm/ft^2 (0.95 m/h). At the Mesa plant, treated turbidities were consistently below 20 NTU at loading rates of up to 0.6 gpm/ft^2 (1.5 m/h). Polymer did not have much impact on turbidity removal at the Mesa plant. Turbidities of the influents to the recycle treatment facilities at both plants ranged from below 20 NTU to about 100 NTU.

Another study reported on the use of plate settlers to treat spent-filter backwash water from a direct filtration plant.[83] It was noted that the backwash solids are of low density, are highly organic, and have poor settling characteristics. The plate settlers were operated at a maximum 0.25 gpm/ft^2 (0.1 m/h) with polymer addition. The treated water averaged less than 1.5 NTU and was returned to the plant influent, where the raw water is typically less than 1.0 NTU.

In a test of microfiltration to treat a combination of spent-filter backwash water and clarifier sludge blowdown from a conventional treatment train, the waste stream was spiked with *Giardia* cysts and *Cryptosporidium* occysts before MF treatment. No cysts, oocysts, or coliforms were detected in the MF-treated water, and turbidities were consistently 0.1 NTU. High levels of particle removal were also shown using particle counters.[84] There is the potential that membrane treatment of the recycle flows might allow direct introduction into the final disinfection process, eliminating the added hydraulic load on the balance of the treatment plant.

RADIOACTIVE WASTE DISPOSAL

A variety of treatment processes can be used to remove radioactive contaminants from drinking water, such as conventional coagulation/filtration, ion exchange, lime softening, reverse osmosis, and granular activated carbon.[85–90] These processes separate the contaminants from drinking water and concentrate them in the waste streams. Because these processes are also commonly used to remove other nonradioactive contaminants, the possibility exists for these processes to concentrate significant levels of radioactivity in waste streams, even if the treatment was not originally designed or intended to remove radioactivity.

In addition to the waste streams containing the concentrated contaminants, some materials used in treating drinking water will adsorb radioactive contaminants and permanently retain them, such as sand used in filtration processes, which adsorbs and retains radium on its surfaces. Although the radium does not interfere with the treatment process, when the sand is replaced, it must be disposed of in a hazardous waste disposal site.

At the time of this writing, the disposal of naturally occurring radioactive materials (NORM) from drinking water treatment processes are not regulated by the federal government. Because of the concern for the proper disposal of these wastes, in July 1990 the U.S. EPA Office of Drinking Water published ‘‘Suggested Guidelines for the Disposal of Drinking Water Treatment Wastes Containing Naturally-Occurring Radionuclides.’’[91] EPA emphasizes that in most cases, state agencies with authority over NORM wastes are the most informed sources and should be consulted. The current methods of disposal are shown in Table 23–22.[1]

The information that is presented in the following sections on radioactive liquids, sludges, and solids disposal has been summarized from the EPA guidelines mentioned above.[91]

Liquid Disposal

Discharge to Surface Waters The NRC has established levels of radionuclides that are allowed to be released into unrestricted areas of the environment by its licensees (10 CFR Section 20,1302 (b)(2)(i), and Sections 20.1001–20.2401). The referenced NRC limits are as follows: 60 pCi/L and 300 pCi/L for Ra-226 and Ra-228, and

TABLE 23–22. Summary of Current Disposal Practices of Water Treatment Waste Containing Radionuclides

Water Type	Disposal Method
Liquids	Direct discharge to surface water
	Direct discharge to sanitary sewer
	Deep well injection
	Irrigation
	Lagooning/Evaporation ponds
Sludge	Lagooning (temporary)
	Landfill disposal
	• No pretreatment
	• With prior lagooning
	• With mechanical dewatering
	Land disposal
	Licensed low-level radioactive waste disposal site
Solids	Landfill disposal
	Licensed low-level radioactive waste disposal facility

Source: Reference 1.

natural uranium, respectively. If no state or local standards are in place, the NRC standards may be used as a guideline for surface water discharge. Some states and local authorities have promulgated conservative limits of 10 percent of the NRC levels under 10 CFR 20 for release of radionuclides into the environment.[1]

Discharge into Sanitary Sewers State or local regulations that set limits for wastes discharged to sewer systems will govern those discharges. The NRC limits the discharge of wastes containing radioactive materials into sanitary sewers by licensees. For NRC licensees, the monthly quantity of soluble radium-226, radium-228 and natural uranium, diluted by the average monthly quantity of total water treatment wastes released into the sewer, should not exceed 600 pCi/L, 600 pCi/L, and 3,000 pCi/L, respectively. Also, the gross quantity of all radioactive material combined, excluding tritium and carbon-14, released by the facility into the sanitary sewer should not exceed 1 curie per year, according to the same NRC standards. These are put forth by EPA in this context as standards that might be considered.[1]

Well Disposal A drinking water treatment plant owner interested in disposing of wastes containing radionuclides into an injection well should consult with the appropriate state agency first. State regulations may be more stringent than federal requirements and may ban such practices.

Regulation of water treatment plant wastes containing radionuclides depends on the concentrations of radionuclides present and on the type of well. Shallow wells are defined as those above or in an underground source of drinking water (USDW). Shallow injection of radioactive wastes is currently a banned practice.

Radioactive wastes are treated differently from nonradioactive wastes under applicable Underground Injection Control (UIC) programs. Currently a waste is considered radioactive if it contains concentrations of more than 60 pCi/L, or natural uranium of more than 300 pCi/L.

Well disposal of radioactive waste below a USDW is currently considered a Class V well injection and is under study by EPA as part of the Class V regulatory development effort.

Other Options Other treatment or disposal options for a liquid waste containing naturally occurring radionuclides include evaporation, sand drying or lagooning, chemical precipitation of contaminants, and other solids separation techniques. Lagoons or other impoundments should, at a minimum, be lined to prevent infiltration. Any evaporation unit should be designed and operated properly to ensure isolation of the waste from the water table.

Solids and Sludge Disposal

Table 23–23 shows disposal guidance according to current EPA guidelines for landfill disposal. In all cases, sludge concentrations are expressed in terms of dry weight.

Land Disposal EPA does not recommend the application, mixing, or otherwise spreading of water treatment wastes that contain naturally occurring radionuclides onto open land (i.e., farmland, pasture, orchard or forestry lands, construction sites, roadbeds, etc.).

TABLE 23–23. EPA Guidelines for Radium and Uranium Disposal

Sludge Concentration	Disposal
<3 pCi/g of radium and <50 g/g of uranium	Can mix and spread dewatered sludges with other municipal refuse in a municipal landfill. Sludges should be <10% of volume going into landfill.
3 to 50 pCi/g of radium	Use physical barrier (earthen cover) to prevent radon release; avoid inappropriate use of site.
50 to 2,000 pCi/g of radium	Consider RCRA permitted hazardous waste sites.
50 to 500 g/g of uranium	Consider disposal at licensed lowland radioactive waste facility or recovery of the uranium.
>2,000 pCi/g of radium or > 500 g/g of uranium	Use low-level radioactive waste facility or other facility with permit for NORM waste disposal.

MEMBRANE CONCENTRATE DISPOSAL

Introduction and Background

Reverse osmosis (RO), electrodialysis (ED) or electrodialysis reversal (EDR), nanofiltration (NF), ultrafiltration (UF), and microfiltration (MF) are all membrane processes that produce residual waste streams or concentrates.

A publication of the U.S. Office of Technology Assessment has ranked the waste concentrate generation from various membrane processes in terms of percent recovery of feedwater and percent disposal as waste concentrate (Table 23–24).

Conventional methods employ disposal of concentrates to surface bodies of water; spray irrigation combined with another dilution stream; deep well injection; drainfields; boreholes; and wastewater collection systems. More costly, nonconventional methods include evaporation and crystallization technologies, evaporation by solar ponds, and solar distillation.

Table 23–25[1] summarizes concerns, constraints, and requirements of conventional concentrate disposal methods.

TABLE 23–24. Membrane Concentrate Generation

Membrane Process	Percent Recovery of Feedwater	Percent Disposal as Concentrate
UF	80–90	10–20
NF	80–95	5–20
Brackish water RO	50–85	15–30
Seatwater RO	20–40	60–80
ED	80–90	10–20

Source: Reference 1.

TABLE 23–25. Concerns and Requirements Associated with Conventional Disposal Methods

Disposal Method	Regulatory Concerns	Other Requirements
Disposal to surface water	Receiving stream limitations Radionuclides, Odors (hydrogen sulfide) Low dissolved oxygen levels Sulfide toxicity Low pH	Mixing zone Possible pretreatment Multiple port diffusers Modeling of receiving stream
Deep well injection	Confining layer Upconing to USDWs Injection well integrity Corrosivity	Well liner Monitoring well Periodic integrity test Water quality of concentrate must be compatible with the water quality in the injection zone
Spray irrigation	Groundwater protection	Monitoring wells Possible pretreatment Backup disposal method Need for irrigation water Availability of blend waters
Drainfield or borehole	Groundwater protection	Monitoring wells Proper soil conditions and/or rock permeability
Sanitary sewer collection systems	Effect on local wastewater treatment plant performance (toxicity to biomass or inhibited settleability in clarifiers)	None

Source: Reference 1.

Conventional Disposal Methods

Surface Water Discharge A number of constituents in membrane concentrates have been identified as potential threats to surface water quality. Groundwaters that may be treated with membranes may be devoid of dissolved oxygen and contain hydrogen sulfide. A lack of dissolved oxygen and the presence of detectable quantities of hydrogen sulfide are toxic to aquatic organisms in surface waters. Fortunately, both conditions are easily remedied with relatively simple and inexpensive treatment. Aeration will add dissolved oxygen and remove the hydrogen sulfide simultaneously. Some facilities have also removed the hydrogen sulfide through chemical treatment.

The presence of radionuclides in the concentrate can pose a major water quality problem and an even more significant public relations problem, since combined radium values exceeding 100 pCi/l have been measured in reverse osmosis brines. An obvious disposal concern for the surface water discharge of concentrate is the impact of the concentrated salts (dissolved solids) on the receiving water. Water quality criteria for freshwater surface waters will limit the capacity of a freshwater body to receive a brine. Fluoride and several metals have been found in high concentrations in the concentrates of specific facilities.[1]

The nitrogen and phosphorus concentrations in large-volume brines can provide a significant nutrient load to receiving waters.

Treatment of the brine before disposal has generally not been found to be practical except for aeration and hydrogen sulfide removal. Therefore, dilution of the brine has been the only way to meet receiving water quality criteria that are not met at the ‘‘end of pipe.’’ Dilution calculations do not usually require sophisticated models. Often, desktop calculations can suffice to determine the necessary effluent limitations since the question is one of mass balance. This is especially true when the surface water flow past the brine outfall is easily measured and an assumption of complete mixing is appropriate. When an assumption of complete mixing is not appropriate, simple mixing models such as the EPA-supported CORMIX1, CORMIX2, UPLUME, UMERGE, etc., can be used to establish effluent limits.

Disposal to Sanitary Sewers Discharge of concentrate into sanitary sewer systems is sometimes feasible if the concentrate mixture is not toxic or otherwise adversely affects the wastewater treatment plant processes. The restrictions and requirements of sewer agency user ordinances must be complied with. Special concern will be given to the discharge’s effect on the metals and hazardous constituent levels in the plant biosolids. In California and other western states, some cities have used regional dedicated brine interceptor lines, which may discharge to the ocean. They were originally installed to carry off waste brines from the oil industry and other sources of industrial brine. Now they are used for concentrates from inland desalters used to reduce total dissolved solids in groundwaters, and for residual brines from municipal well head treatment such as nitrate-reduction ion-exchange systems. Typically, costs associated with the construction of these regional brine interceptors are high and in some cases exceed the cost of the proposed treatment processes.

Deep-Well Injection

Regulations. EPA has consolidated the rules into the Underground Injection Control (UIC) regulations and compiled them in the Code of Federal Regulations (40 CFR Parts 144 and 146), with the general program requirements compiled in 40 CFR Part 122. Most states have opted to allow full regulatory control by EPA on certain kinds of injection wells, by choosing not to develop and enforce rules that would give them primacy over that type of wells.

Classes of Injection Wells. Injection wells used for disposal of membrane concentrations are classified under current federal regulations as Class I, Group 2, injection wells that inject nonhazardous industrial and/or municipal fluids beneath the lowermost formation containing a source of drinking water at least ¼ mile (0.5 km) from the well.

When water treatment facilities are used to remove hazardous waste from a source water, then the residuals are considered hazardous water, and extra precautions must be taken to ensure proper operation and monitoring of the injection system. In some states hazardous waste injection wells are banned, and in several other states with primacy rights, banning hazardous waste injection wells is being considered at this time.

Water treatment plant residuals may be in suspension or dissolved, and an evaluation may be completed on the effect that mixing these residuals with the receiving waters in the injection zone of a deep well will have. This mixing could cause the formation of precipitants, flocculants, gases, and bacterial mats, which could hinder the injection process and could harm the receiving aquifer by plugging it or could otherwise reduce its permeability.

Requirements of Injection Sites. Injection must be below the lowermost formations containing an underground source of drinking water (USDW). Other key requirements include:

- The injection must be into an aquifer zone that has water with at least 10,000 mg/L TDS—that is, waters that do not qualify as USDWs.
- The injection zone of the aquifer must be separated from USDW zones above it by hydrologically impermeable formations that preclude the upward migration of the injected fluids into the USDWs.
- The injection zone must be a good receiving zone, both from the water quality point of view and the hydrologic point of view. The receiving and injection waters must be compatible physically, chemically, and bacteriologically. The receiving zone must be of high permeability and effective porosity, such that the volumes of water injected can be discharged into the zone without excessively raising the pressure in the receiving formation.

Boreholes Drainfields and boreholes for concentrate disposal can obtain regulatory approval when the concentrate water quality meets the regulatory requirements for discharge into the surficial groundwater aquifers, and when soil conditions and permeability of rock strata permit use of the site. In Florida, this occurs in coastal areas where the groundwater quality is greater than 10,000 mg/L TDS and U.S. Drinking Water (USDW) regulatory parameters are not exceeded.

Surficial aquifer discharge of brine from coastal seawater RO plants using shallow boreholes or seawells, where the groundwater is very brackish, may be more cost-effective and environmentally palatable than ocean outfall disposal.

Spray Irrigation Certain conditions must be met for the spray irrigation disposal option to be viable. These conditions are:

- The total dissolved solids (TDS), chloride content, or salinity of the concentrate or the mixture of concentrate and a blend liquid cannot exceed a level that will damage the grass or crop being irrigated.
- There must be a requirement for irrigation water in the vicinity of the water treatment plant to avoid a long conveyance system.
- Generally, a backup disposal system is required, such as storage for use during sustained periods of rainfall when irrigation is not needed.
- The water quality of the surficial groundwater must be protected from degradation by the concentrate disposal method.
- A complete system of monitoring wells is usually required to check the overall irrigation system performance. A monitoring plan will probably be a prerequisite to obtaining an operating permit.
- Local, state, and federal regulations must be met.

Spray irrigation of concentrate almost always requires dilution prior to irrigation so as to:

- Prevent pollution or degradation ground water resources while meeting regulatory requirements.
- Reduce the chloride content to acceptable levels so as not to damage grass or crops being irrigated.

Other Methods for Concentrate Disposal

The Brine Concentrator The brine concentrator (BC) is most often designed as a vertical-tube, falling film evaporator, although horizontal spray film and plate type evaporators are also used in this application. The brine concentrator blowdown is usually processed further to a solid waste in either solar ponds or a crystallizer.

The common characteristic of most brine concentrator designs is the circulation of a slurry of calcium sulfate ($CaSO_4$) crystals, which act as seeds. Calcium sulfate and other scale-forming compounds preferentially precipitate on the circulating seed crystals over the heat transfer surfaces to prevent scaling. The development of the seeding technique for calcium sulfate and silica allowed brine concentrators to process waste waters at or near saturation in calcium sulfate and silica to very high concentrations. Up to 20 percent total solids by weight is often achievable in the brine concentrator discharge, while a distillate of better than 10 ppm TDS can be achieved. Figure 23–27 shows the typical components of a brine concentrator.[93]

Mechanical vapor recompression (MVR) evaporators, or more simply, vapor compression (VC), utilize a compressor to raise the condensing temperature and pressure

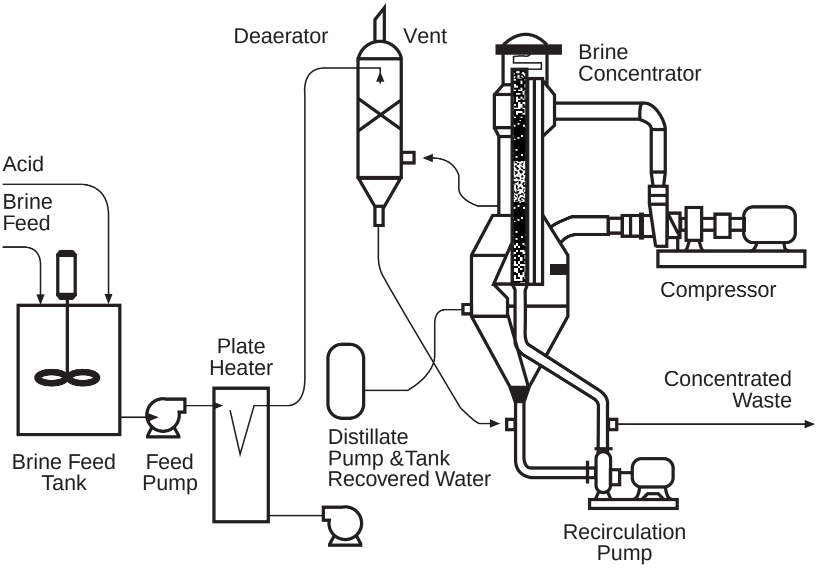


Fig. 23–27. Typical components of a brine concentration (*Source:* Reference 92)

of the evaporated water sufficiently to reuse the vapor as the primary heat source for evaporation. In this manner, the heat of vaporization in the evaporated water is recovered. MVR systems can obtain very high thermal efficiencies, with only 35 BTUs required to evaporate one pound of water (about 85 kWh/1,000 gallons).

Representative brine concentrator process conditions for zero discharge applications are listed in Table 23–26.[1] These conditions are shown as ranges and should be used as guidelines.

Crystallizers are much more costly to build and operate than evaporators or brine concentrators. Because of this great cost differential, operating a membrane system at its limits of concentration may become counterproductive in some instances. If the membrane system waste brine is too high in concentration, a brine concentrator may be unusable and a much larger crystallizer may be required to reach zero discharge.

Waste Crystallizers A basic waste crystallizer flow scheme is shown in Figure 23–28.[92] Because the crystallizer circulates a slurry, some method of dewatering is required. The drawing shows a centrifuge, but a pressure filter, rotary filter, or belt filter press could also be used. The crystallizer does not generate a liquid blowdown. The total dissolved solids level eventually reaches an equilibrium point, determined by the composition of the feed stream.

An excellent source of information on brine disposal, brine concentrators, and crystallizers is the 1996 EPA, AWWA, ASCE *Technology Transfer Handbook on Management of Water Treatment Plant Residuals*.[1]

Capital and Operating Costs. Figure 23–29 provides an order of magnitude estimate of the operating costs for typical brine concentrator systems. Capital costs (1997 dollars) range from about \$2.5M to \$6.5M, for a brine concentrator with capacities ranging from 100 to 450 gpm. For smaller systems, those below 100 gpm capacity, the equipment is generally skid-mounted. Larger systems will require some field fabrication of the vapor bodies and assembly of the structural steel. Capital cost estimates are based on installed outdoor systems with foundations. Control rooms are assumed to be located with the central plant facilities.

TABLE 23–26. Typical Brine Concentrator Process Conditions in Zero Discharge Applications

Feed TDS	2,000–20,000 ppm
Feed temperature	40–120°F
Preheater approach temperature	7–15°F
Concentration factor	8–120
Seed slurry concentration	1–10%
Boiling point rise	1–10°F
Compression ratio	1.20–2.0
Recirculation pump rate	20–40 × feed
TDS of waste brine, weight %	15–22%
Total solids waste brine	15–30%
Overall power/1,000 gal	175–110 kWh
Distillate TDS	5–25 ppm

Source: Reference 1.

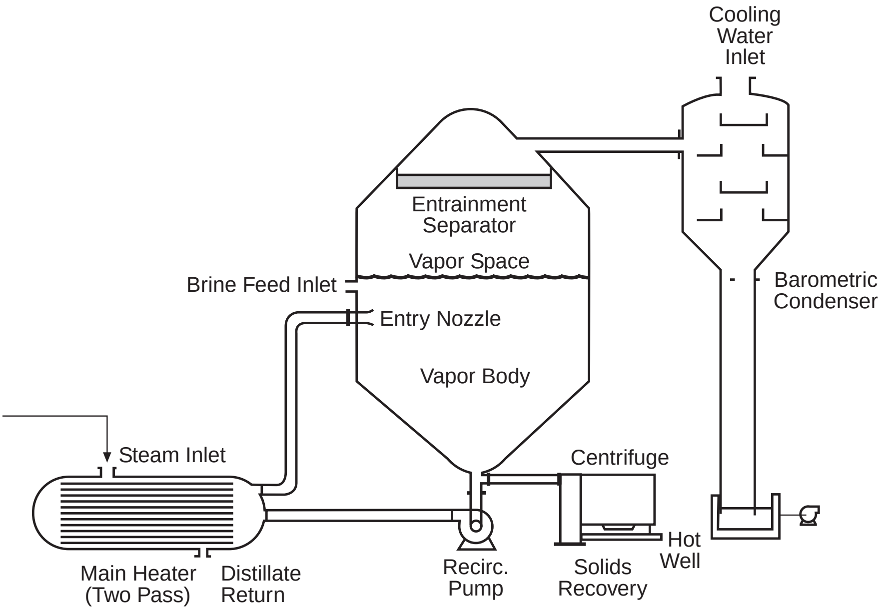


Fig. 23–28. Brine crystallize schematic

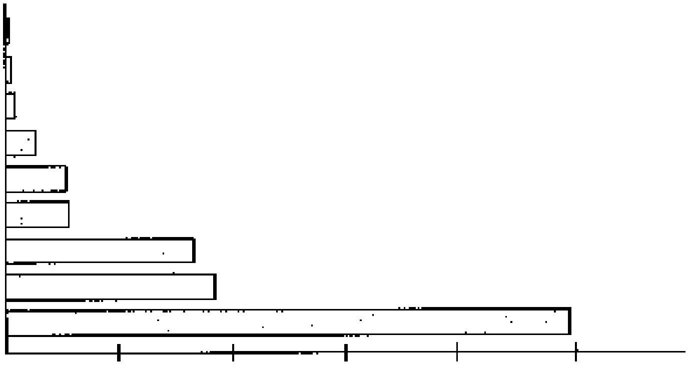


Fig. 23–29. Operating costs of brine concentration systems

Solar Technologies, Evaporation Ponds The suitability of evaporation ponds for concentrate disposal depends on local climatological conditions. The principal factors that affect the evaporation rate are relative humidity, wind velocity, barometric pressure, air and water temperature, and the salinity of the water.

Since evaporation losses are directly proportional to area, large areas of land must be available for this option to be viable. A backup system is necessary, since an unusual wet-weather period could affect the evaporation rate. Some large brackish water RO plants in the Middle East have employed this method of concentrate disposal successfully, or have used it in combination with percolation onto the desert.

REFERENCES

1. EPA, AWWA, ASCE, *Technology Transfer Handbook Management of Water Treatment Plant Residuals,* EPA Office of Research and Development, Washington, D.C. 20460, EPA/625/R-95/006, April 1996.
2. Tanner, S. A., and Ongerth, J. E., ‘‘Evaluating the Performance of Slow Sand Filters in Northern Idaho,’’ *JAWWA* 82(12):51, 1990.
3. Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers. *Recommended Standards for Water Works.* New York, NY: Health Education Services, 1992.
4. Pontius, F. W. (ed.), *Water Quality and Treatment: A Handbook of Community Water Supplies,* 4th ed., McGraw-Hill, New York, New York, 1990.
5. American Water Works Association Research Foundation (AWWARF), ‘‘Disposal of Wastes from Water Treatment Plants,’’ *JAWWA* 61:541, 619, 1969.
6. Mickley, M., Hamilton, R., Gallegos, L., and Truesdall, J., *Membrane Concentrate Disposal,* American Water Works Association/American Water Works Association Research Foundation, Denver, CO, 1993.
7. USEPA, ‘‘National Primary Drinking Water Regulations; Radionuclide; Proposed Rule,’’ *Fed. Reg.* 56(138):33,050, July 18, 1991.
8. Cornwell, D., Bishop, M. M., Gould, R. G., and Vandermeyder, C., *Handbook Of Practice: Water Treatment Plant Waste Management,* American Water Works Association Research Foundation, Denver, CO, 1987.
9. AWWA Committee on Sludge Disposal, ‘‘Water Treatment Plant Sludges—An Update of the State of the Art,’’ *JAWWA,* pp. 498–503 and 543–554, September and October 1978.
10. Gruninger, Robert M., ‘‘Disposal of Waste Alum Sludge from Water Treatment Plants,’’ *J.WPCF* 47:3:543–556, 1976.
11. Chapman, Robert L., *Solids Disposal Study Summary Report,* Denver Board of Water Commissioners Foothill Treatment Plant, June 1974.
12. AWWA Committee on Sludge Disposal, ‘‘Lime Softening Sludge Treatment and Disposal,’’ *JAWWA*, pp. 600–608, November 1981.
13. Cornwell, D. A., and Westerhoff, G. P., ‘‘Management of Water Plant Sludge,’’ *Sludge Management*, Ann Arbor Science Publishers, Ann Arbor, Mich., 1981.
14. AWWA Water Treatment Plant Waste Committee Report, unpublished data, December, 1972.
15. AWWA, *Water Quality and Treatment*, 3rd ed., McGraw-Hill Book Company, New York, 1971.
16. EPA, *Slow Sand Filter Maintenance Costs and Effects of Water Quality.* EPA/600/2–85/056. NTIS PB85199669, Cincinnati, OH, 1985.

17. O'Connor, J. T., and Novak, J. T., ''Management of Water Treatment Plant Residuals,'' AWWA Water Treatment Waste Disposal *Proceedings*, AWWA Annual Conference, 1978.
18. EPA, *Barium and Radium in Water Treatment Plant Wastes.* EPA/600/2–85/006. Cincinnati, OH, 1984.
19. Saltonstall, C.W., Jr., and R.W. Lawrence, ''Calculation of the Expected Performance of Reverse Osmosis Plants,'' *Desalination* 42:247–253, 1982.
20. Cornwell, David A., and Susan, James A., ''Characteristics of Acid-Treated Alum Sludges,'' *JAWWA*, pp. 604–608, October, 1979.
21. Knocke, W. R., J. R. Harmon, and B. E. Dulin, ''Effects of Coagulation on Sludge Thickening and Dewatering,'' *JAWWA* 79:89, 1987.
22. Dulin, D.E and W.K. Knooke, ''The Impact of Incorporated Organic Matter on the Dewatering Characteristics of Aluminum Hydroxide Sludges,'' *JAWWA* 81(5):74, 1989.
23. Pigeon, Paul E., et al., ''Recovery and Reuse of Iron Coagulants in Water Treatment,'' *JAWWA*, pp. 397–403, July, 1978.
24. Singer, P. C., ''Softener Sludge Disposal—What's Best,'' *Water and Waste Engineering*, p. 25, December, 1974.
25. Black and Veatch, ''Report on Water Treatment Plant Waste Disposal,'' Wichita, Kans., December, 1969.
26. Laurence, Charles, ''Lime Soda Sludge Recirculation Experiments at Vandenberg Air Force Base,'' *JAWWA* 55:2:177, 1963.
27. Burris, Michael A., et al., ''Softening and Coagulation Sludge—Disposal Studies for a Surface Water Supply,'' *JAWWA*, 68(s):247, May, 1976.
28. Calkins, R. J., and Novak, J. T., ''Characterization of Chemical Sludges,'' *JAWWA* 65(6):423, 1973.
29. *Recovery of Lime and Magnesium in Potable Water Treatment*, USEPA Technol. Series, EPA-600/2-76-285, December 1976.
30. Novak, J. T., and Calkins, D. C., ''Sludge Dewatering and Its Physical Properties,'' *JAWWA* 67:1:42, 1975.
31. Knocke, W. R., Ghosh, M. M., and Novak, J. T., ''Vacuum Filtration of Metal Hydroxide Sludges,'' *J. Environ. Eng.*, ASCE 106(2):363, 1980.
32. Werle, C. P., Novak, J. T., Knocke, W. R., and Sherrard. J. H., ''Mixing Intensity and Polymer Sludge Conditioning,'' *J. Environ. Eng.*, ASCE 110(5):191, 1984.
33. Bishop, Stephen L., ''Alternate Processes for Treatment of Water Plant Wastes,'' *JAWWA*, pp. 503–506, September 1978.
34. Black, A. P., ''Split-Treatment Water Softening at Dayton,'' *JAWWA* 58:1:97, 1966.
35. Shuey, B. S., ''Economics of Split-Treatment Water Softening,'' *JAWWA* 58:1:107, 1966.
36. Singley, J. E., and Brodeur, T. P., ''Control of Precipitative Softening,'' AWWA Water Quality and Technology Conference, 1980.
37. Malcolm Pirnie, Inc., ''Evaluation of Water Treatment Facilities,'' unpublished report for the Mahoning Valley Sanitary District, Youngstown, Ohio, May 1980.
38. Knocke, W. R., ''Thickening and Conditioning of Chemical Sludges,'' *Proceedings of ASCE Environmental Engineering Conference*, San Francisco, Calif., July 1979.
39. Foster, W. S., ''Get the Water Out of Alum Sludge,'' *American City and County*, September 1975.
40. American Society of Civil Engineers (ASCE)/American Water Works Association (AWWA), *Water Treatment Plant Design*, 2nd ed., McGraw-Hill, New York, 1990.
41. Brown and Caldwell, Inc., ''City of Phoenix Water Residuals Management Study,'' study produced for the City of Phoenix, AZ, 1990.

42. Cornwell, D. A., and Koppers, H. M. M. (eds.), ‘‘Slib, Schlamm, Sludge,’’ American Water Works Association Research Foundation and KIWA Ltd., Denver, CO, 1990.
43. Kwczyinski, E., and Achterrman, V., ‘‘A Water Industry Data Base Report,’’ in *Proceedings of the AWWA/WEF Joint Residuals Management Conference,* Research Triangle Park, NC, 1991.
44. Howson, Louis R., ‘‘Sludge Disposal,’’ Chapter 10 in *Water Treatment Plant Design* (Robert L. Sanks, ed.), pp. 183–193, Ann Arbor Science Publishers, Ann Arbor, Mich., 1978.
45. Neubauer, W. K., ‘‘Waste Alum Sludge Treatment,’’ *JAWWA* 60:819, July 1968.
46. AWWA Research Foundation Report, ‘‘Disposal of Wastes from Water Treatment Plants,’’ *JAWWA* 61:541, October 1969.
47. Kansas Department of Health and Environment, *Design Criteria—Sludge Production and Storage Requirements,* Division of Environment, Bureau of Water Supply, Topeka, Kans., 1983.
48. King, P. H., et al., ‘‘Treatment of Waste Sludges from Water Purification Plants,’’ Bulletin 52, Virginia Water Resources Research Center, VPI, Blacksburg, Va., 1972.
49. Novak, J. T., and Langford, Mark, ‘‘The Use of Polymers for Improving Chemical Sludge Dewatering on Sand Beds,’’ *JAWWA* 69:106, February 1977.
50. Willhelm, J. H., and Silverblatt, C. E., ‘‘Freeze Treatment of Alum Sludge,’’ *JAWWA* 68:312, June 1976.
51. Parsons, Jack (project manager, Denver Board of Water Commissioners, Denver, Colo.), personal communication, June 1983.
52. Fulton, G. P., ‘‘Disposal of Wastewater From Water Filtration Plants,’’ *JAWWA* 61:322, July 1969.
53. Albertson, O. E., and Guidi, E. E., Jr., ‘‘Centrifugation of Waste Sludges,’’ *Water Pollution Control Federation* 41:4, 607, 1969.
54. AWWA Research Foundation Report, ‘‘Disposal of Wastes from Water Treatment Plants: Part 2,’’ *JAWWA* 61:11:619, 1969.
55. Kramer, A. J. and Whitaker, J., ‘‘Sludge Handling,’’ *Water and Waste Engineering,* May 1975.
56. Vesilind, P. A., *Treatment and Disposal of Wastewater Sludges,* Ann Arbor Science Publishers, Ann Arbor, Mich., 1979.
57. Rohen, M., ‘‘Characterization of Lime Sludges from Water Reclamation Plants,’’ D.Sc. dissertation, University of Pretoria, South Africa, 1978.
58. U.S. EPA, *Design Manual—Dewatering Municipal Wastewater Sludges,* EAP/625/1–87/014, Cincinnati, OH, September 1987.
59. Finney, J. Wiley, Jr., ‘‘Dewatering of Alum Sludge at Somerset, Kentucky,’’ presented at Kentucky–Tennessee AWWA annual meeting, Owensboro, Ky., November 11–14, 1979.
60. Parkson Corporation, ‘‘Summary of Test Results on Magnum Press,’’ Western Pennsylvania Water Company, New Castle, Pa., March 2–6, 1981.
61. Tait-Andritz, ‘‘Sludge Dewatering Test Report,’’ Houston, Tex., March 23 to April 15, 1976.
62. Hambor, J. Michael, ‘‘Dewatering of Water Treatment Sludges—The Belt Filter Press,’’ presented at the AIChE Joint Meeting—Central and Peninsular Sections, Clearwater Beach, Fla., May 28, 1983.
63. Ide, T., and Yusa, M., ‘‘Solid-Liquid Separation by Pelletting Flocculation Process,’’ 2nd Pacific Chemical Engineering Congress, August 1977.
64. Culp, Gordon L., and Culp, Russell L., *New Concepts in Water Purification,* Van Nostrand Reinhold, New York, 1974.
65. Westerhoff, Garrett P., and Cornwell, David A., ‘‘A New Approach to Alum Recovery,’’ *JAWWA,* pp. 709–714, December 1978.

66. Saunder, M., and Roeder, M., *Coagulant Review: A Critical Assessment,* American Water Works Association Research Foundation, Denver, CO, 1991.
67. Cornwell, David A., and Susan, James A., ''Characteristics of Acid-Treated Alum Sludges,'' *JAWWA,* pp. 604–608, October 1979.
68. Cornwell, David A., ''An Overview of Liquid Ion Exchange With Emphasis on Alum Recovery,'' *JAWWA* 71:12:741–744, 1979.
69. Cornwell, David A., and Lemunyon, Roger M., ''Feasibility Studies on Liquid Ion Exchange for Alum Recovery From Water Treatment Plant Sludges,'' *JAWWA* 72:1:64–68, 1980.
70. Cornwell, David A., et al., ''Demonstration Testing of Alum Recovery by Liquid Ion Exchange,'' *JAWWA,* pp. 326–332, June 1981.
71. AWWA Committee Report, ''Disposal of Water Treatment Plant Wastes,'' *JAWWA,* pp. 814–820, December, 1972.
72. Benjes, H. H., Jr., ''Treatment of Overflow from Sanitary Sewers,'' presented at the 9th Texas WPCA Conference, Houston, July 1970.
73. Culp, R. L., and Wilson, W. I., ''Is Alum Sludge Advantageous in Wastewater Treatment?,'' *Water and Wastes Engineering,* p. 16, July 1979.
74. Reeves, T., ''The Mahoning Valley Sanitary District Plans to Spread Slurry on Farmland,'' *Farms & Dairy,* Salem, OH, p. 10, November 13, 1980.
75. U.S. EPA, *Process Design Manual: Land Application of Sewage Sludge and Domestic Septage,* EPA/625/R-95/001, Cincinnati, OH, 1995.
76. Dempsey, B. A., and Elliott, A. H., ''Land Application of Water Treatment Sludges,'' in *Proceedings of the 44th Purdue Industrial Waste Conference,* Purdue University, Lewis Publishers, W. Lafayette, IN, 1990.
77. California Department of Health Services (CDHS), ''*Cryptosporidium* Action Plan,'' April 1995.
78. CDHS, ''California Safe Drinking Water Act,'' *California Health and Safety Code,* Articles 4 and 5, 1995.
79. Cornwell, D. A., and Lee, R. G., *Recycle Stream Effects on Water Treatment,* AWWA Research Foundation Report #90629, May 1993.
80. Cornwell, D. A., ''Treatment of Recycle and Backwash Streams,'' *Proceedings of Conference on Water Residuals and Biosolids Management: Approaching the Year 2000,* WFE/AWWA, August 1997.
81. Ashcroft, C. T., et al., ''Modifications to Existing Water Recovery Facilities for Enhanced Removal of *Giardia* and *Cryptosporidium,*'' *Proceedings of Conference on Water Residuals and Biosolids Management: Approaching the Year 200*0, WEF/AWWA, August 1997.
82. Narasimhan, R., et al. ''Design Criteria Evaluation for Washwater Treatment and Water Residuals Thickening Processes,'' *Proceedings of Conference on Water Residuals and Biosolids Management: Approaching the Year 2000,* WEF/AWWA, August 1997.
83. Hess, A., Affinito, A., Dunn, H., Gaewski, P., and Norris, E. ''Relationship of WTP Residual Characteristics, Facility Design and Operational Practices on the Performance of Residual Treatment Facilities at a Direct Filtration Plant,'' *Proceedings of the Joint Residuals Conference*, Phoenix, Arizona, AWWA/WEF, December 1993.
84. Thompson, M. A., Vickers, J. C., Wiesner, M. R., and Claney, J. L., ''Membrane Filtration for Backwash Water Recycle,'' *AWWA Annual Conference on Water Quality Proceedings*, pp. 1051–1064, 1995.
85. U.S. EPA, ''Determination of Radium Removal Efficiencies in Iowa Water Supply Treatment Process,'' Technical Note ORP/TAD-76–1, NTIS PB2S9509, Washington, D.C., 1976.
86. Brinck, W. L., et al., ''Radium Removal Efficiencies in Water Treatment Processes,'' *JAWWA* 70(1):31–35, 1978.

87. U.S. EPA, *Manual of Treatment Techniques for Meeting the Interim Primary Drinking Water Regulations,* EPA/600/8-77-995, Cincinnati, OH, 1977.
88. Lassovszky, P., and Hathaway, S., ‘‘Treatment Technologies to Remove Radionuclides from Drinking Water,’’ Easton, Maryland, EPA, Washington, D.C., May 24–28, 1983.
89. Sorg, T. J., Forbes, R. W., and Chambers, D. S., ‘‘Removal of Radium-226 from Drinking Water by Reverse Osmosis in Sarasota County, FL,’’ *JAWWA,* 72(4):230–237, April 1980.
90. Lowry, J. D., and Brandow, J. E., ‘‘Removal of Radon from Water Supplies,’’ *Journal of Environmental Engineering,* Vol. III, No. 4, 72(4):230–237, August 1985.
91. U.S. EPA, ‘‘Suggested Guidelines for the Disposal of Drinking Water Wastes Containing Naturally Occurring Radionuclides,’’ Office of Drinking Water, EPA, Washington, D.C., 1990.
92. HPD, Inc., product literature, Naperville, IL, 1994.

CHAPTER 24

Chemical Storage and Feeding Systems

INTRODUCTION

The importance of the design of chemical systems in a water treatment plant cannot be overemphasized. Although the chemical systems account for a only relatively small percentage of the capital costs of a water treatment plant, they usually account for a large percentage of the annual operation and maintenance costs. Proper design of these systems can reduce operation and maintenance costs and improve treatment efficiency. This chapter describes specific design considerations for liquid, solid, and gaseous chemical systems. The groups of chemicals used for particular water treatment functions are discussed, and the specific chemicals in each group are listed, as well as their pertinent physical data and characteristics. The rationale for selecting a particular chemical is presented and explained. Important design considerations for each type of chemical system are discussed, including the selection of chemical form, delivery, storage, feeding, conveyance methods, and safety.

CHEMICAL SELECTION

Function of Chemicals

Chemicals are used for coagulation and as coagulation aids, for softening, taste and odor control, disinfection, dechlorination, fluoridation, fluoride adjustments, pH adjustments, corrosion control, and removal of certain constituents such as iron, manganese, and heavy metals. Each of these functional classifications contains a variety of chemicals that can be used for the same purpose, and in some cases the chemicals serve more than one purpose. Table 24–1 describes the chemicals within these various unit processes.

Coagulants In most instances, surface waters require the use of a coagulant, while well waters do not. Aluminum sulfate (alum) is the most widely used coagulant. It is available in lump, ground, or liquid form and can be shipped in bulk or in 100-pound (45.4-kg) bags for dry forms, and drums, totes, and bulk for liquid form. However, it may be appropriate to use a coagulant other than aluminum sulfate, such as those listed in Table 24–1.

TABLE 24–1. Chemicals Used in Water Treatment*

Coagulants
Aluminum ammonium sulfate
Aluminum potassium sulfate
Aluminum sulfate (alum)
Calcium hydroxide (hydrated lime)
Calcium oxide (quicklime)
Ferric chloride
Ferric sulfate
Ferrous chloride
Ferrous sulfate
Polyaluminum chlorides
Polymers
Sodium aluminate

Coagulant aids
Bentonite
Calcium carbonate
Microsand
Organic coagulant aids
Polymers
Sodium silicate

Disinfection agents
Ammonia, anhydrous
Ammonium hydroxide
Ammonium sulfate
Bromine
Calcium hypochlorite
Chlorine
Chlorine dioxide
Chlorinated lime
Iodine
Monochloramine
Ozone
Peroxone (ozone and hydrogen peroxide)
Sodium hypochlorite

Dechlorination agents
Activated carbon
Calcium thiosulfate
Hydrogen peroxide
Ion-exchange resins†
Sodium bisulfite (sodium pyrosulfite)
Sodium meta-bisulfite
Sodium sulfate
Sulfur dioxide

Iron and manganese removal
Chlorine
Chlorine dioxide
Ozone
Potassium permanganate
Sodium hexametaphosphate

Softening
Calcium hydroxide (hydrated lime)
Calcium oxide (quicklime)
Carbon dioxide
Ion-exchange resins
Sodium carbonate (soda ash)
Sodium chloride
Sulfuric acid

Taste and odor control
Activated carbon
Bentonite
Chlorine
Chlorine dioxide
Hydrogen peroxide
Ozone
Peroxone
Potassium permanganate

Fluoridation
Ammonium silicofluoride
Calcium fluoride
Hydrofluoric acid
Hydrofluosilicic acid
Sodium fluoride
Sodium silicofluoride

Fluoride adjustment
Activated alumina
Ion-exchange resins
Magnesium oxide

pH adjustment
Calcium carbonate
Calcium hydroxide (hydrated lime)
Calcium oxide (quicklime)
Carbon dioxide
Hydrochloric acid
Sodium bicarbonate
Sodium carbonate (soda ash)
Sodium hydroxide (caustic soda)
Sulfuric acid

Corrosion control
Calcium hydroxide (hydrated lime)
Calcium oxide (quicklime)
Silicates
Sodium carbonate (soda ash)
Sodium hexametaphosphate
Sodium hydroxide
Sodium tripolyphosphate
Zinc orthophoshate

Algae control
Copper sulfate

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* See Appendix G for more information on specific chemicals.

† Consult manufacturer of particular resin for more information.

For instance, a raw water with high magnesium content may dictate the use of lime to allow for coagulation as well as precipitation of magnesium hydroxide. Heavy metals are removed more effectively at the high pH values resulting from the use of lime. Other coagulants include aluminum ammonium sulfate, aluminum potassium sulfate, ferric chloride, ferric sulfate, ferrous sulfate, and sodium aluminate.

Coagulant Aids In some cases, a coagulant aid is used to improve the efficiency of the coagulation process.[1] Bentonite has been used in low-turbidity waters and in upflow clarifiers. Bentonite clay provides a nucleus for floc formation and helps produce a heavy sediment blanket. Microsand is also used as a coagulant aid for a sand-ballasted process called Actiflo, described elsewhere in this text. Calcium carbonate also has been used to provide a nucleus for aluminum and iron hydroxide floc formation, and to add weight to the floc to aid in settling. Organic coagulant aids or polymers are widely used. There are both natural and synthetic types. Some are suitable for water treatment, while others are not. As discussed in Chapter 10, there are different types of polymers that can perform many functions. Polymers are supplied in solid or liquid form, and in bulk or 50-pound (22.7-kg) bags.

Sodium silicate plus activated silica has been used as a coagulant aid to toughen the floc through ionic and electronic bond formation.

Softening Softening can be accomplished by either ion exchange or the lime–soda softening process. Each process requires different chemicals. In ion exchange, the exchange resin typically is regenerated with sodium chloride, as described in Chapter 18. The lime–soda process requires calcium hydroxide or calcium oxide and sodium carbonate; carbon dioxide or sulfuric acid is required for subsequent pH adjustment. These processes are described in Chapter 13.

Taste and Odors The most common chemicals used for removal of tastes and odors are powdered or granular activated carbon. For intermittent or occasional taste and odor problems, oxidizing agents such as potassium permanganate, ozone, chlorine, peroxone, and chlorine dioxide may be used. See Chapter 19 for a detailed review of taste and odor control.

Disinfection Agents Chlorine is the principal disinfecting agent, although ultraviolet light, ozone, peroxone, monochloramine, and chlorine dioxide are also used. Bromine iodine and potassium permanganate have been used in specific circumstances. More detail is presented in Chapter 11.

Dechlorination Agents The materials that have been used, or proposed, for the dechlorination of waters include granular and powdered activated carbon, hydrogen peroxide, sodium thiosulfate (Na), and the sulfur (IV) species, which includes sulfur dioxide (SO_2), sodium bisulfite ($NaHSO_3$), sodium sulfate (Na_2SO_3), sodium metasulfite ($Na_2S_2O_5$), calcium thiosulfate, and vitamin C. Sulfur dioxide comes in gaseous form and uses the same type of storage and feeding equipment as chlorine.

Fluoridation Fluoride is added to the drinking water of some communities to reduce dental decay among children. Correctly proportioned, fluoridation is effective, but overdoses can be detrimental to teeth. The optimum fluoride concentration varies with air temperature, as shown in Table 24–2, because during warmer weather people drink

TABLE 24–2. Recommended Optimal Fluoride Level

Annual Average of Maximum Daily Air Temperature* (F)	Recommended Fluoride Concentrations		Recommended Control Range					
			Community Systems			School Systems		
	Community (ppm)	School† (ppm)	0.1 Below		0.5 Above	20% Low		20% High
High40.0–53.7	1.2	5.4	1.1	–	1.7	4.3	–	6.5
53.8–58.3	1.1	5.0	1.0	–	1.6	4.0	–	6.0
58.4–63.8	1.0	4.5	0.9	–	1.5	3.6	–	5.4
63.9–70.6	0.9	4.1	0.8	–	1.4	3.3	–	4.9
70.7–79.2	0.8	3.6	0.7	–	1.3	2.9	–	4.3
79.3–90.5	0.7	3.2	0.6	–	1.2	2.6	–	3.8

Source: Reference 2.

* Based on temperature data obtained for a minimum of five years.

† Based on 4.5 times the optimum fluoride level for communities.

larger quantities of water. Fluospar and calcium fluoride are the commercial fluoride compounds usually used in water treatment. Sodium silicofluoride is the most commonly used compound for fluoridation of municipal water supplies, whereas sodium fluoride is used less frequently because of higher costs. Ammonium silicofluoride and hydrofluosilicic acid also can be used for fluoridation. More detail is presented in Chapter 20.

Fluoride Reduction Because fluoride compounds are present in natural environments (13th rank among the elements), a raw-water supply may contain too much fluoride. Excess fluoride concentrations may cause blackening or mottling of teeth. Several chemicals can be used for fluoride reduction, including commercially produced products such as fluorex and fluo-carbon, as well as magnesium oxide and activated alumina. The defluoridation process is discussed in Chapter 20.

pH Adjustment There may be several points in the water treatment process where the pH must be adjusted. Several acids and bases can be used, depending on the final pH desired. Those typically used are listed in Table 24–1.

Iron and Manganese Removal Iron and manganese concentrations above 0.3 mg/L total should be removed from water. Although not harmful to health, they cause staining and taste problems. These constituents are removed through oxidation, by settling and filtration, using chlorine, chlorine dioxide, ozone, or potassium permanganate. Small amounts (below 1 mg/L) can be sequestered by sodium hexametaphosphate. More detail is presented in Chapter 14.

Corrosion Control At the completion of the water treatment process and prior to entering the distribution system, the water must not be corrosive. Corrosive water will cause costly problems in the distribution and storage systems as well as for the individual consumer. Lime, soda ash, sodium hydroxide, sodium hexametaphosphate, and sodium tripolyphosphate are all used to minimize the corrosive property of waters.

Algal Control Algal control in reservoirs is usually accomplished by addition of copper sulfate to the water. This topic is discussed in Chapter 8.

Many chemicals are not compatible with each other, and great care must be taken to avoid the possibility of them coming into contact with each other. When chemicals are not compatible, they should be separated either by locating them in another building, or at least in separate containment areas. Table 24–3 shows frequently used chemicals and their compatibility with one another.

Raw-Water Quality

There are many considerations in selecting the specific chemical to be used in a water treatment process, one important item being the raw-water quality. The raw water must first be analyzed to determine the level and type of treatment required. It is helpful to consider the effectiveness of each chemical under consideration, based on experience and operating results from full-scale plants at other locations with similar raw-water quality. However, caution should be exercised when using data from other plants. The designer should not assume that a particular chemical used at one plant will produce identical results at a different plant. The effectiveness of each chemical varies in different applications, depending upon operating conditions. Laboratory and/or pilot tests can be used to predict the effectiveness of a chemical for a given application. The objectives of laboratory studies usually are: (1) to determine what chemical dosages are needed to obtain the desired results, and (2) to obtain data for the design and operation of a pilot or full-scale facility. After the chemicals that perform unsatisfactorily are eliminated, a preliminary cost comparison can be made for the remaining chemicals to determine which should receive further consideration. Also, it should be recognized that laboratory and pilot tests do not always accurately predict plant-scale dosages and performance.

Availability and Cost

Other important considerations when selecting a chemical are its availability in a particular region, reliability of supply, and cost. There is little advantage in selecting a chemical that satisfies all the requirements of a water treatment process if the chemical is not readily available. Capital costs for handling and feeding various chemicals vary considerably, depending on the characteristics of the chemical to be fed, the form (liquid, solid, gas) in which the chemical is purchased, and the form in which the chemical ultimately is used in the treatment process.

Transportation is a significant cost for some locations. The cost at the point of origin usually is quoted by the manufacturer in cents or dollars per pound, per 100 pounds, or per ton, and varies according to the size of the order. It may be a price ‘‘f.o.b. cars’’ at the point of manufacture or at a regional stock point. When small lots are purchased, the f.o.b. point is important because the manufacturer ships to the regional stock point in bulk and at lower rates in order to give the customer the benefit of this savings. The point of shipment origin should always be clearly stated, because transportation costs on some chemicals may be more than the cost of the chemical, especially if long hauls are involved.

Many manufacturers quote prices ‘‘f.o.b.’’ from a distribution point but also will give the customer information on the expected cost of transportation by rail or truck to the point of usage. Sometimes manufacturers will also quote ‘‘freight allowed,’’

TABLE 24–3. Chemical Compatibility of Compounds Used in Water Treatment

Compound		Alum	PACl	HFA	FSA	$FeCl_3$	H_2SO_4	PAC	Polymers	Silicate	NH_3	NH_4OH	CaOH	Soda Ash	NaOH	$KMnO_4$	Cl_2	ClO_2	O_3	NaOCl	H_2O_2
Aluminum sulfate	Alum			X			X			X	X	X	X	X	X					X	
Polyaluminum chloride	PACl			X			X			X	X	X	X	X	X					X	
Hydrofluoric acid	HFA	X	X			X	X	X	X	X	X	X	X	X	X	X				X	
Hydrofluosilicic acid	FSA							X	X	X	X	X	X	X	X	X				X	
Ferric chloride	$FeCl_3$			X						X	X	X	X	X	X	X	X	X	X	X	X
Sulfuric acid	H_2SO_4	X	X	X						X	X	X	X	X	X	X				X	
Powdered activated carbon	PAC			X	X											X	X	X	X	X	X
Polymers	Polymers			X	X											X	X	X	X	X	X
Sodium silicate	Silicate	X	X	X	X	X	X									X	X			X	
Anhydrous ammonia	NH_3	X	X	X	X	X	X						X	X	X	X	X	X	X	X	X
Ammonium hydroxide	NH_4OH	X	X	X	X	X	X						X	X	X	X	X	X	X	X	X
Calcium hydroxide	CaOH	X	X	X	X	X	X				X	X									X
Sodium carbonate	Soda Ash	X	X	X	X	X	X				X	X									X
Sodium hydroxide	NaOH	X	X	X	X	X	X				X	X									X
Potassium permanganate	$KMnO_4$			X	X	X	X	X	X	X	X	X									X
Chlorine	Cl_2					X		X	X	X	X	X									
Chlorine dioxide	ClO_2					X		X	X		X	X									
Ozone	O_3					X		X	X		X	X									
Sodium hypochlorite	NaOCl	X	X	X	X	X	X	X	X	X	X	X									
Hydrogen peroxide	H_2O_2			X		X	X	X	X		X	X	X	X	X	X					

X = Incompatible.

which means that they will assume the freight charge on the shipment. In some shipments, it is important to compare the cost of shipment by truck to the cost of shipment by rail. While the truck rate may be higher than the rail rate, the material will be taken from the manufacturer's plant and delivered to the door of the plant by truck at no extra cost. On the other hand, by rail, even though the price given is ''f.o.b. your nearest freight station,'' there will be extra costs for handling and hauling of the material from the freight station to the plant. The overall delivered cost calculation must consider these factors.

To avoid or minimize potential problems in obtaining chemicals, suppliers of the specific chemical should be contacted for details on chemical availability before the chemical is selected for use in the treatment plant. It is advisable to consider market trends for water treatment chemicals to anticipate possible chemical shortages or large cost increases.

Several chemicals must be generated on-site: ozone, chlorine dioxide, monochloramine, and peroxone. Sodium hypochlorite can be generated on-site.

Storage Life

The length of time a chemical will retain its full potency limits the amount of chemical to be purchased and delivered at any one time. If the chemical retains its full potency for 6 months, then it would not be economical to purchase it in quantities that last much longer. If potency will last for 1 year, it may be more economical to purchase the chemical in quantities sufficient to last the longer period, because of discounts for large-quantity purchases. When quantities are ordered to last over long periods, there must be adequate storage facilities. Depending on the characteristics of the selected chemical, the cost of such storage facilities may be a significant factor in determining the optimum amount of chemical purchase.

A sufficient supply of chemicals should be on hand to cover the daily operation requirements, plus an additional amount to cover the time between placement of the reorder and receipt of the material.

Compatibility with Existing System

In expanding an existing water treatment plant, important considerations are the type of chemicals currently in use, the type of equipment currently in use, the type of equipment to be used in the expansion, and compatibility with the other treatment processes. The water treatment plant under consideration may have several processes, such as softening or iron and manganese removal, in addition to coagulation and filtration. The selection of the chemical may add one or more of the other processes while still achieving its primary function. Another consideration is the type of control system, which may favor one chemical over another.

Labor Requirements

The level of operation and maintenance labor required to store and feed chemicals is a consideration. It is important to recognize that the labor depends on both the characteristics of the chemical and the form in which it is purchased, stored, and fed. For example, less labor is usually required for a chemical that is purchased and fed in the same form, in contrast to a chemical that is purchased in dry form, dissolved for storage, and later diluted for feeding. Table 24–4 illustrates the operation and maintenance labor for several chemicals commonly used as coagulants and coagulant aids. These labor requirements include unloading, storing, and feeding operations. Unslaked lime requires relatively high labor for slaking and feeding equipment. As Table 24–4 illustrates, there are differences in labor requirements, and these differences should be considered in selecting chemicals. Water plant treatment costs, discussed in Chapter 32, include chemical feeding systems.

DESIGN CONSIDERATIONS

General Considerations

The design of chemical handling systems must take into account the type of chemical to be fed and the form of the chemical. It must also consider the methods of chemical delivery, storage, handling, mixing and feeding, and the conveyance of the chemicals to the final feed points. The design must result in efficient handling of the chemicals from delivery and storage to the application point. The operation should be convenient and easy without placing an excess burden on operating personnel.

The chemical feeding and handling design should be versatile and allow maximum operator flexibility. A sufficient number of feed points should be included in the treatment plant, as well as control features, to allow the operator to vary the chemical dosages to these points. The chemical feed equipment must have enough capacity to provide for an adequate range of chemical addition. There must be enough feeders

TABLE 24-4. Typical Chemical Treatment Labor Requirements

	Capacity		Operation and Maintenance*
Chemical	lb/hr	(kg/h)	Labor, hr/yr
Alum	10	4.54	150
	50	22.7	210
	100	45.4	300
	500	227	800
Lime (slaked)	100	45.4	1,800
	500	227	1,850
	1,000	454	2,100
Lime (unshaked)	100	45.4	2,400
	500	227	2,400
	1,000	454	2,900
Polymer (dry)	0.5	0.23	500
	1.0	0.45	580
	5.0	2.27	750
	10.0	4.54	850
Polymer (liquid)	0.5	0.23	390
	1.0	0.45	400
	5.0	2.27	420
	10.0	4.54	440

*Labor for operation and maintenance of unloading, storing, and feeding facilities.

that the final water quality will protect the health of the consumer even with mechanical failures. The chemical feeders and pumps should operate at no lower than 20 percent of the feed capacity. The chemical feed and handling design also should take into account future expansions of the plant.

The materials chosen for conveyance, storage, and measurement of a chemical must be compatible with the properties of that chemical. Corrosive chemicals must be handled in such a manner as to minimize potential for corrosion.

The conveyance of chemicals from unloading to the application point must be done in separate conduits for each chemical. Slurry-type chemicals, especially lime, should be fed by gravity where practical. If at all possible, locate pneumatic conveyance tubing for chemicals vertically, because horizontal tubing frequently plugs.

Liquid, Solid, or Gaseous Chemical Form

Once a chemical has been selected, it is necessary to decide whether to feed it in a liquid, solid, or gaseous form. Some chemicals are available only in one form; however, many of the chemicals, as shown in Table 24–1, are available in a variety of forms.

There are certain advantages to using the liquid form of chemicals. Liquid chemicals are generally easier to handle during loading, unloading, and feeding. The liquid form also eliminates the dust problem associated with the use of solids. Because the liquid form can be directly fed to the process, the feeding equipment is simplified, as mixers and dissolvers associated with dry chemicals are not needed. Some of the disadvantages of the liquid form are the additional cost of hauling the extra water and the additional storage space required.

The solid form of chemicals generally requires less storage space, may be less expensive, and provides a greater selection of chemicals. The disadvantages of using a solid chemical include: dusting problems, caking and lumping problems during storage and feeding, additional labor, and the need for more equipment to dissolve and feed the chemical.

Chemicals fed in the gaseous form are not so widely available as those fed in the liquid or solid form. The most common gaseous chemicals are chlorine, sulfur dioxide, carbon dioxide, oxygen, and anhydrous ammonia. Ozone and chlorine dioxide are fed as gases but must be generated on-site. When chemicals are available in the gaseous form, that may simplify the feeding and control of the chemical as wall as its storage and handling. With increased safety concerns and resulting regulations and code requirements, means to contain, and treat, a gaseous chemical release must be provided. Even with these facilities, some utilities are electing to feed liquid chemicals.

The selection of the chemical form will be dictated by such factors as cost, local availability, method of feed or control anticipated, compatibility with the existing facilities, and the quantity of the chemical required. The larger the quantity used, the more favorable the solid form of most chemicals becomes. The solid form may be the more economical when large quantities are used because of the high cost of liquid storage. Also, the cost of transporting the water associated with the liquid form becomes a more significant portion of the costs. However, although a liquid chemical may require a greater storage volume and have higher freight costs than the solid, it may still be more economical when used in small quantities, because the feeding equipment is less complex.

Chemical Delivery

Following selection of the chemical form, it is necessary to determine how that chemical will be delivered to the plant. Generally, bulk chemical deliveries are made by truck or rail; however, for some plants located adjacent to waterways, chemicals have been delivered by barge.

Design considerations for truck delivery include: sufficiently wide access roads to the plant to allow adequate traffic movement; the turning radius for a particular type truck for delivery and unloading of the chemical; the height clearances of monorails, underpasses, and so on; the grade of the access road; the type of pavement for the access road; and weight limits on roads and bridges. See reference 3 for useful design information.

In large plants, it may be feasible to obtain chemicals through rail service if the plant is located near rail lines that could be extended to the plant economically. The capital cost required to obtain service may be offset by the savings in operation and maintenance costs, including the cost of chemicals.

The facility design for unloading the chemical depends on the form of the chemical. For chemicals in the liquid form, trucks are unloaded by pumping, air padding, or gravity. The storage tank, unloading lines, and vent lines are designed to accommodate the rate at which the transporter would unload the material. The transporter should be contacted for this information.

Chemicals in the solid form generally are delivered in bulk or bag by truck or rail. Bagged chemicals typically are delivered on pallets. Design considerations for bag unloading include the location of the bag storage with respect to the unloading facility and day tanks, the manner in which the bags are to be conveyed, and the type of facility for unloading the truck. The most common way to unload bag deliveries is to use a loading dock designed so that the entire truck or rail car can be unloaded with a forklift truck, bag conveyor, or hoist. In small plants, hand trucks are sometimes used for individual bag unloading.

Bulk chemicals delivered by truck or rail can be unloaded in a variety of ways. The most common method of unloading bulk carriers is by pneumatic conveyance. The storage silos must be designed to allow for pneumatic conveyance when this type of delivery is anticipated. Dust control on the silo vent must be sized for off-loading rate. Figure 24–1 illustrates a typical pneumatic conveying system. Some bulk carriers can be unloaded by gravity means, such as a chute. Other methods use mechanical means, such as bucket elevators and belt or screw conveyors. Mechanical systems generally are not satisfactory for fine material because of the dust emitted into the surrounding environment. Each type of conveyance requires particular design considerations for the unloading and storage facilities. The bulk carrier should be consulted prior to design to determine all pertinent parameters.

Chemicals in the gaseous form are usually delivered in ton cylinders, 150-pound (68.1-kg) cylinders, or large bulk truck or rail carriers. A hoist with a monorail is used to unload cylinder-type containers. Special facilities must be designed for bulk truck gas tanks and rail gas tanks so the gas can be transferred from the tanker. Figure 24–2 illustrates a bulk chlorine storage facility.[4]

Chemical Storage

Location Chemical storage facilities should be designed to make operations as convenient and efficient as possible. The location of the storage relative to its unloading

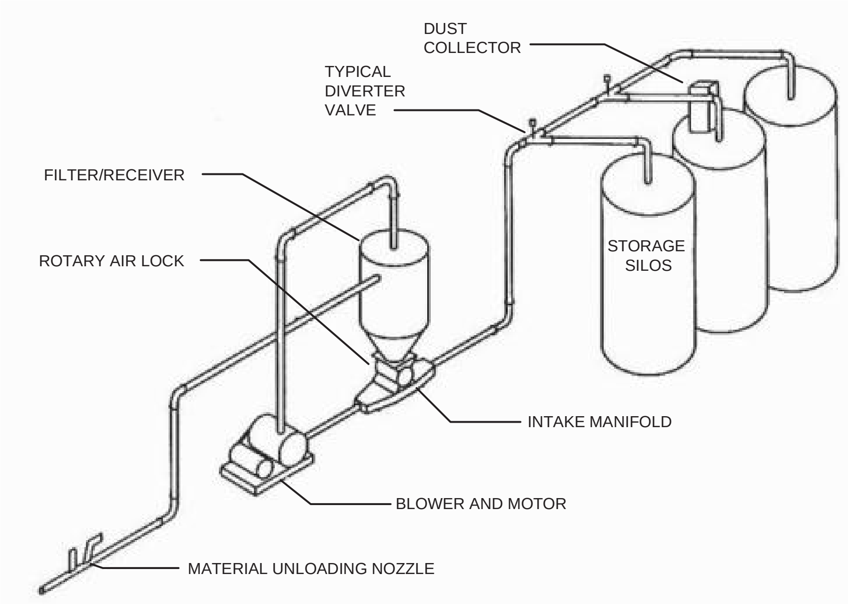


Fig. 24-1. Typical positive-negative pneumatic conveying system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

point, the feeding equipment, and the application point is important. For example, the use of hand and forklift trucks should be maximized, and the need to carry bag materials upstairs or lift heavy loads by operators should be eliminated. There should be adequate access around all storage space. Storage of stacked chemicals in bags and pallets should be placed in such a way that pallet trucks or hand trucks can be maneuvered very easily and efficiently.

The location of chemical storage must also take into consideration the effects of thermal and moisture changes. Liquid chemicals should be stored in an area of adequate temperature to prevent crystallization. Temperature of the space for dry chemical storage is also important to prevent condensation and to allow for proper working conditions in handling the material. Dry chemical forms should be protected from moisture. Consideration of dust control must also be given to storage of dry chemicals. It is desirable to keep dust from accumulating in the air for several reasons: to prevent dust contamination in the atmosphere; to prevent dust in the working area from being introduced into the chemical; and to keep accumulations well below explosive levels.

Chemicals in the gaseous form should be located in areas where thermal protection is provided. Because withdrawal of gas from containers is a function of temperature, heated space is required to ensure an adequate supply of gaseous chemical. The maximum withdrawal rate of chlorine gas, at 65°F, is 40 lb/day from a 150-lb cylinder, and 400 lb/day from a ton cylinder.

Sodium hypochlorite solutions are temperature sensitive; the higher the temperature, the faster the degradation of hypochlorite. Sodium hypochlorite solutions are also light sensitive.

All storage facilities for chemicals should be adequately ventilated, not only for workers' safety but also to maintain the proper thermal and moisture protection for

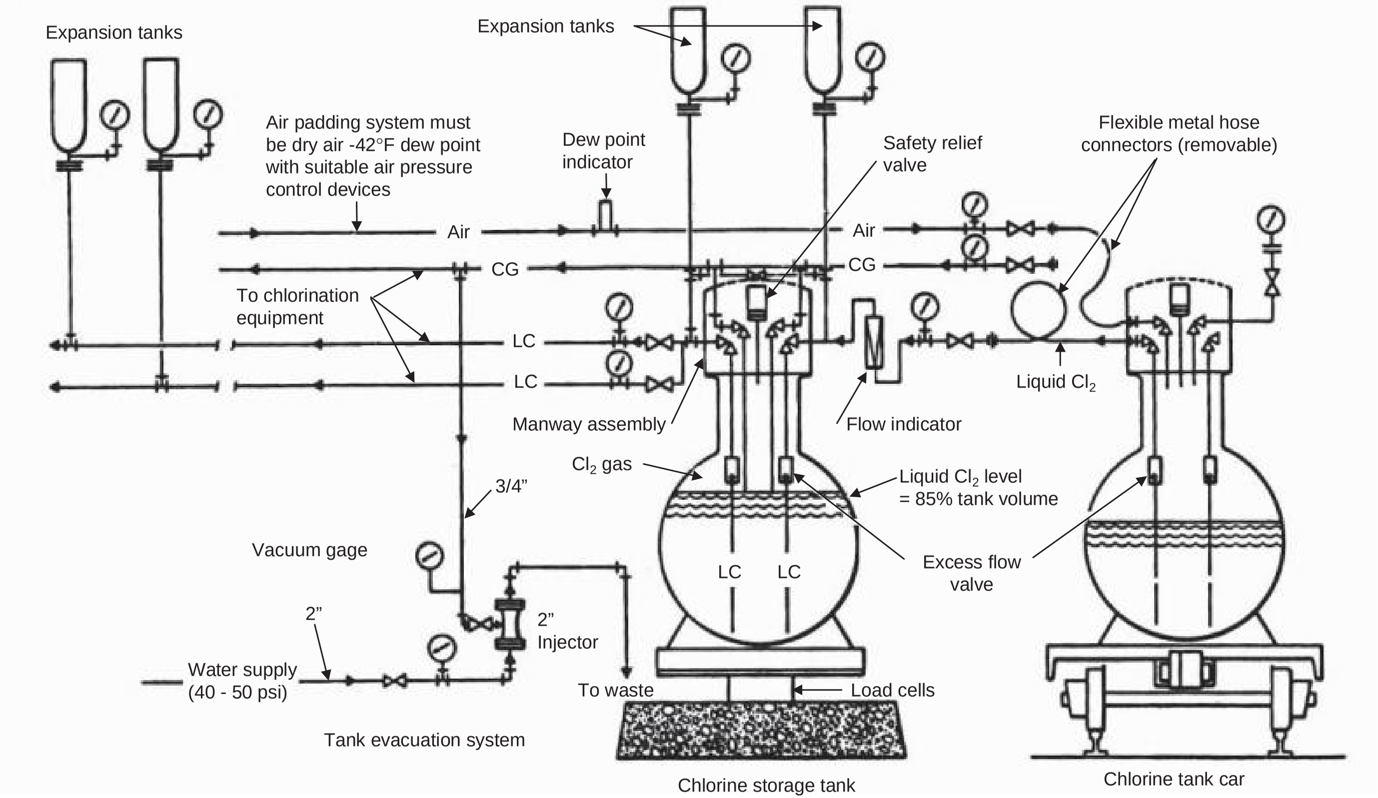


Fig. 24-2. Bulk chlorine storage and tank car and unloading system (From White, George C., *Handbook of Chlorination, and Alternative Disinfectants*, 3d ed. Copyright © 1992 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

the chemicals. Consideration also must be given to the density of gases when working with gases and volatile chemicals. The density of the chemical gases in relation to air will dictate the location of ventilation facilities. For example, chlorine gas is heavier than air, and so the ventilation intake is close to the floor of the room housing the chemical.

Size The size of chemical storage facilities should be based on cost-effectiveness, amount of chemical to be used per day, and ease of operation. Generally, storage for chemicals should contain a minimum of a 30-day supply to allow for chemical delivery, ordering, and any contingencies, or one and one-half times the bulk transport capacity, whichever is greater. The economics of bulk chemical delivery may also dictate storage size, although the quantity to be used on a daily basis may be small. The economics of sizing considerations includes: labor requirements for accepting delivery of the chemical, the actual cost of the physical storage space for the chemical, the cost of bulk versus bag delivery for a dry chemical, and the reliability of delivery. With larger plants it may be desirable to have a large amount of storage for the primary chemicals, with the ability to transfer the chemicals to day tanks.

Miscellaneous Considerations Storage sizing should be based on the useful life of the chemical, as discussed earlier. Appurtenances that are included in the design of storage facilities include: mixing, vibration, or aeration to prevent arching and aid flow of dry chemicals; high- and low-level indicators; access hatches; and pressure and vacuum control devices. For chemicals in solution, mixing can be accomplished with pumping or conventional mixers. Level indicators can be in the form of sight devices, tank level gauges, or electronic sensors.

Safety Considerations Workers' safety must be considered in designing facilities for handling and storing chemicals. They should be stored in such a manner that chemicals accidentally spilled or leaked do not interact with other chemicals. Areas in which acids are stored should be separated from storage areas for bases; liquids should be separated from dry forms of chemicals. Should a spill or accident occur, the contamination should be contained by use of containment walls or separate storage rooms. The accumulation of dusts and gases must also be controlled. Because of its hazardous nature and widespread use, the safety of chlorine has received a great deal of attention.[4,5,6]

Chemical handling also requires safety considerations. Worker contact with chemicals should be minimized by use of machinery. If it is necessary for the workers to handle the chemicals, they should be protected from any contact with them. Consideration should be given to special clothing, emergency showers, and eyewashes. In the design of facilities for bag loading and unloading, the size of the bags and the manner in which they must be handled should be considered. Lifting of 100-lb (45.4-kg) bags higher than the waist should be avoided.

Bulk Storage. A typical bulk storage tank or bin for dry or solid chemicals is shown in Figure 24–3. Dust collectors should be provided on manually and pneumatically filled bins. The material of construction and the required slope on the bin outlet vary with the type of chemical stored. In addition, some dry chemicals such as lime must be stored in airtight bins to keep moisture out. Dry air may be injected into the silo to keep positive pressure in the silo, and keep moisture away from the chemicals.

Bulk storage bins should have a discharge bin gate so feeding equipment can be isolated for servicing. The bin gate should be followed by a flexible connection and a transition chute or hopper that acts as a conditioning chamber over the feeder.

Liquid storage tanks should be sized according to maximum feed rate, shipping time required, and quantity of shipment. The total storage capacity should be one and one-half times the largest anticipated shipment, and should provide at least a 30-day supply of the chemical at the design average dose. Storage tanks for most liquid chemicals may be located inside or outside. However, outdoor tanks usually must be insulated and/or heated to prevent crystallization. Storage tanks for some liquids, such as liquid caustic soda, should be provided with an air vent for gravity flow. Recirculation pumping systems frequently are used to prevent crystallization.

Liquid storage tanks can be located either at ground level or above ground level, depending upon whether gravity feed or pressure feed is desired at the point of application. Figure 24–4 shows two common liquid feed systems, one with overhead storage and one with ground storage. Overhead storage can be used to gravity-feed the rotary wheel liquid feeder, as shown in (a). A centrifugal transfer pump may also be used, but requires an excess recirculation line to the storage tank, as shown in (b). The rotary wheel liquid feeder or rotameter often is used for gravity feed and the metering pump for pressure feed systems.

Bag and Drum Storage In general, bags or drums should be stored in a dry, cool, low-humidity area and used in proper rotation (i.e., first in, first out). Bag- or drum-loaded hoppers should have storage capacity for 8 hours at the nominal maximum feed rate so personnel are not required to fill or change the hopper more than once a shift. Bulk bags (up to 2 tons) are available for some dry chemicals. Totes (up to about 500 gallons) are available for some liquid chemicals, and can be moved using a forklift.

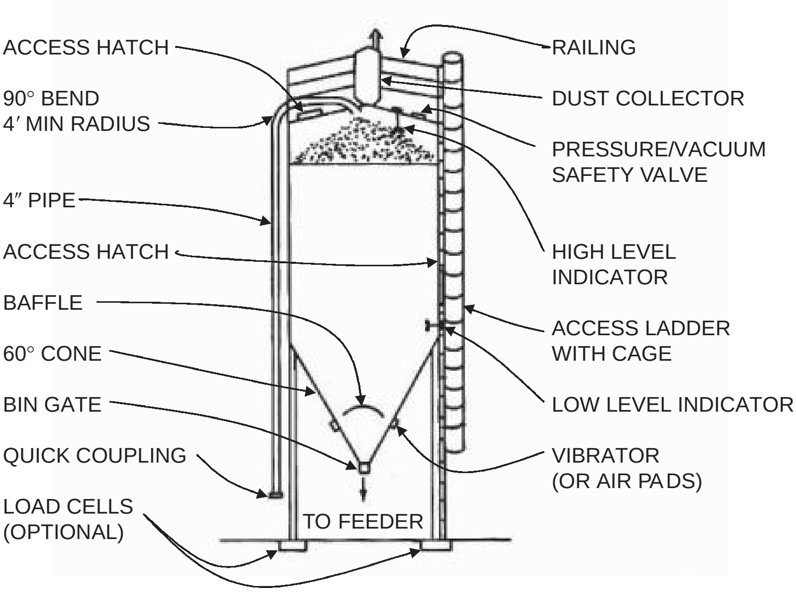
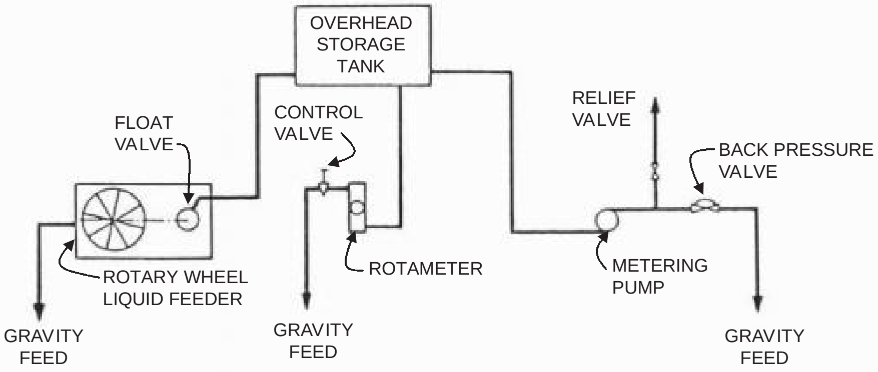
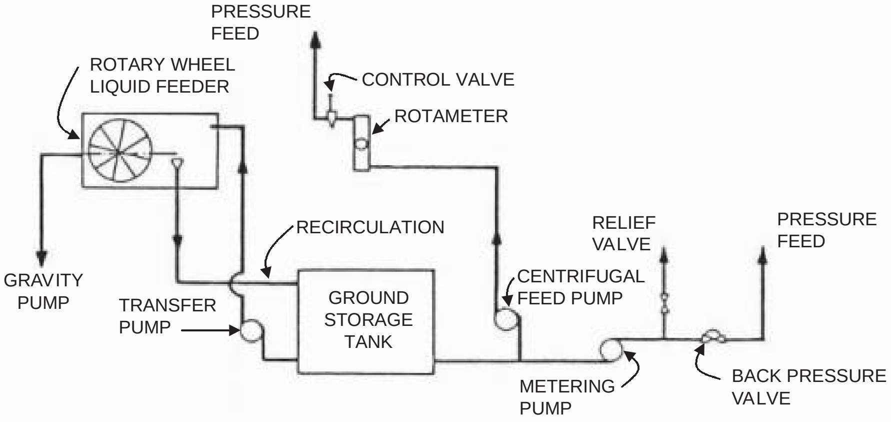


Fig. 24-3. Typical bulk storage tank for solid chemicals



(a) Alternative liquid feed system for overhead storage.



(b) Alternative liquid feed system for ground storage.

Fig. 24-4. Liquid storage and feed systems (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems*. Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Cylinder and Ton Container Storage Whether in storage or in use, cylinders should not be permitted to stand unsupported. They should be chained to a fixed wall or support, and in such a manner as to permit ready access and removal. Ton containers should be stored horizontally, slightly elevated from ground or floor level, and blocked to prevent rolling. A convenient storage rack is obtained by supporting both ends of containers on rails of I-beams. Ton containers should not be stacked or racked more than one high unless special provision is made for easy access and removal. Chlorine cylinders and containers should be protected from impact, and handling should be kept to a minimum. Full and empty cylinders and ton containers should be stored separately. Figure 24–5 illustrates chlorine storage and handling for ton containers.[4]

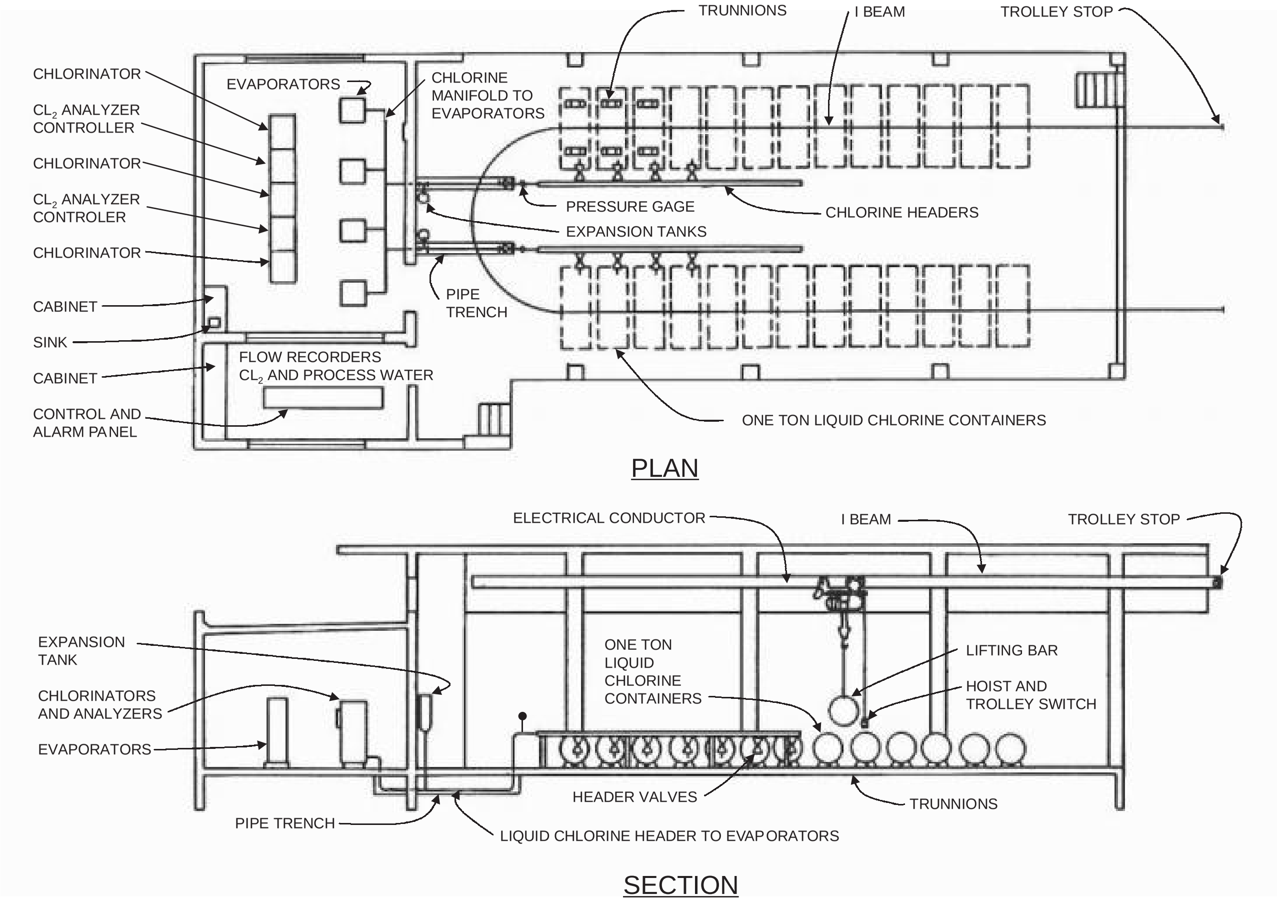


Fig. 24-5. Chlorine storage and handling facilities (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Storage areas should be clean, cool, well ventilated, and protected from corrosive vapors and dampness. Cylinders and ton containers stored indoors should be in a fire-resistant building, away from heat sources (such as radiators, steam pipes, etc.), flammable substances, and other compressed gases. Subsurface storage areas should be avoided, especially for chlorine and sulfur dioxide. If natural ventilation is inadequate, as would be the case for chlorine or sulfur dioxide, storage and use areas should be equipped with suitable mechanical ventilators. Cylinders and ton containers stored outdoors should be shielded from direct sunlight and protected from accumulations of rain, ice, and snow.

All storage, handling, and use areas should be of such design that personnel can quickly escape in emergencies. It is generally desirable to provide at least two ways to exit. Doors should open out and lead to outside galleries or platforms, fire escapes, or other unobstructed areas. Storage areas for ton cylinders should include a means for moving the cylinders to active use, and loading and unloading the delivery vehicle. Typically this is accomplished using a monorail system with a hoist. For larger facilities, a bridge crane may be more appropriate.

Feeding Systems

Feeding systems for chemicals involve conveying the chemicals from storage to the application point(s), and include pumps, conveyors, dry or liquid chemical feeders, eductors, and vacuum and pressure gas systems. This section discusses liquid chemical feeding, dry or solid chemical feeders, and gas feeding systems.

It is sometimes preferable to plan the feeding equipment for the first few years of flow in the plant and replace or add on equipment in later years. This will allow a more accurate feed range to be selected. The capacity of the feeding equipment must meet the maximum dosage required on a maximum day demand, and it must be able to feed that dosage while maintaining reserve units. It is generally accepted practice to install 50 percent more than the maximum dose. For chemicals that are of primary importance in the plant, such as coagulants and disinfectants, backup units or reserve units should be provided for periods of time when the feeding equipment may be out of service. The feeding equipment's construction materials should be compatible with the chemicals that may be used.

Feeding systems should also be designed so that an accurate inventory of chemicals can be maintained at all times. The feed rate should be checked often, as well as the total amount of chemical fed. The change in tank level should be recorded daily and possibly at more frequent intervals, depending upon feed rate and the cost of the chemical.

Chemical feeder control can be manual, automatically proportioned to flow, dependent on some form of process feedback, or a combination of any two of these methods. More sophisticated control systems are feasible, if proper sensors are available. If manual control systems are specified with the possibility of future automation, the feeders selected should be able to be converted with a minimum of expense. An example would be a feeder with an external motor that could easily be replaced with a variable-speed motor or drive when automation is installed.

Standby or backup units should be included for each type of feeder used. Points of chemical addition and piping to them should be capable of handling all possible changes in dosing patterns in order to have proper flexibility of operation. Designed flexibility in hoppers, tanks, chemical feeders, and solution lines is the key to maximum benefits at least cost. More than one feed point should be provided for each chemical so that the operator can try different combinations of chemicals at the various feed points to optimize the chemical dosages.

Liquid Chemical Feeding The feeding systems for liquid chemicals generally are simple, requiring only one or more metering pumps. Liquid chemicals also may be diluted and then pumped, or used with eductors or other hydraulically controlled devices. A typical solution feed system is shown in Figure 24–6, and consists of a storage tank, transfer pump(s), a day tank, and liquid feeder(s). Dilution water is usually added to the solution feed pump discharge line after the chemical is metered, to prevent plugging of the chemical line due to crystallization, and to help mix the chemical with

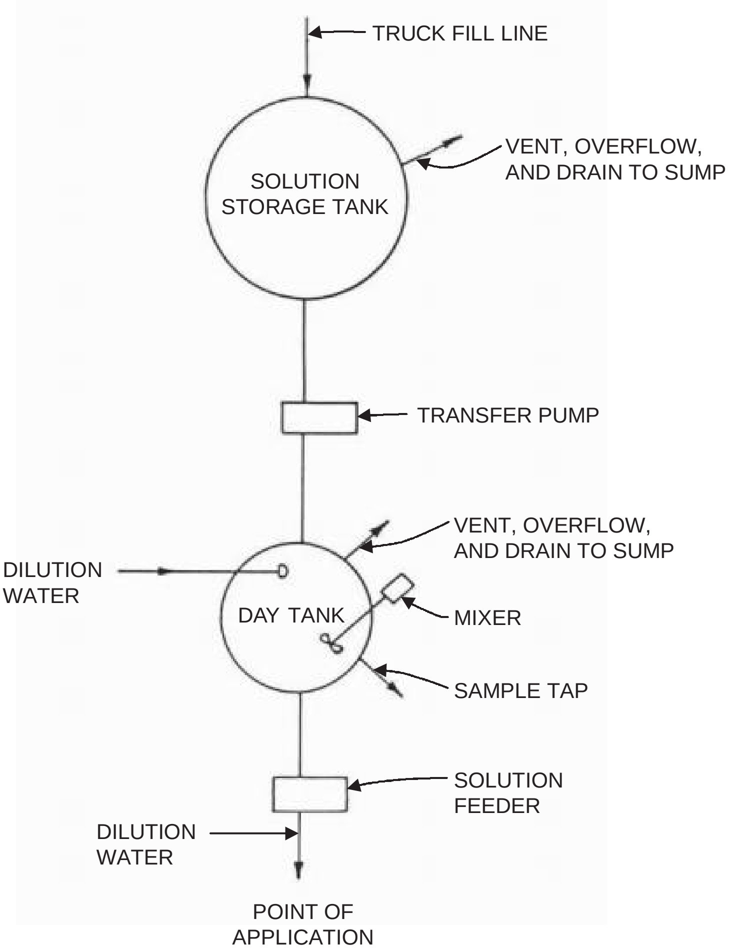


Fig. 24-6. Typical liquid chemical solution feed system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

the water being treated. In some cases, dilution with unsoftened water increases the potential for scaling if the pH is elevated.

Liquid feed systems typically are recommended for use:

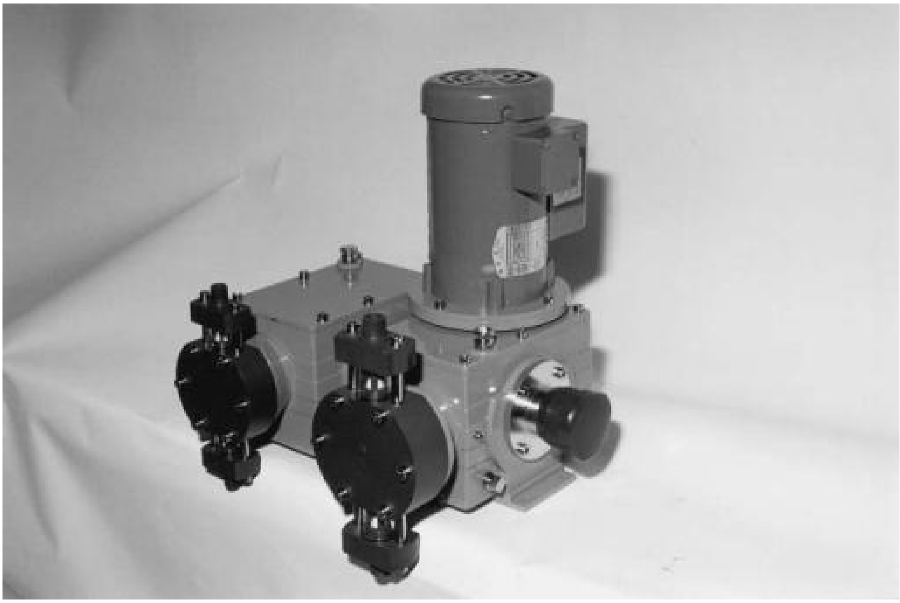
- When low chemical quantities are required
- With less stable chemicals
- With chemicals that are fed more easily as a liquid
- Where handling of dusty chemicals or dangerous chemicals is undesirable
- With chemicals available only as liquids

Liquid feeders usually are metering pumps or orifices. These metering pumps may be positive displacement, plunger, or diaphragm-type pumps. Examples of plunger and diaphragm pumps are given in Figure 24–7, and pictures of two styles of pump are included in Figures 24–7a and 24–7b. Positive displacement pumps can be set to feed over a wide range by adjusting the pump stroke length. Diaphragm metering pumps include mechanical, hydraulic, and solenoid-activated type units. Solution piping systems must have pulsation dampers when using plunger or diaphragm metering pumps. Piping systems associated with these types of pumps should include pressure relief arrangements to protect against over pressurization. In some cases, control valves and rotameters may be all that is needed, while in other cases the rotating dipper-type feeder may be satisfactory. For uses such as lime slurry feeding, however, centrifugal pumps with open impellers are employed. The type of liquid feeder used depends on the viscosity, corrosivity, solubility, suction and discharge heads, and internal pressure relief requirements of the chemical. Chemical metering can also be accomplished using progressive cavity- and peristaltic-type pumps.

Another system that has grown in popularity is the automatic polymer blending unit, which mixes polymer and water to attain the correct concentration of the feed solution. These units rely on the pressure of the feed water to provide the mixing and the hydrodynamic shear energy to disperse the polymer particles and allow them to hydrate fully. These units have been used successfully in numerous locations around the country. Typical units are shown in Figure 24–8.

Solid Chemical Feeding There are a number of dry or solid chemical feeding systems available that involve direct feed of solutions or slurries. The solutions or slurries generally can be fed using pumping equipment, eductors, and other hydraulic control devices as described for liquid systems. The solutions and slurries typically are not so easy to handle as the liquid forms because they tend to form scales and precipitates. Proper solution strength and proper mixing of the dry material must be achieved prior to pumping. The proper solution strength is attained through accurate measurement of the chemical by chemical feeders. Chemical feeders must accommodate the minimum and maximum feeding rates required. Manually controlled feeders have a common range of 10:1, but this range can be increased to about 20:1 or 30:1 with dual-control systems.

Solids characteristics vary considerably, and the selection of a feeder must be made carefully, particularly in a smaller-sized facility where a single feeder may be used for more than one chemical. In general, provisions should be made to keep all dry chemicals cool and dry. Dryness is important, as hygroscopic (water-absorbing) chemicals may become lumpy, viscous, or even rock hard; other chemicals that absorb water less



(a)

(b)

Fig. 24-7. Positive-displacement pumps: (a) Single-head solution metering pump (Courtesy of USFilter/Wallace and Tiernan); (b) Diaphragm chemical metering pump (Courtesy of USFilter/Wallace and Tiernan)

Fig. 24-8. Automatic polymer blending unit (Courtesy of USFilter/Wallace and Tiernan)

readily may become sticky from moisture on the particulate surfaces, causing increased arching in hoppers. In either case, moisture will affect the density of the chemical and may result in incorrect feed rates. Also, the effectiveness of dry chemicals, particularly polymers, may be reduced. Dust-removal equipment should be used at shoveling location, bucket elevators, hoppers, and feeders for neatness, corrosion prevention, and safety reasons. In general, only limited quantities of chemical solutions should be made from dry chemicals, because the shelf life of mixed chemicals (especially polymers) may be short.

Dry Chemical Feeders The simplest method of feeding dry or solid chemicals to a mixing tank is by hand. Solid chemicals may be preweighed and added or poured by the bagful into a dissolving tank. This method is generally limited to very small operations, however, and dry chemical feed equipment is required in larger installations.

A dry feed installation is shown schematically in Figure 24–9, and consists of a storage bin and/or hopper, a feeder, and a dissolver tank. Dry feeders are either of the volumetric of the gravimetric type. Volumetric feeders usually are used only where low feed rates are required. These feeders deliver a constant, preset amount of chemical and do not recognize changes in material density. This type of feeder must be calibrated by trial and error at the outset, and then readjusted periodically if the material changes in density.

The gravimetric feeder delivers chemicals based on the required weight per unit volume. Typically, the volumetric feeders are less expensive than the gravimetric units.

Most types of volumetric feeders generally fall into the positive displacement category. All designs of this type use some form of moving cavity of a specific or variable

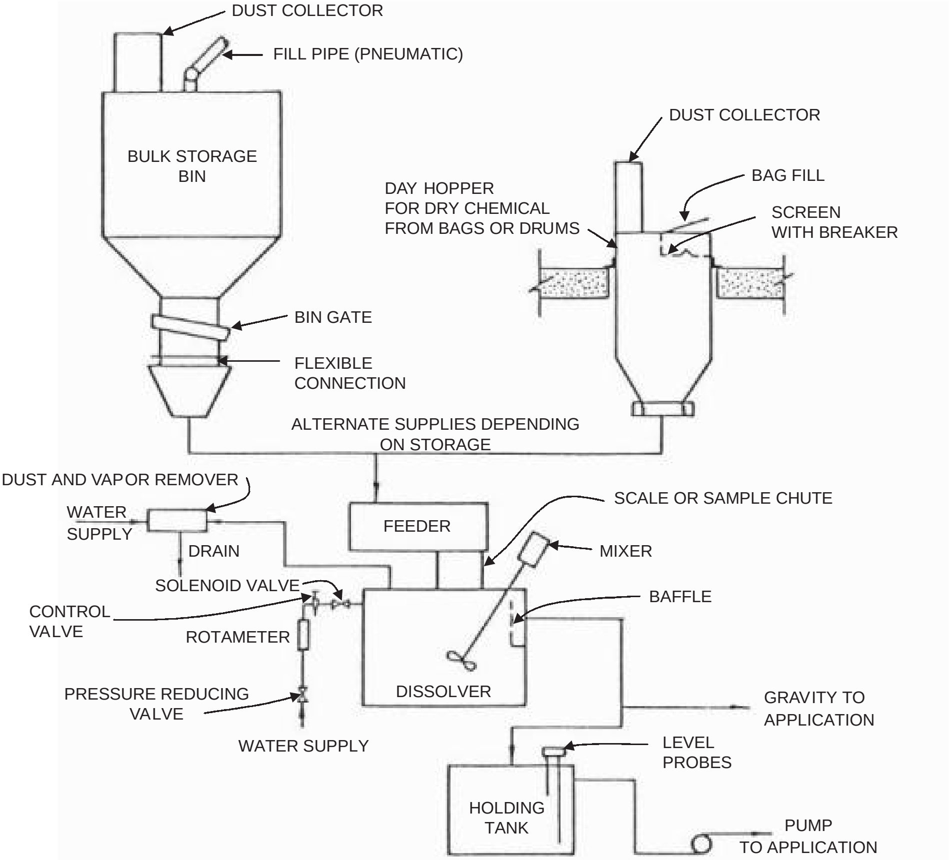


Fig. 24-9. Typical solid or dry chemical feed system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

size. In operation, the chemical falls by gravity into the cavity, and it is almost fully enclosed and separated from the hopper's feed. The rate at which the cavity moves and is discharged, together with the cavity size, governs the amount of chemical fed. Positive-displacement feeders often use air injection to enhance flowability of the material.

Rotary Paddle Feeder. A rotary paddle feeder is especially effective for fine materials that tend to flood. The paddle or vane is located beneath the hopper discharge, with the feed being varied by means of a sliding gate and/or variable-speed drive. The feed rate can be varied easily by adjusting the variable speed drive on vane shaft. A variant of the rotary paddle feeder is the pocket feeder, also called the star or revolving door feeder, in which the paddle is tightly housed to permit delivery against vacuum or pressure. Figure 24–10 illustrates this feeder type.

Oscillating Hopper Feeder. Another type of volumetric feeder is the oscillating hopper, or oscillating throat feeder. This feeder consists of a main hopper and an oscillating hopper that swivels on the end of the main hopper. The material completely fills both hoppers and rests on the tray beneath. As the oscillating hopper moves back and forth, the scraper, which rests on the fixed tray below, is moved first to the left and then to the right. As it moves, it pushes a ribbonlike layer of dry chemical off the tray. The capacity is fixed by the length of the stroke, which may be varied by means of a micrometer screw. Further adjustment is possible by changing the clearance between

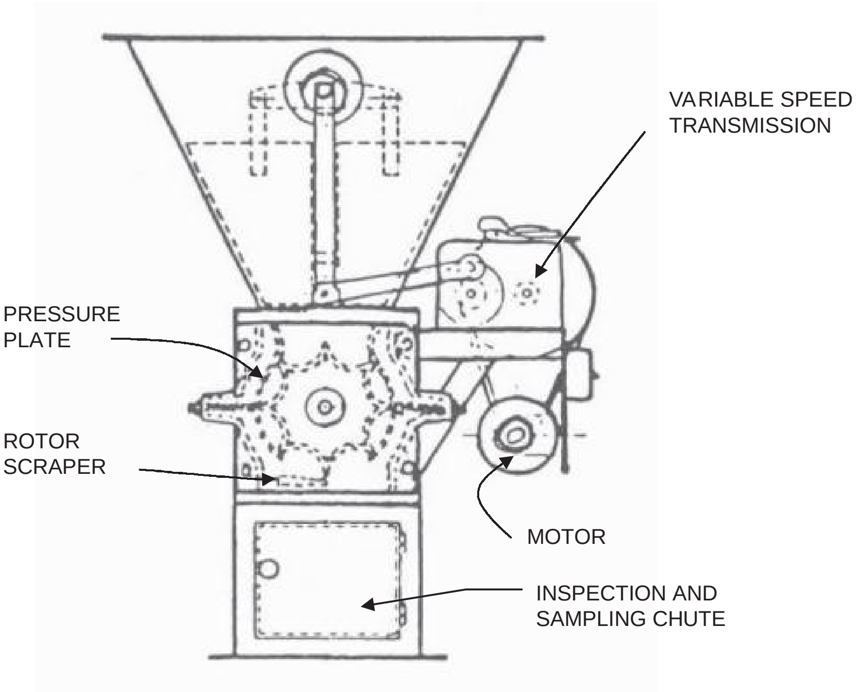


Fig. 24–10. Positive-displacement rotary feeder (*Source:* Reference 7)

the hopper and the fixed tray, which may be raised or lowered. This type of feeder is one of the most widely used in small water plants.

Oscillating Plate Feeder. In the oscillating plate feeder, a plate is mounted below the bottom spout in the storage hopper so that the chemical spills out onto the plate as it comes out of the storage hopper. A leveling bar is mounted above the plate on each of the two ends. The plate is mechanically linked to the drive motor and slowly oscillates from side to side as the feeder operates. The magnitude of oscillation can be adjusted with the mechanical linkage. This provides a dosage adjustment. Each time the plate oscillates from one side to the other, a measured amount of chemical drops off it into the solution tank. The rake bar above each end of the plate helps to regulate the repeatability of the feed rate.

Vibrating Feeder. The vibrating feeder is also a volumetric feeder. With it, motion is obtained by means of an electromagnet anchored to the feeding trough, which in turn is mounted on flexible leaf springs. The magnet, energized by pulsating current, pulls the trough sharply down and back; then the leaf springs return it up and forward to its original position. This action is repeated 3,600 times per minute (when operating on 60-cycle AC), producing a smooth, steady flow of material.

Volumetric Belt Feeder. The volumetric belt feeder uses a continuous belt of specific width moving from under the hopper to the dissolving tank. The material falls on the feed belt from the hopper and passes beneath a vertical gate. For a given belt speed, the position of the gate determines the volume of material passing through the feeder. A volumetric belt feeder is illustrated in Figure 24–11.

Screw-Type Feeder. The volumetric screw-type feeder employs a screw or helix at the bottom of the hopper to transfer dry chemical to the solution chamber, as illustrated in Figure 24–12 and pictured in Figures 24–12a and 24–12b.

The basic drawback of the volumetric feeder is that it cannot compensate for changes in the density of materials and, therefore, is not as accurate as two other types of dry feeders: the gravimetric and loss-in-weight types. For these feeders, the volumetric design is modified to include a gravimetric or loss-in-weight controller, which

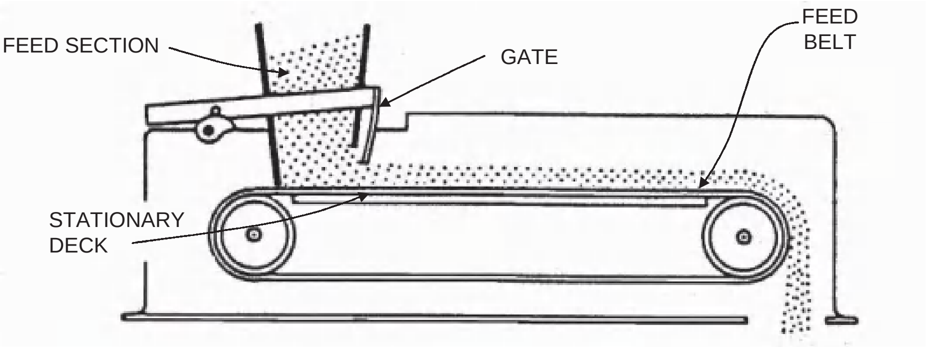
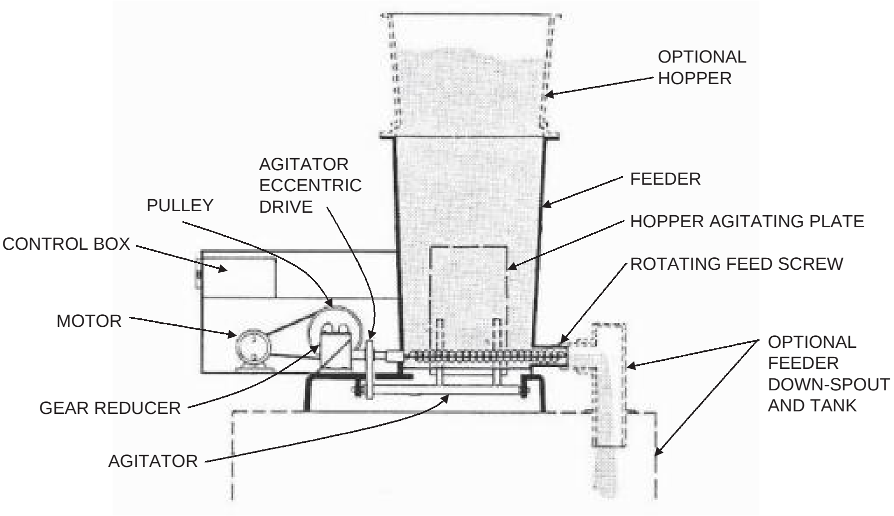
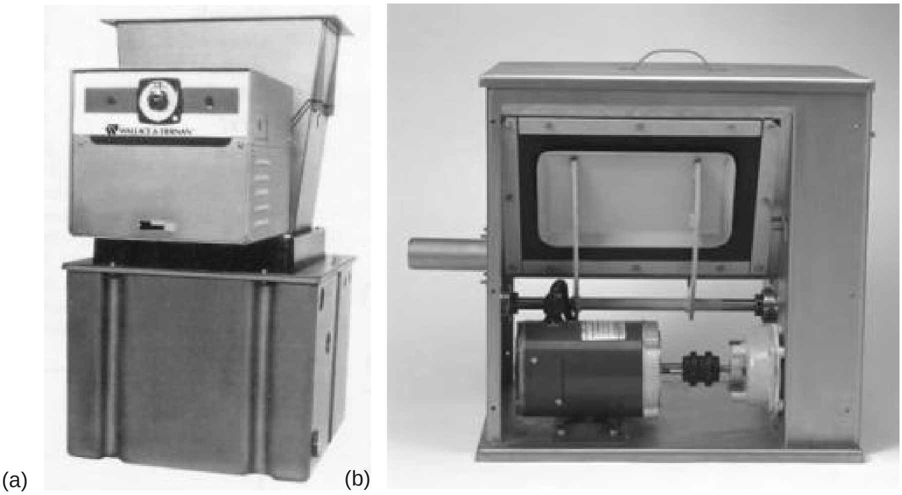


Fig. 24-11. Volumetric belt-type feeder (*Source:* Reference 7)





(a) (b)

Fig. 24-12. Typical helix or screw-type volumetric feeder: (a) Photo of volumetric feeder (Courtesy of USFilter/Wallace and Tiernan); (b) Helical feeder (Courtesy of Penn Process/Omega) (*Source:* Reference 7) allows for weighing of the material as it is fed. Both gravimetric and volumetric feeders can be used to feed in proportion to the flow of wastewater.

Belt-Type Gravimetric Feeder. Belt-type gravimetric feeders have a wide capacity range and usually can be sized for any use in a treatment plant. Belt-type gravimetric feeders use a basic belt feeder with a weighing and control system. Feed rates can be changed by adjusting the weight per foot of belt, the belt speed, or both. Two types of gravimetric belt-type feeders are available: the pivoted belt and the rigid belt-type. The pivoted belt feeder consists of a feed hopper, an endless traveling belt mounted on a pivoted frame, an adjustable weight that counterbalances the load on the belt, and a means of continuously and automatically adjusting the feed of material to the belt. Dry chemical flow to the feeder can be controlled by a gate placed between the feed hopper and the belt or by controlling the amplitude of vibration in a vibrating deck placed between the feed hopper and belt. Figure 24–13a shows a schematic of this feeder, and Figure 24–13b presents a picture of one unit.

The rigid belt feeder is similar to the pivoted belt feeder except for the chemical feed rate adjustment method. The pivoted belt filter is adjusted through action of the belt tilting up and down, while the rigid belt adjustment occurs through action of the scale beam dependent only on the weight of the belt. See Figure 24–14. Good housekeeping and the need for accurate feed rates dictate that the gravimetric feeder be shut down and thoroughly cleaned on a regular basis. Chemical buildup can affect accuracy and can even jam the equipment in some cases.

Loss-in-Weight Feeder. The loss-in-weight feeder should be used where the greatest accuracy or more economical use of chemical is important. This feeder works only for feed rates up to 4,000 lb/hour (1,815 kg/h).

The loss-in-weight feeder has a material hopper and feeder set on enclosed scales. The feed rate controller is used to deliver the dry chemical at the desired rate. See Figure 24–15.

Dissolvers. Dissolvers are also important to dry feed systems because any metered chemical must be wetted and mixed with water to provide a chemical solution free of

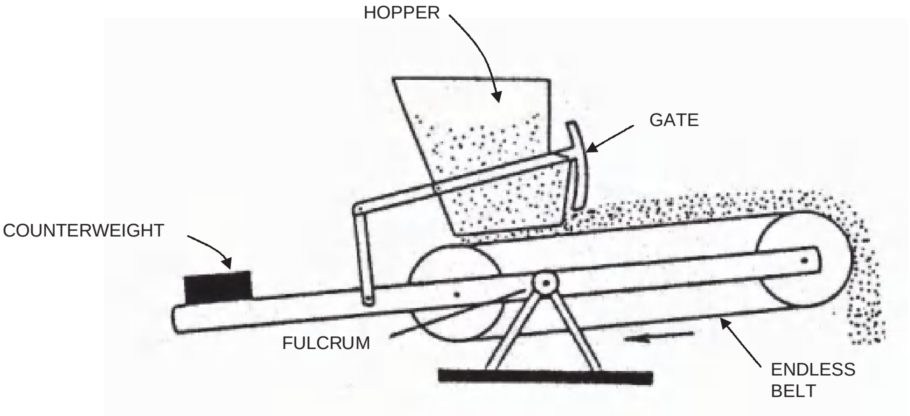


Fig. 24–13a. Pivoted belt gravimetric feeder (*Source:* Reference 7)

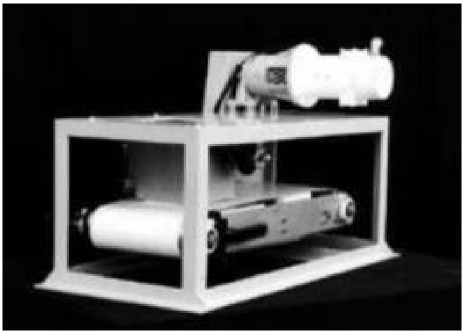


Fig. 24-13b. Gravimetric feeder (Courtesy of Infilco Degremont, Inc.)

lumps and undissolved particles. Most feeders, regardless of type, discharge their material to a small dissolving tank that is equipped with a nozzle system and/or mechanical agitator, depending on the solubility of the chemical being fed. It is important that the surface of each particle become completely wetted before entering the feed tank to ensure complete dispersal and to avoid clumping, settling, or floating.

A dissolver for a dry chemical feeder is unlike a chemical feeder, which by simple adjustment and change of speed can vary its output tenfold. The dissolver must be designed for the job to be done. A dissolver suitable for a rate of 10 lb/hour (4.54 kg/h) may not be suitable for dissolving at a rate of 100 lb/hour (45.4 kg/h).

The capacity of a dissolver is based on detention time, which is directly related to the wettability or rate of solution of the chemical. Therefore, the dissolver must be

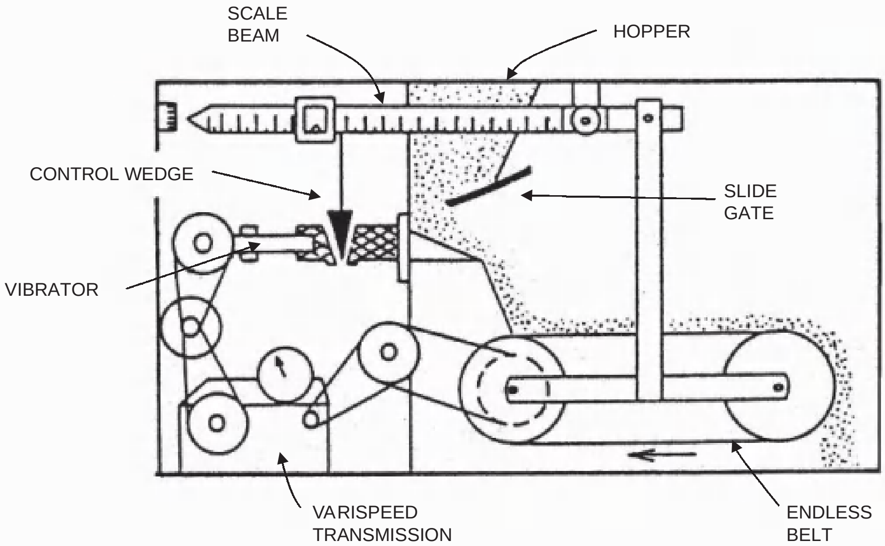


Fig. 24-14. Rigid belt gravimetric feeder (*Source:* Reference 7)

Fig. 24–15. Loss-in-weight feeder (Courtesy of Penn Process/Omega)

large enough to provide the necessary detention for both the chemical and the water at the maximum rate of feed.

Gas Feeding Systems Gas feeding systems are mechanically and operationally simple. Gases can be fed through pressure systems, vacuum systems, or direct feed. Also, they may be diluted and fed as a solution. Therefore, gas feeders can be classified as solution feed or direct feed. Solution feed vacuum-type feeders are commonly used in chlorination and in dechlorination with sulfur dioxide. In chlorination, chlorine gas is metered under vacuum, and it is mixed with water in an injector to produce a chlorine solution. The flow of chlorine gas is automatically shut off on loss of vacuum, stoppage of the solution discharge line, or loss of operating water pressure.

Direct feed or ‘‘dry feed’’ equipment is infrequently used, and only when either water or electricity or both is unavailable at a site. This type of equipment is nearly the same as the solution feed type except that there is no device for making and injecting an aqueous solution. The gas itself is piped directly into the water to be treated. The same equipment is used to control, withdraw, and meter the gas from the containers.

Feed System Requirements for Common Chemicals

Alum Dry alum must be made into a solution before being fed to the plant. Dissolving tanks must be made of a noncorrosive material, and dissolvers should be the right size to obtain the desired solution strength. The most common solution strength is 0.5 pound (0.23 kg) of alum to 1 gallon (0.06 kg/L) of water, or a 6 percent solution. The dissolving tank should be designed for a minimum detention time of 5 minutes at the maximum feed rate.

Dissolvers should have water meters and mixers so that the water/alum mixture can be controlled. Most liquid alum is fed as it is delivered in a standard 50 percent solution.

Alum is usually fed by positive-displacement metering pumps. Normally, dilution water is added to an alum feed pump discharge line to prevent line plugging, and to help mix the alum with the water being treated. The output of the pumps can be controlled automatically by streaming current detectors, or proportional to plant flow.

Carbon Dioxide Stack gases from on-site furnaces, such as recalcination or incineration, have been used as a source of carbon dioxide. Feeding systems for the stack gases include simple valving arrangements for admitting varying quantities of makeup gas to the suction side of constant-volume compressors. Venting of excess gas from the compressor discharge can be valved to the suction side of the compressors. The compressors deliver the stack gases to the point of use.

Pressure generators and submerged gas burners are regulated by valving arrangements on the fuel and air supply. Generation of carbon dioxide is accomplished by the combustion of a fuel (natural gas)-air mixture under water. This system is more difficult to control, requires operator attention, and demands considerable maintenance over the life of the equipment, when compared to liquid CO_2 systems.

Commercial liquid carbon dioxide is used more often because of its high purity, the simplicity and range of feeding equipment, ease of control, and smaller, less expensive piping systems. After vaporization, carbon dioxide with suitable metering and pressure reduction may be fed directly to the point of application as a gas. Metering of directly fed pressurized gas is difficult owing to the high adiabatic expansion characteristics of the gas. Also, direct feed requires extremely fine bubbles to ensure that the gas goes into solution; this in turn can lead to scaling problems. Hence, vacuum-operated, solution-type gas feeders are preferred. Such feeders generally include safety devices and operating controls in a compact panel housing, with construction materials suitable for carbon dioxide service. Absorption of carbon dioxide in the injector water supply approaches 100 percent when a ratio of 1.0 pound (2.2 kg) of gas to 60 gallons (0.002 kg/L) of water is maintained.

Chlorine Elemental chlorine is a poisonous yellow-green gas at ordinary temperature and pressure. The gas is stored as a moisture-free liquid under pressure in specially constructed steel containers, and is vaporized from the liquid form either directly or with heated vaporizers. Chlorine gas feeders may be classified into two types: direct feed or solution feed.

Direct or dry feed gas feeders deliver chlorine gas under pressure directly to the point of application. Direct feed gas chlorinators are less safe than solution feed chlorinators, and are used when there is no adequate water supply available for injector operation. In solution feed vacuum-type feeders, chlorine gas is maintained under vacuum throughout the apparatus. Vacuum is created by water flow through an injector, or by an eductor pump to move the chorine from the supply system through the chlorine gas metering devices to the injector or eductor pump. Chlorine gas is mixed with water in the injector, and the chlorine solution is moved to the point of application In the feeder, the vacuum controls the operation of the chlorine inlet valve so that the chlorine will not feed unless sufficient vacuum is induced through the apparatus. This type of feeder is most common because its safe operation is assured. It employs direct indication metering, and the flow of chlorine is automatically shut off on loss of vacuum, stoppage of the discharge line, or loss of operating water pressure.

Chlorine Dioxide Chlorine dioxide is a greenish-yellow gas that is quite unstable and, under certain conditions, explosive. It cannot be shipped in containers because of its explosive nature; it must be generated at the point of use and applied immediately.

Although readily soluble in water, ClO_2 does not react with water as does chlorine. Chlorine dioxide is easily expelled from aqueous solution by blowing a small amount of air through the solution. Aqueous solutions of ClO_2 are also subject to some photodecomposition.

Chlorine dioxide is generated by oxidizing sodium chlorite with chlorine (either chlorine gas or hypochlorite) at a pH of 4 or less. This means that the injector system of the chlorination assembly must be capable of delivering a chlorine solution strength greater than about 500 mg/l. Because the upper limit of this solution strength should not exceed 3,500 mg/l to prevent breakout of molecular chlorine at the point of application, the effective range of chlorine dioxide production is about 7:1. Chemical feed devices can handle ranges up to 20:1 on a flow proportional basis and 100:1 on a compound loop control system.

Ferric Chloride Ferric chloride is always fed as a liquid, and is normally obtained in liquid form containing 20 to 45 percent $FeCl_3$. When iron salts such as ferric chloride are used for water coagulation in soft waters, a small amount of base (such as sodium hydroxide or lime) is needed to neutralize the acidity of these strong acid salts.

Dilution of ferric chloride solution from its shipping concentration to a weaker feed solution should be avoided, because of a potential to hydrolyze. Ferric chloride solutions may be transferred from underground storage to day tanks with rubber-line self-priming centrifugal pumps having Teflon rotary and stationary seals. Because liquid ferric chloride can stain or leave deposits, glass-tube rotameters are not used for metering. Instead, rotary-wheel feeders and diaphragm metering pumps made of rubber-lined steel and plastic are often used for feeding ferric chloride.

Ferric Sulfate Feed solutions are usually made up at a water-to-chemical ratio of 2:1 to 8:1 (on a weight basis). The usual ratio is 4:1, and the feed solution is made up in a 20-minute detention tank. Care must be taken not to dilute ferric sulfate solutions to less than 1 percent, in order to prevent hydrolysis and deposition of ferric hydroxide.

Dry feeding requirements are similar to those for dry alum except that belt-type feeders are rarely used because of their open type of construction. Closed construction, as found in the volumetric- and loss-in-weight–type feeders, generally exposes a minimum of operating components to the vapor, and thereby minimizes maintenance. A water-jet vapor remover should be provided at the dissolver to protect both the machinery and the operator.

Ferrous Sulfate The granular form of ferrous sulfate has the best feeding characteristics, and gravimetric or volumetric feeding equipment may be used. The optimum chemical-to-water ratio for continuous dissolving is 0.5 lb/gallon, or 6 percent, with a detention time of 5 minutes in the dissolvers. Mechanical agitation should be provided in the dissolver to ensure complete solution.

Hydrogen Peroxide Hydrogen peroxide is obtained from commercial suppliers and is available in 35 percent, 50 percent, and 75 percent strengths. Hydrogen peroxide is supplied in either drums or in bulk by tankcar. It can be stored on-site but deteriorates rapidly if it is contaminated. Storage is typically in polyethylene drums or tanks. Hydrogen peroxide is added to water using metering pumps, and the pump heads should be constructed of hydrogen peroxide–resistant materials. It has a specific gravity of 1.39 for 50 percent solution. Acceptable pipe materials include 316 stainless steel, polyethylene, CPVC, and Teflon. Pipe gaskets should be Teflon.

Lime Although lime comes in many forms, quicklime and hydrated lime are used most often for water coagulation or softening. Quicklime is almost all calcium oxide (70 to 96 percent CaO). High-calcium quicklime contains more than 88 percent CaO and less than 5 percent magnesium oxide (MgO), while dolomitic lime may contain up to 40 percent MgO.

Quicklime (unslaked lime) is almost all CaO and first must be converted to the hydrated form ($Ca(OH)_2$). Hydrated or slaked lime is a powder obtained by adding enough water to quicklime to satisfy its affinity for water. Hydrated lime needs only enough water to form milk of lime**.** Wetting or dissolving tanks usually are designed for 5 minutes' detention with 0.5 lb/gallon (0.06 kg/L) of water or 6 percent slurry at the highest feed rate. Hydrated lime often is used where maximum feed rates are less than 250 lb/hour (110 kg/h).

Dilution is not too important in lime feeding; therefore, it is not necessary to control the amount of water used in feeding. Hydraulic jets may be used for mixing in the wetting chamber of the feeder, but the jets should be the right size for the water supply pressure.

Lime is never fed as a solution because of its low solubility in water. Also, quicklime and hydrated lime usually are not applied dry directly to water for the following reasons:

- They are transported more easily as a slurry.
- A lime slurry mixes better with the water than dry lime.
- Prewetting the lime in the feeder with rapid mixing helps to ensure that all particles are wet and that none settles out in the treatment basin.

Major components of a lime feed system (illustrated in Fig. 24–16) include a storage bin, dry lime feeder, lime slaker, slurry holding tank, and lime slurry feeder. The slurry holding tank is usually needed only when the point of application is at a remote location. Quicklime feeders usually must be the belt or loss-in-weight gravimetric types, because bulk density changes so much. Feed equipment usually has an adjustable feed range of at least 20:1 to match the operating range of the slaker.

There are two basic types of lime slakers: the paste or "pug mill" type (Fig. 24–17) and the detention-type slaker (Fig. 24–18). The paste-type slaker adds water as required to maintain a desired mixing viscosity, so that the viscosity sets the operating retention time of the slaker. The detention-type slaker adds water to maintain a desired ratio with the lime, so that the reaction time is set by the lime feed rate. The detention slaker produces a lime slurry of about 10 percent $Ca(OH)_2$, while the paste type produces a paste of about 36 percent $Ca(OH)_2$. Other differences between the two slakers are that the detention-type slaker operates with a higher water:lime ratio, a lower temperature, and a longer retention time. For either slaker type, vapor removers are required for feeder protection because lime slaking produces heat in hydrating the CaO to $Ca(OH)_{2.}$ Lime slakers are shown in Figure 24–19a and Figure 24–19b.

The required slaking time varies with the source of lime. Fast-slaking limes will complete the reaction in 3 to 5 minutes, but poor-quality limes may require up to 60 minutes and an external source of heat, such as hot water or steam. Before selecting a slaker, it is advisable to determine the slaking time, best initial water temperature, and optimum water:lime ratio for the lime to be used. Procedures for slaking tests have been recommended by the American Water Works Association. More information about lime storage, handling, and use can be found in reference 8.

Polyaluminum Chloride Polyaluminum chloride (PACl) is a clear, pale yellow liquid sometimes used instead of alum for coagulation purposes. It can be stored in drums,

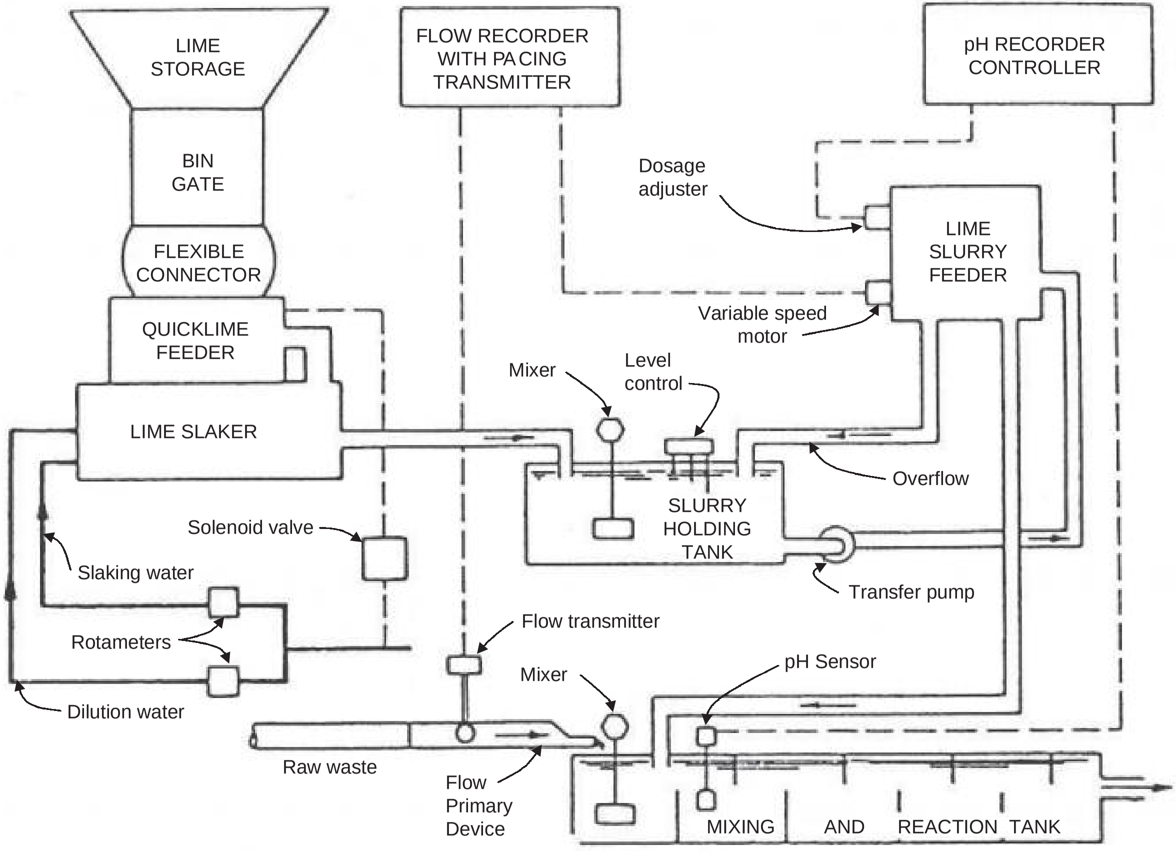


Fig. 24-16. Illustrative lime feed system for water coagulation or softening (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems*. Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

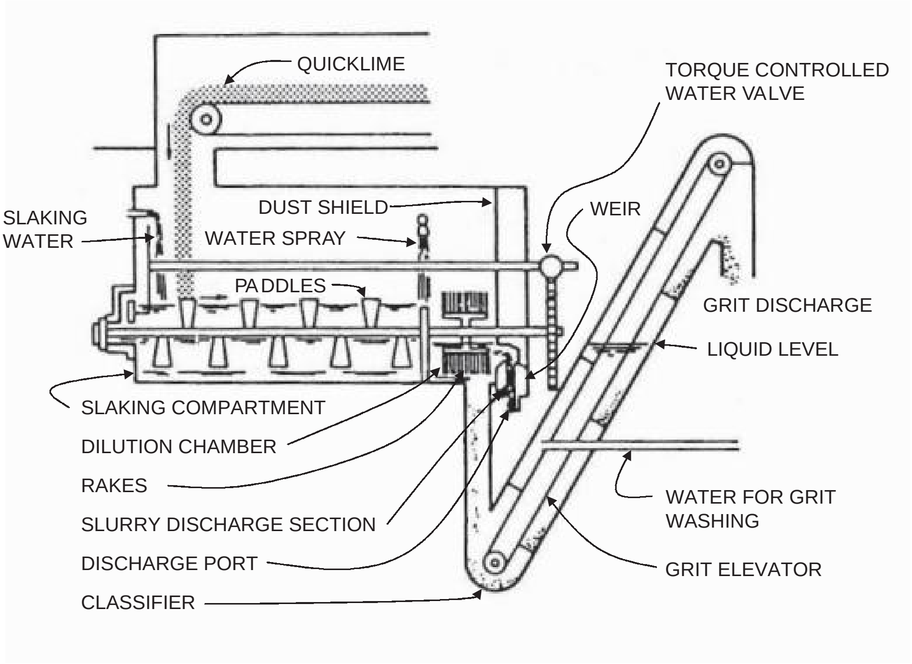


Fig. 24-17. Typical paste-type lime slaker (*Source:* Reference 7)

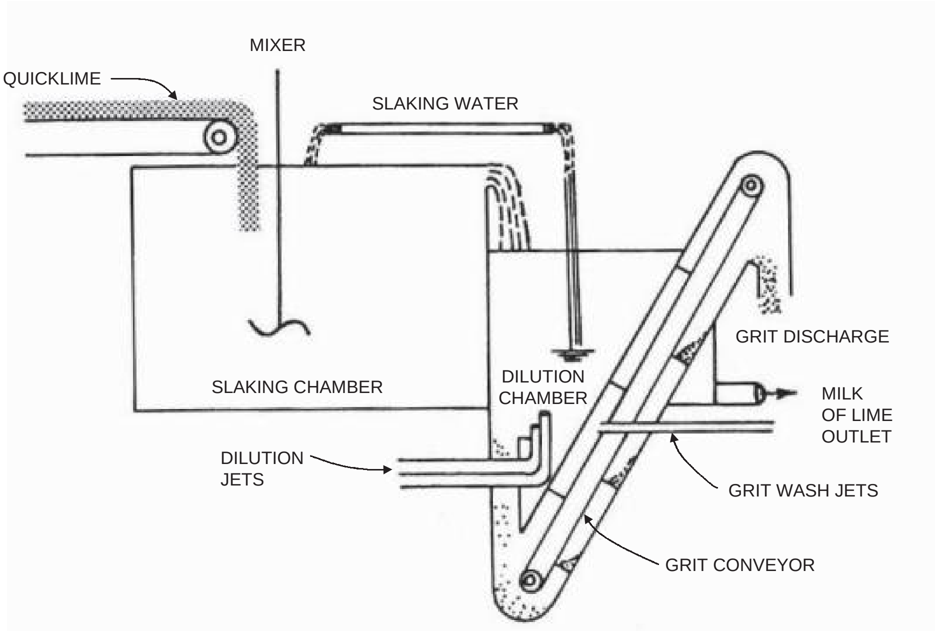


Fig. 24-18. Typical detention-type slaker (*Source:* Reference 7)

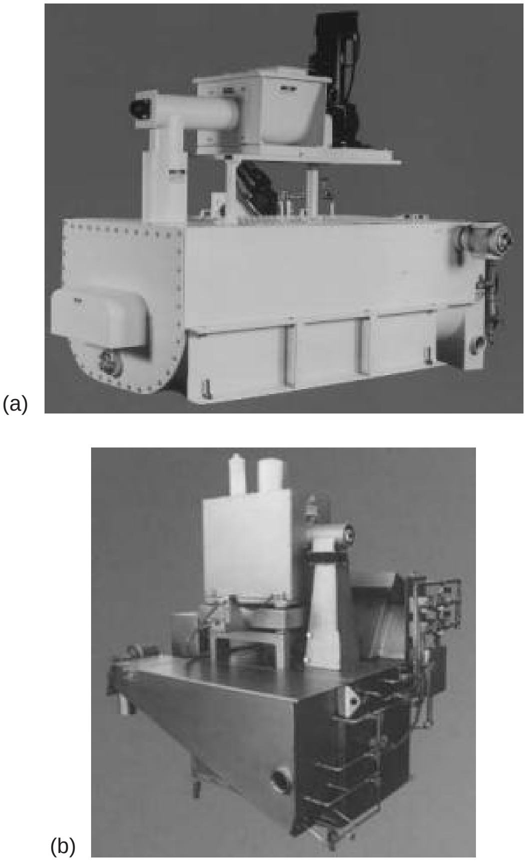


Fig. 24-19. (a) Photo of paste-type lime slaker (Courtesy of USFilter/Wallace and Tiernan); (b) Photo of detention-type slaker (Courtesy of Penn Process/Omega)

portable bins, or bulk tanks. Diaphragm metering pumps are typically used to feed the chemical. Some vendors claim that as much as 30 percent less PACl can be fed compared to alum. The specific gravity of the chemical is in the range of 1.1 to 1.4.

Ozone Ozone is produced commercially by the reaction of an oxygen-containing feed gas in an electrical discharge. The feed gas, which may be air, pure oxygen, or oxygen-enriched air, is passed between electrodes separated by an insulating material. A high voltage of up to 20,000 volts is applied to a high-tension electrode. The ozone molecule, made up of three oxygen atoms, is highly unstable, and is one of the most powerful oxidizing agents known. This is discussed in detail in Chapter 19.

Polymers When dry polymers are used, the polymer and water must be blended and mixed to obtain the desired solution. Initially, complete wetting of the polymer is necessary, using a funnel-type aspirator. After wetting, warm water should be added, with gentle mixing for about 1 hour. Polymer feed solution strengths are usually in the range of 0.1 to 0.75 percent. Stronger solutions are often too viscous to feed. Often the metered solution is diluted just prior to injection to the process to obtain better dispersion at the point of application.

The solution preparation system can include either a manual or an automatic blending system, with the polymer dispensed by hand or by a dry feeder to a wetting jet and then to a mixing aging tank at a controlled rate. The aged polymer solution is transported to a holding tank where metering pumps or rotary-wheel feeders feed the polymer to the process. A schematic of a manual dry polymer feed system is shown in Figure 24–20.

The solution preparation system may be an automatic batching system, as shown in Figure 24–21a and pictured in Figure 24–21b. These systems fill the holding tank with aged polymer, as required by level probes. Such a system is usually provided only at large plants.

Polymer solutions above 1 percent in strength should be avoided, because they are very viscous and difficult to handle. Most powdered polymers are stable when dry, but even in cool, dry conditions, they should not be stored as powders in unopened bags for more than 1 year. Once polymers are dissolved, they may become unstable within 2 to 3 days.

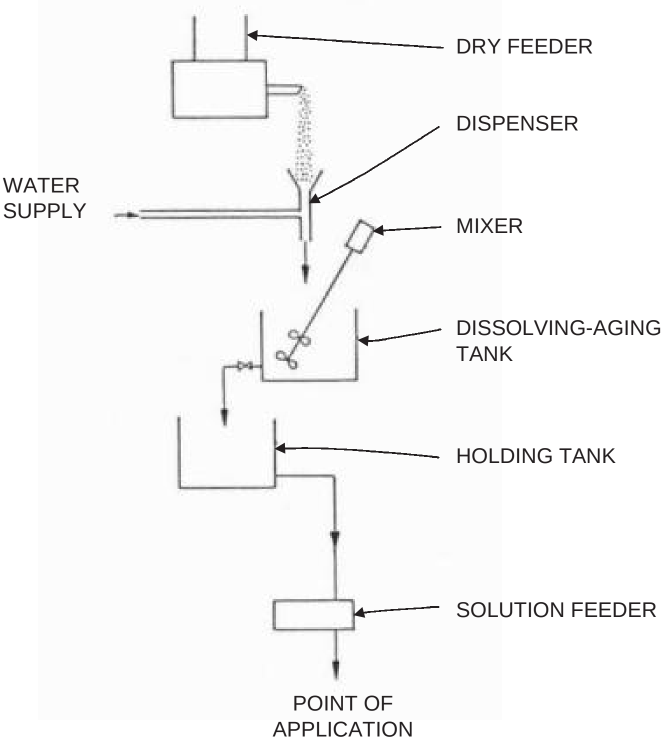


Fig. 24-20. Manual dry polymer feed system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

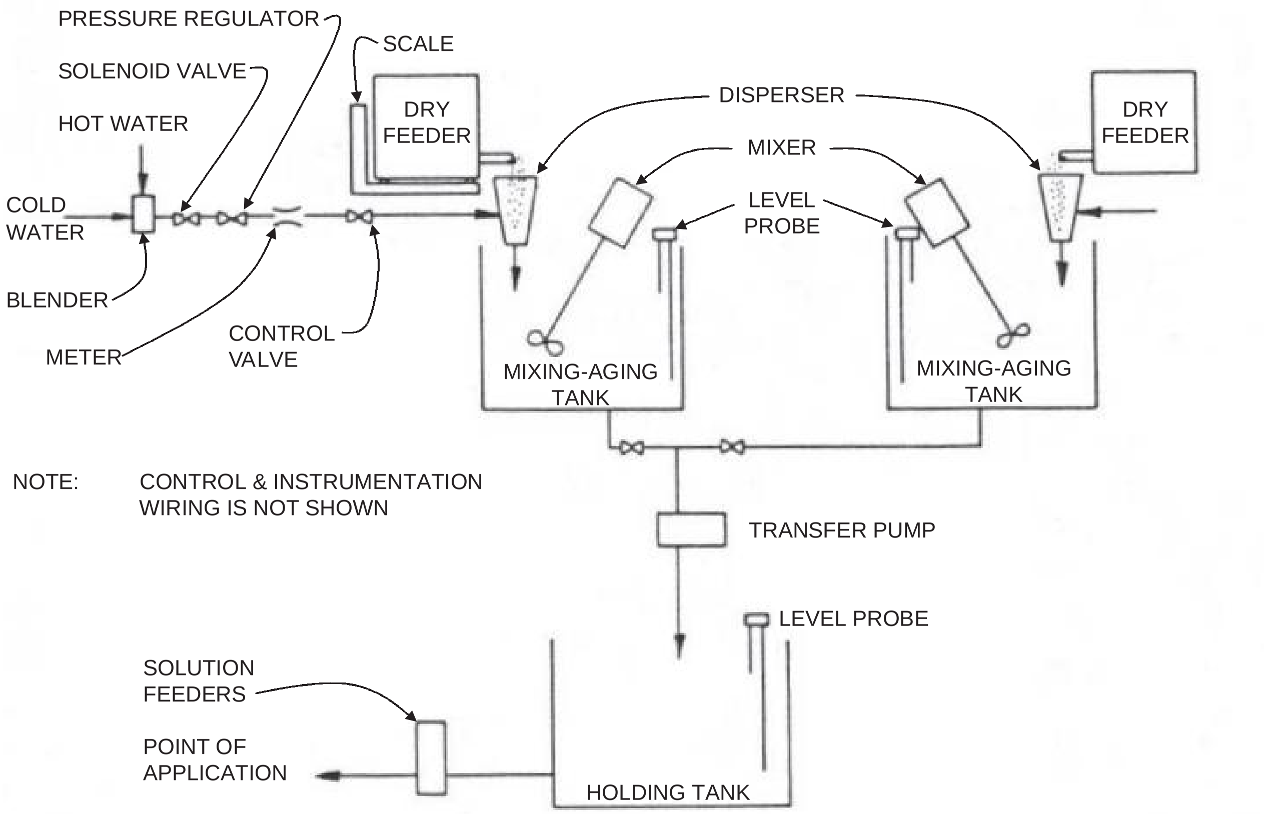


Fig. 24-21a. Automatic dry polymer feed system (From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Fig. 24-21b. Polyelectrolyte batching system (Courtesy of USFilter/Wallace and Tiernan)

Liquid polymers need no aging, and simple dilution is the only requirement for feeding. The dosage of liquid polymers may be accurately controlled by metering pumps or rotary wheel feeders.

Because polymers can cause slippery conditions in a treatment plant, spills should be cleaned up immediately. Other safety precautions should also be observed, as specified by the manufacturer.

Powdered Activated Carbon When powdered activated carbon is used, it is mixed directly with the water, fed as a slurry, and removed by coagulation and settling. The carbon slurry is transported by pumping the mixture at a high velocity to keep the particles from settling and collecting along the bottom of the pipe. The velocity of the slurry should be kept between 3 and 5 feet/sec. At velocities less than 3 feet/sec (0.91 m/s), carbon will settle out in the pipeline; and at velocities greater than 10 feet/sec (3 m/s), excessive carbon abrasion and pipe erosion will occur. At most plants, carbon slurries are fed at a concentration around 10.7 percent or 1 pound carbon/gallon (0.12 kg/1) water; typically, at this concentration, either centrifugal pumps or a combination of centrifugal pumps and eductors are used to transport the carbon slurry. Diaphragm slurry pumps or double-acting positive-displacement pumps are used for transporting higher concentrations. Another transport method used is a pressure pot system in which carbon is loaded into a pressure tank and forced out by pressurizing the vessel. The carbon slurry may be fed using a rotary wheel feeder. PAC can also be fed using an eductor system. The dry PAC is metered into an eductor using a volumetric feeder, and water passing through the eductor causes a vacuum that draws the PAC into the flow stream and conveys it to the point of application. Typically, about 0.4 pounds carbon/gallon of water is used in this application.

Activated carbon is a dusty respiratory irritant, which smolders if ignited. It should be isolated from flammable materials such as rags, chlorine compounds, and all oxidizing agents.

Soda Ash Dense soda ash is generally used in municipal applications because of its superior handling characteristics. It has little dust and good flow characteristics, and will not arch in the bin or flood the feeder. It is relatively hard to dissolve, so ample dissolver capacity must be provided. Normal practice calls for 0.5 pound (0.23 kg) of dense soda ash per gallon of water, or a 6 percent solution retained for 20 minutes in the dissolver. Dissolving of soda ash may be hastened by the use of warm dissolving water. Mechanical or hydraulic jet mixing should be provided in the dissolver.

Sodium Aluminate Dry sodium aluminate is not available in bulk quantities; therefore, small day-type hoppers with manual filling arrangements are used. Dissolvers for the free-flowing grade of sodium aluminate are normally sized for 0.5 pound/gallon (0.06 kg/L), or 6 percent solution strength with a dissolver detention time of 5 minutes at the maximum feed rate. After it is dissolved, agitation should he minimized or eliminated to prevent deterioration of the solution. Solution tanks must be covered to prevent carbonation of the solution.

Liquid sodium aluminate may be fed at shipping strength or diluted to a stable 5 to 10 percent solution. Stable solutions are prepared by direct addition of low-hardness water and mild agitation. Air agitation is not recommended.

Sodium Chlorite Sodium chlorite ($NaClO_2$) for the generation of chlorine dioxide is available as an orange powder or as a solution.

The sodium chlorite pump is sized so as not to exceed a solution strength of 20 percent by weight or 1.66 pound/gallon (0.20 kg/L). Diaphragm pumps rather than piston pumps normally are used for handling the solutions. The solution container for sodium chlorite is sized for at least 1 day’s operation

Sodium chlorite will withstand rough handling if it is free from organic matter. However, in contact with organic materials (clothing, sawdust, brooms), it may ignite. It is sensitive to heat, friction, and impact. These problems are minimized with sodium chlorite solutions.

Sodium Hydroxide Liquid sodium hydroxide, or caustic soda, usually is delivered in bulk shipments, and must then be transferred to storage. The caustic soda often is heated, and is fed by metering pumps as a concentrated solution. Dilution water usually is added after feeding to the pump discharge line. Note that heat is generated when adding dilution water to caustic soda, and the piping system must be designed to accommodate. Also, any calcium in the dilution water will precipitate and cause scaling. Feeding systems for caustic soda are about the same as for liquid alum except for materials of construction.

Caustic soda is poisonous and dangerous if handled improperly. To avoid accidental spills, all pumps, valves, and lines should be checked regularly for leaks. Operators should be properly instructed, in the precautions needed for the safe handling of this chemical. Emergency eyewashes and showers should be provided close to the caustic soda storage and feed area to protect personnel from accidental spills. Table 24–5 provides a list of acceptable materials for piping and accessories for caustic feed systems.

TABLE 24-5. Materials Suitable for Caustic Soda Systems

Components	Recommended Materials for Use with 50% NaOH up to 140°F
Rigid pipe	Standard-weight black ion
Flexible connections	Rigid pipe with ells or swing joints, stainless steel or rubber hose
Diluting tees	Type 304 stainless steel
Fittings	Steel
Permanent joints	Welded or screwed fittings
Unions	Screwed steel
Valves—Nonleaking (plug)	
Body	Steel
Plug	Type 304 stainless steel
Pumps (centrifugal)	
Body	Steel
Impeller	Ni-resist
Packing	Blue asbestos
Storage tanks	Steel

Sodium Hypochlorite On-site generation of sodium hypochlorite is gaining in popularity. It is typically produced at 0.8 percent strength and contains 0.07 pounds of equivalent chlorine per gallon of solution. To generate a pound of chlorine in solution requires 3.5 pounds of salt (NaCl), 2.5 kW-hrs of electricity, and 15 gallons of water. Because the sodium hypochlorite is generated on-site, the solution will not degrade due to the almost immediate use of the material. Also, its strength is lower than the commercially delivered material, which usually has a strength of 15 percent. In order to provide the reliability of production, several days' supply of salt is required. A typical unit is shown in Figure 24–22.

In some plants, sodium hypochlorite is used to disinfect the water. When feeding this chemical, extreme care must be taken in designing the piping system. The following are some of the issues that need careful design consideration:

- *Scaling*—Because of the high pH of the chemical, it raises the pH of the dilution water. At pH's at or above 10, the calcium in the dilution water will react to form calcium carbonate and scale the carrier pipe.
- *Leakage*—The solvent-welded joints of a PVC piping system will leak if they are not prepared and installed per the manufacturers' recommendations.
- *Gas production*—This chemical loses strength over time and produces a gas in the process. This gas can accumulate in the pipeline, valves and pumps. This effect has caused pumps to ''air lock'' and ball valves to explode due to trapped gas. It has been estimated that a 15 percent solution produces about 1 percent of gas per day at room temperature.
- *Storage*—The chemical is very aggressive and will attack many materials. The most reliable we have found is fiber-reinforced plastic or polyethylene. Either tank may last only from 7 to 10 years.
- *Crystallization*—Place tank outlets 6 to 12 inches above the bottom of the tank to minimize formed crystals from entering the pipe and finding their way to the pumps.

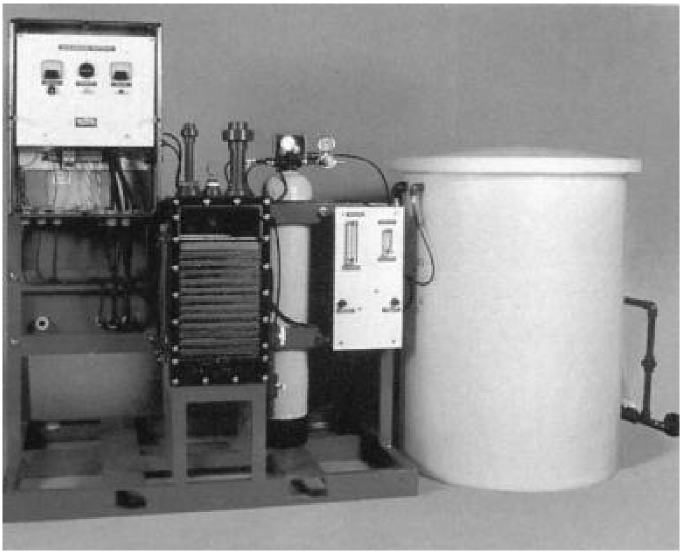


Fig. 24-22. Sodium hypochlorite generator (Courtesy of USFilter/Wallace and Tiernan)

- *Piping*—Size discharge piping to have a velocity of at least 6 ft/sec at average day feed rates to minimize gas production and pump binding. Another design criterion is to limit the detention time in the discharge piping to 2 hours.

Compatibility of chemicals is the key to having reliable systems. Chemical compatibility tables are included in Appendix G.

Manual and Low Flow Pacing for Chemical Feed Systems

There are three commonly used chemical feed systems: (1) dry feeders, (2) solution feeders, and (3) gas feeders. Each type of feeder should have one or more means to adjust the chemical feed rate (dosage) easily. The adjustment(s) may be manual or by flow pacing, and must be accurate, repeatable, and easy to change. Also, they should provide the broadest possible adjustment span from minimum to maximum feed rate. Typical ranges are 10:1 to 20:1, but greater ranges are possible.

Dry Feeders Most dry feeders are of the belt, screw, or oscillating plate type. The feeding device (belt, screw, disc, etc.) is usually driven by an electric motor. Many belt feeders, particularly gravimetric-type feeders, also contain a material flow control device such as a movable gate or rotary inlet device for metering or controlling the flow of chemical to the feed belt.

Volumetric Dry Feeders Most volumetric dry feeders are of the rotating screw or disc type, but the belt and rotary star valve types also are used. Generally, the screw or disc type is driven by an electric motor through a gear reducer drive. In some cases the drive assembly (excluding the motor) contains a variable speed or a linkage adjustment that allows feed rate changes. Otherwise, the feed rate adjustment must be made directly to the motor drive.

Manual Feed. Manual dosage adjustment of volumetric dry feeders is accomplished by one or more of the following means:

- Drive motor for feed screw, belt, disc, or rotary valve
- Manual variable speed
- Percentage time control motor that operates on a run-stop repeat cycle with run time set by percentage timer and adjustable from 0 to 100 percent of the total cycle. Typical total run-stop cycle times are 15, 30, or 60 sec. For a system set up on a 60-sec basis, the following is typical operation:

Timer Setting, Percentage	Run Time, sec	Off Time, sec
100	60	0
75	45	15
50	30	30
25	15	45
0	0	60

- Manually adjustable speed reducer or adjustable linkage on drive assembly. Control gate for belt feeder.
- Manual setting of gate position to allow more or less material on belt

Flow Pacing. Automatic proportioning of volumetric dry feeders to flow (commonly called flow pacing) is readily accomplished for a variety of flow signals. The more common flow and control signals are:

- Pulse duration (on-off), with frequently used cycle times of 15, 30, and 60 cycles
- 3 to 15 psi (20.7 to 103.5 kPa) pneumatic
- 4 to 20 or 10 to 50 milliamperes (mA), as shown in Figure 24–23
- 1 to 5 volts dc

Most modern feeders can accept one or more of the flow and control signals. If the feeder will not accept the particular signal available, signal converters are readily available to convert the signal to one that the feeder will accept. For example, a 3 to 15 psi (20.7 to 103.4 kPa) pneumatic signal can be converted to a 4 to 20 mA signal. Signal converters are relatively inexpensive and reliable.

If a volumetric feeder is automatically flow-proportioned, a means still must be available for setting the feed dosage. Typically, this is accomplished by one of two methods:

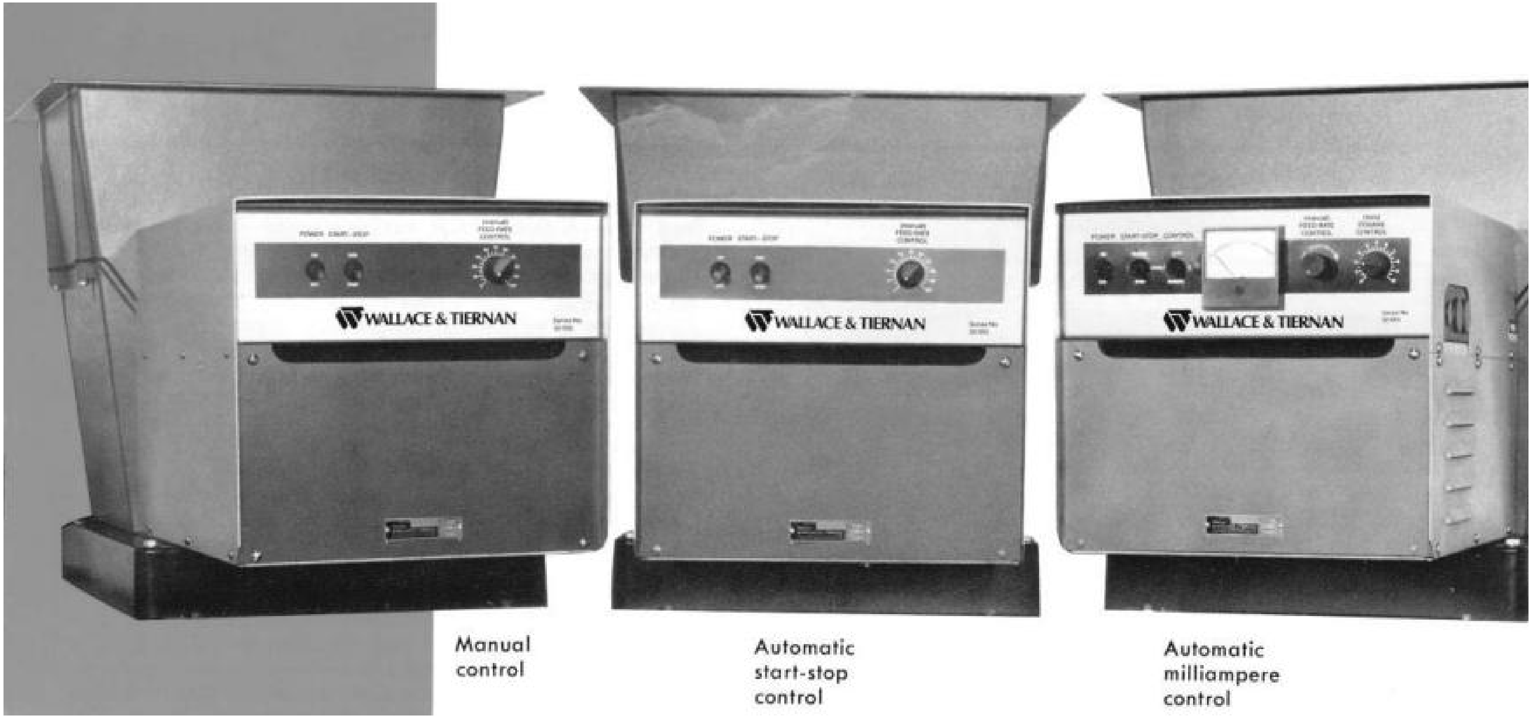


Fig. 24-23. Volumetric feeder automatic milliampere control (Courtesy of USFilter/Wallace and Tiernan)

- A “manual feed rate control” built into the feeder that modifies the automatic proportioning signal within the feeder control system
- A separate manual adjustment, such as a mechanical linkage or feed gate adjustment, or a manual speed adjustment on the gear reducer drive system

Gravimetric Dry Feeders A gravimetric dry feeder has a built-in control system that ensures a constant feed weight, rather than volume, for any given dosage setting.

Manual Feed. For manual feed systems, an operator dosage adjustment is provided as part of this gravimetric control system. Continuous weighing of the feed belt establishes automatic internal control of the gate position (or rotary inlet valve speed), thereby maintaining a constant belt weight for a given dosage setting.

Flow Pacing. The simplest way to obtain automatic proportioning control is to provide a variable-speed drive for the belt. The automatic proportioning control can be used to vary the belt drive motor speed.

If the feeders are to be shut down automatically, provisions should be made for shutdown and start-up of system components such as the feeder, storage bin vibrators, water supply to dissolvers, mixers, and solution transfer pumps.

The dissolver water supply and mixer should operate on an adjustable time delay, after the feeder is stopped to prevent chemicals from settling in the dissolver.

Solution Feeders The most common solution feeders are motor-driven diaphragm- and plunger-type feed pumps. Generally, these pumps have a built-in stroke adjustment mechanism that permits variation of the output feed rate.

Manual Feed. For manual dosage control, the motor operates at a constant speed, and the operator adjusts the dosage with the stroke adjustment.

Flow Pacing. Automatic proportioning control can be accomplished readily by modification of the drive motor to provide variable speed or on-off proportioning, or installation of an automatic stroke adjuster.

The drive motor can be set up to operate at variable speeds proportional to pneumatic or electric signals. On-off pulse duration signals can be applied directly to the motor starter so that the motor operates on and off in proportion to the signal. This involves many start-stop cycles for the motor, and it is recommended that only three-phase motors be used for this type of duty. When the automatic proportioning is accomplished with the drive motor, the manual stroke adjustment is still available for operator-adjusted manual dosage changes.

Automatic stroke adjusters can be installed in place of the manual adjuster on most feeders. The automatic adjusters will accept various analog control signals, including pneumatic and electric signals. These automatic stroke adjusters can be used for automatic proportioning, but in most cases there are not manual settings available for manual dosage adjustments. In some cases, a manual dosage adjustment can be incorporated into the automatic stroke adjustment mechanism, or a manual variable-speed motor drive can be provided for operator manual dosage adjustment.

Probably the most satisfactory arrangement is to proportion automatically with a variable-speed or pulse-duration motor drive, and to leave the manual stroke adjuster for operator dosage adjustments. This approach minimizes the amount of automation, while still providing fully flow-paced operation.

If the solution feed system is to be started and stopped with plant operation, or on some other basis, consideration must be given to starting and stopping auxiliary systems such as the solution feeder, dilution water, transfer pumps, mixers, and similar equipment.

There are a number of solution batching and feed systems on the market, particularly for polymers. These systems automatically produce a feedable polymer solution from dry polymer. Typical systems include a dry polymer storage hopper, dry feed, wetting, a dissolver tank with mixing, a feed tank, and solution feeders. Most of these systems are sold as a pre-engineered package and can be supplied for manual control, automatic proportioning, and automatic start and stop operation. Generally, these systems use solution feed pumps for metering, and the previous comments concerning solution feeders are applicable.

Gas Feeders Most modern gas feeders are vacuum-operated. The gas is accurately metered through an orifice with a fixed pressure drop across the orifice. For adjustment of the gas feed rate, the orifice size can be changed manually or automatically. The vacuum is produced by pumping water through an injector, which provides the vacuum as well as the mixing required to dissolve the chlorine in the solution water. Another method is to use an eductor pump. The units utilize an impeller design to produce a vacuum. The pumping action provides mixing of the gas into the entire treatment stream.

Manual Feed. Most small, inexpensive gas feeders have provisions for manual adjustment of the orifice size to change the gas-flow rate (dosage). This adjustment is made with a knob on the front of the feeder. Normally the gas-flow rate is indicated by a visual flow indicator calibrated in pounds per day.

Flow Pacing. Gas feeders may be automatically proportioned in a number of ways. Two common methods are variable vacuum control and automatic positioning of the orifice control.

Variable vacuum control systems are an economical method of automatically proportioning gas feeders. The vacuum control system consists of a vacuum-producing device such as the gas injector, a restricting orifice, and an intermediate vacuum transmitter. The primary flow signal is converted to a proportional vacuum signal, which is applied to the vacuum-regulating valve on the downstream side of the gas feeder orifice. The pressure ahead of the orifice is maintained at a constant value by the inlet gas pressure–regulating valve. The gas feed rate is varied automatically as this proportioning vacuum signal changes with the flow.

Changes in the control vacuum signal cause comparable changes in the pressure downstream of the orifice and, therefore, in the differential across the orifice. Because the square of gas flow is proportional to the differential pressure across the orifice, the gas feed rate will vary in accordance with the control vacuum signal.

Automatic positioning of the gas-flow control orifice is accomplished with a power positioner, which can be selected to operate from a number of input signals such as:

- 3 to 15 psi (20.7 to 103.4 kPa) pneumatic
- 4 to 20 or 1 to 50 mA
- 1 to 5 volts DC electric
- Potentiometer position
- Pulse duration
- Pulse frequency
- Others by special application

When an automatic proportioning positioner is used, a manual dosage control knob is provided on the chlorinator so the operator can make manual adjustments of dosage. This adjustment modifies the automatic proportioning over a wide range. A small flow-paced system is shown in Figure 24–24.

Fig. 24–24. Flow-paced chlorinator (Courtesy of USFilter/Wallace and Tiernan)

Almost all gas feeder control schemes are based on the use of variable vacuum or the use of an orifice proportioning positioner. In most cases, the manual dosage adjustment is retained for operator use.

Gas feeders can be started and stopped simply by starting or stopping water flow through the injector. Usually this is all that is necessary to start or stop a typical gas feed system such as a chlorinator or sulfonator.

Automatic Control for Chemical Feed Systems

Various automatic control schemes are possible for chemical feed systems beyond the automatic flow proportioning discussed for each type of feeder. For example, automatic pH control is possible using a pH sensor, controller, and pH adjustment chemical feeder for sodium hydroxide with an automatic stroke positioner.

Automatic chlorine residual control is possible using a chlorine residual analyzer, controller, and chlorinator with an automatic orifice positioner. These are ‘‘feed back’’ systems where the final control parameter, such as pH, is controlled by a previous feed of chemical. ‘‘Feed forward’’ systems are also possible where a parameter concentration prior to chemical feed is determined and related to the chemical feed for automatic control.

Another method that is gaining wide acceptance is the use of an ORP probe, to control chlorine feed. These probes, which measure the change in redox potential, have been used as a surrogate for direct chlorine residual measurement with success. The ORP probe measures the rate of oxidative disinfection. The output from the ORP probe can be converted into a 4- to 20-millivolt signal that can then be used to control the chlorine feed. The higher the signal, the higher the chlorine dose. Typically, the signal used to control the chlorine feed rate is established by trial and error using a residual analyzer. The probe is typically located near the beginning of the basin, and in some cases a second probe is located at the end of the basin. In order to calculate the *Ct* value, a chlorine residual analyzer may need to be installed as well. Some ORP probes are available with a residual analyzer as part of the unit.

Various types of ‘‘feed forward’’ or ‘‘feed back’’ systems or combinations can be devised in theory. The problem is that there are substantial delay times in such systems that most analog controllers are not designed to handle. The delays result from the time required to change a chemical feeder setting and get the change to the injection point, as well as process delays to the sample point, delay in the sample lines, and delay in the analyzer. The total delay from the time a feed rate is changed until it is read out by an analyzer can be 5 minutes or longer. The control results can be unstable and can lead to wide, cyclic variations of the controlled variable. There are ways of overcoming these problems with proper design and equipment selection, but proper design of such systems is a very specialized skill.

Automatically controlled systems must be arranged so that auxiliary systems such as mixers, dilution water, and slakers are started and stopped as needed. Such automated systems can be designed and applied; however, their complexity and maintenance requirements are such that they should not be used unless their benefits clearly outweigh their operational disadvantages.

Buildings

When designing enclosures for chemical facilities, care must be taken to review the applicable building and fire codes. Many chemicals are hazardous and are not compatible with one another, and special treatment will be required. Make sure that the state and local codes are reviewed to determine which will apply to the project. The design engineer should also determine who the code enforcing official is, what the requirements are with respect to the codes, and if there will be a third-party reviewer.

REFERENCES

1. American Water Works Association, *Water Quality and Treatment,* 4th ed., McGraw-Hill, New York, 1990.
2. U.S. Department of Health, *Water Fluoridation, A Manual for Engineers and Technicians,* U.S. Department of Health and Human Services, Public Health Servic, Atlanta, GA, September, 1986.
3. The American Institute of Architects, *Architectural Graphic Standards,* 8th ed., John Wiley & Sons, Inc. New York, 1988.
4. White, George Clifford, *Handbook of Chlorination, and Alternative Disinfectants,* 3rd ed., Van Nostrand Reinhold Co., New York, 1992.
5. AWWA, Manual M20, *Water Chlorination Principles and Practices,* 1973.
6. *Chlorine Manual,* 6th ed., The Chlorine Institute, New York, New York, 1997.
7. Heim, Nancy E., and Burris, Bruce E., Culp/Wesner/Culp, *Chemical Aids Manual for Wastewater Treatment Facilities,* USEPA, MO-25, 430/9–79–018, December, 1979,
8. *Lime Handling, Application & and Storage,* Bulletin 213, 7th ed., National Lime Association, Washington, D.C., May, 1995.

SUPPLEMENTARY READING

Miller, D. S., *Internal Flow Systems,* 2nd ed., BHR Group Limited, The Fluid Engineering Centre, Bedfordshire, UK, 1990.

CHAPTER 25

Package Water Treatment Systems

INTRODUCTION

Conventional water treatment systems include the following unit processes:

- Coagulation with a primary coagulant such as alum or ferric chloride and
- Flocculation
- Sedimentation
- Filtration

Conventional systems for large plants are most often custom designed and almost completely constructed on site. There is considerable economy of scale in designing and constructing conventional systems, and they can be very expensive for small flows. Package systems are an alternative for small systems, with a capacity of about 3 mgd or less. Package systems include the same unit processes that are predesigned and constructed modular units in a factory, including the control system. The modular units are then assembled on site and are often skid mounted.

As of 1998, it is estimated that over 2,000 package units, ranging in capacity from 5 gpm to 20 mgd, are in service in the United States. These represent only a fraction of the potential applications of modular units. Assuming an annual average per capita water consumption of 150 gpd, water systems serving a population of 10,000 persons would require a 1.5-mgd-capacity treatment plant. In 1998, there were over 42,000 small community water systems that served populations of 10,000 or less. For many of these communities, treatment is required to meet the Safe Drinking Water Act standards.

Package units can be used to treat water supplies for communities as well as noncommunity water systems, such as those for factories, schools, recreational areas, state parks, construction camps, ski resorts, remote military installations, and other locations where potable water is not available from a municipal supply. Several state agencies have mounted package units on trailers for emergency water treatment. Their compact size, low cost, minimal installation requirements, and ability to operate virtually unattended make them an attractive option in locations where revenues are not sufficient to pay for a full-time operator.

Package systems covered in this chapter are designed to provide treatment that is equivalent to conventional systems that are custom designed and constructed. Therefore, the choice between a custom system and a package system is primarily a matter of economics. At some point, generally in the 2 to 5 mgd range, it becomes more economical to design and construct a custom system.

Design Considerations Unique to Small Systems

A primary design consideration for small systems is the need to automate the treatment equipment as much as is practical, to minimize the amount of operator attention required for day-to-day operation of the treatment plant. Also, plant automation reduces the amount of skill and judgment the operator must use to run the equipment. For example, water filtration equipment should be designed to backwash automatically in the absence of the operator and return to service on a preprogrammed basis. Also, sludge wasting from the clarifier should be automated to eliminate another function requiring attention and judgment.

The complexity of the equipment used in a small system should be a major consideration during design. Totally automated equipment does not necessarily need to be complex. The development of solid-state control components has greatly improved the reliability of automatic control systems. Equipment should be used for which readily attainable spare parts are available. To the extent that it is practical, a sufficient inventory of spare parts should be obtained with the construction bid. The availability of spare parts greatly simplifies repair and maintenance work during a plant emergency. This is especially important to a small system, which, unlike a larger utility, may not have the ability to obtain spare parts quickly.

CONVENTIONAL PACKAGE SYSTEMS

The modular unit is factory-built, and its quality and cost can be controlled closely to produce an efficient and economical treatment facility. Package treatment plants have been offered for many years by a number of manufacturers. Treatment concepts in modular units, by and large, have duplicated those used in larger, conventionally designed water treatment plants. Where packaged treatment plants have been properly applied and operated, performance has been satisfactory.

Package water treatment plants are available from several manufacturers in a wide range of capacity, incorporating a complete treatment process (coagulation, flocculation, settling, and filtration). Design criteria used for these modular units vary widely. Some manufacturers adhere closely to accepted conventional design practices, such as 20- to 30-minute flocculation detention time, a 2-hour sedimentation detention time, and rapid sand filters rated at 2 gpm/ft^2. Other manufacturers have utilized new technology, including tube settlers and high-rate dual- and mixed-media filters to reduce the size of a plant and hence extend the capacity range of single factory-assembled units. Often, state regulatory agencies dictate the design criteria that must be met by modular unit manufacturers and exclude units using new technology that does not meet standards adopted by these agencies.

In the mid-1960s, coupled with the development of short-detention-time tube settlers, a modular unit was introduced using the high-efficiency, short-detention-time clarification devices. The use of tube settlers effectively reduces the settling detention time by a factor of 10 to 1 over conventional settling basins. This significant reduction in the volume of the settling basin, when used in conjunction with mixed-media filters rated at 5 gpm/ft^2, led to the development of a compact modular unit with significantly greater capacity per unit volume than other equipment available at the time. The sizable reduction in modular unit tankage greatly increased the capacity attainable in a single truck-transportable unit. For example, using tube settlers and mixed-media filters, a single factory-assembled truck-transportable package unit can be built with a capacity of 1 mgd. Following conventional design criteria, a plant of the same physical dimensions would produce less than 0.25 mgd.

A flow diagram for a conventional package plant is shown in Figure 25–1.

USFilter Waterboy and Aquarius

USFilter manufactures two versions: ‘‘Water Boy’’ with a treatment capacity range of 10 to 100 gpm (Fig. 25–2), and the ‘‘Aquarius’’ (Fig. 25–3), which generally consists of dual units with a combined capacity of 200 to 1,400 gpm. As illustrated in the flow diagrams, raw water is either pumped or flows by gravity to the treatment plant. The influent flow is adjusted to the desired rate; the control system is designed to start and stop the treatment plant according to a clearwell level that reflects system demand. The coagulant and disinfectant chemicals are added at the influent control valve. A polyelectrolyte coagulant aid is applied as the water enters the flash mix chamber. After the treatment chemicals are added and mixed, the water is introduced into a mechanical flocculator designed to form a quick-settling floc. Flocculation detention time can vary from 10 minutes in small units to 20 minutes in larger units. The flocculated water is then distributed through a bank of tube settlers, which consist of many 1-inch-deep, 39-inch-long split-hexagonal-shaped passageways that provide an overflow rate, related to available settling surface area, of less than 150 gpd/ft^2. This overflow rate, together with a settling depth of only 1 inch, results in effective removal of flocculated turbidity with a detention time of less than 15 minutes.

After passing through the tube settlers, the clarified water flows to a gravity mixed-media filter. Type A and B modular units utilize a 30-inch-deep mixed-media filter bed consisting of 18 inches of 1.0 to 1.1 mm effective size anthracite coal, 9 inches of 0.35 to 0.45 mm effective size silica sand, and 3 inches of 0.20 to 0.30 mm effective size garnet sand, designed so that there is a uniform gradation from coarse to fine in the direction of filtration. In all units, the design filtration rate is 5 gpm/ft^2.

The filters are designed to operate at a constant flow rate. Rate control is accomplished with a low-head filter effluent transfer pump discharging through a float-operated valve. With this means of filter rate control, once the plant raw water flow is established, there is no change in the filter rate throughout the entire filtration cycle, provided that the inflow to the plant remains the same. If there is a slight change in

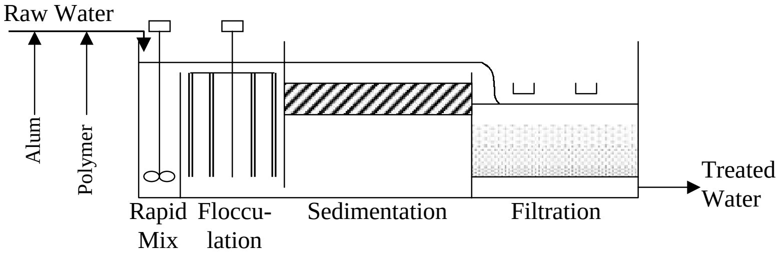


Fig. 25–1. Flow diagram of a conventional package plant

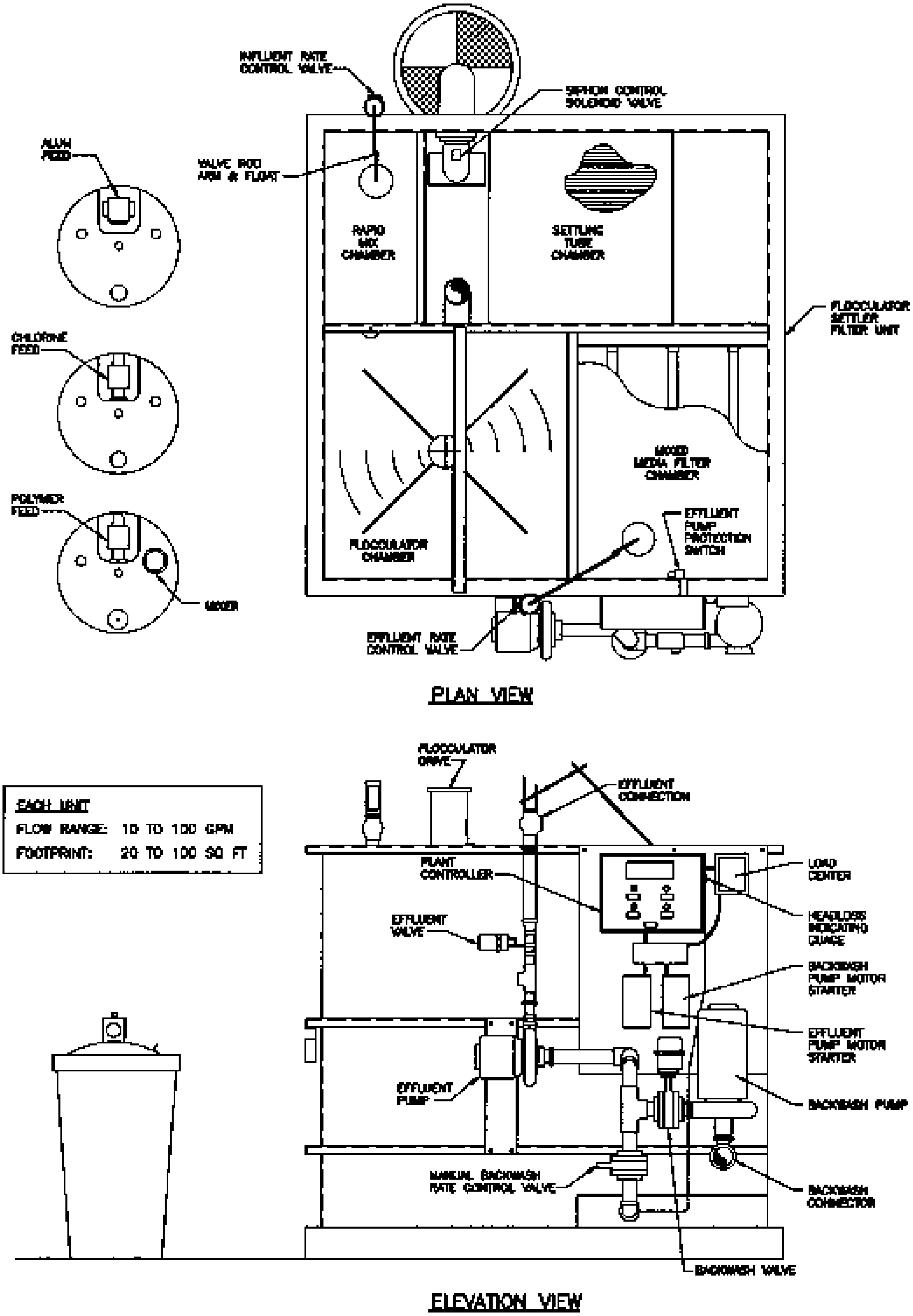


Fig. 25-2. Conventional package plant layouts for Waterboy (Courtesy of USFilter)

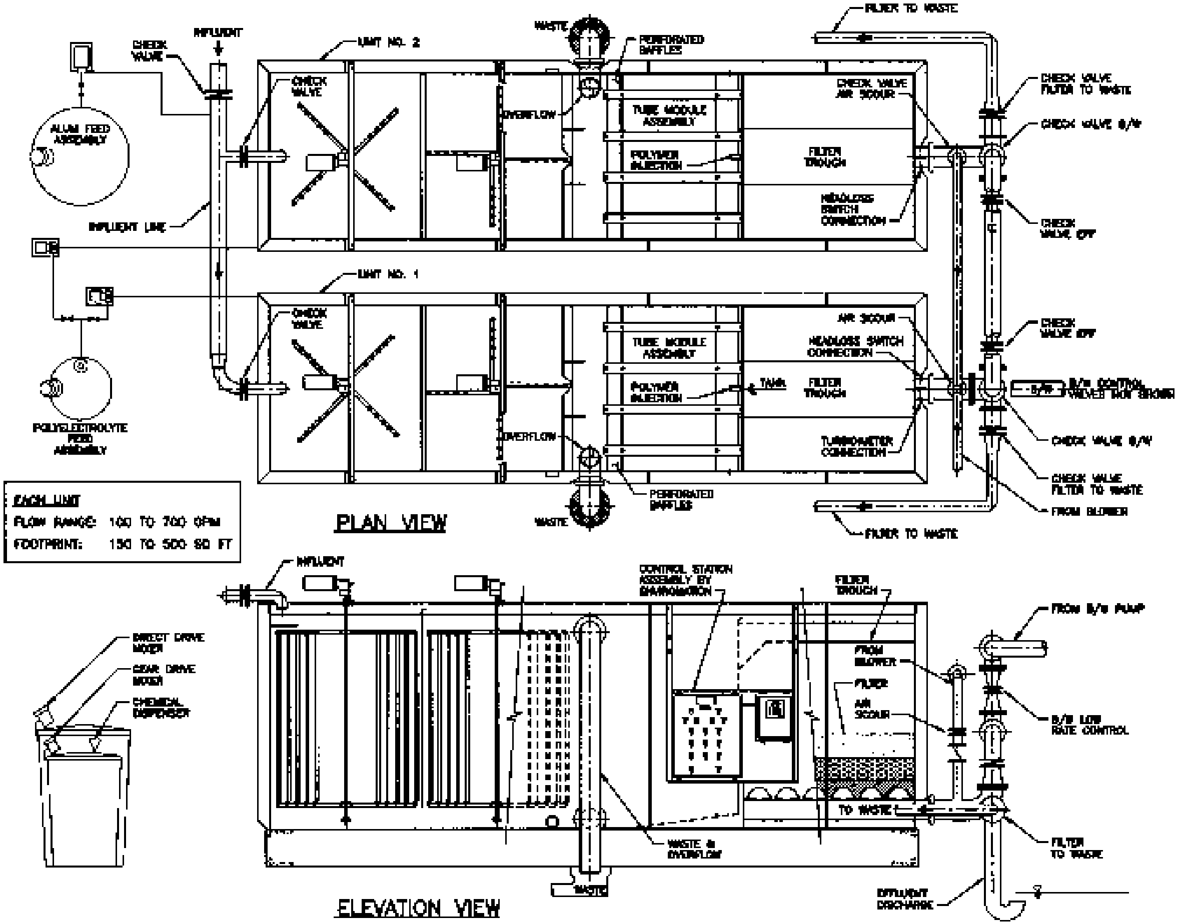


Fig. 25-3. Conventional package plant layouts for Aquarius (Courtesy of USFilter)

flow, increases or decreases in the filtration rate are accomplished slowly, minimizing filter surging and the chance for turbidity breakthrough.

The modular unit filter is designed to backwash automatically once a preset filter headloss is reached, or the operator may override the automatic controls and backwash the filter manually. During backwash, the material accumulated in the tube settlers is automatically drained from the unit. Combining backwashing with draining of the tube settlers for sludge removal eliminates the need for an operator to judge how often or how much sludge should be wasted from the clarifier. This particular feature simplifies operation and reduces the required skill level of the operator.

Prior to the end of filter backwash, the drain valve on the tube settler basin is closed, allowing the remaining backwash water to refill the settling tube compartment. Upon completion of the backwash cycle, the treatment plant is returned automatically to service. Operational requirements are only to replenish chemical feeds, establish proper dosages, conduct routine water-quality tests, and carry out routine daily maintenance activities.

Roberts Water Technologies, Inc.—RELIANT System

Steel and stainless steel Reliant treatment units are available in both modular and packaged (preassembled) designs. The package Reliant units are available with rates of up to 100 gpm; modular systems are available with rates of up to 700 gpm. Units utilize mechanical flocculation to mix chemicals and to form a large, settleable floc. These units include a high-rate, tube settler—equipped clarifier, with an overflow rate of about 2.5 gpm/ft^2. The unit also includes an open gravity filter with a design flow rate of 5 gpm/ft^2. The filter is supplied with either dual media (12 inches 0.5-mm sand and 18 inches 1.0-mm anthracite) or tri-media for improved performance. Effluent from the filter can be by gravity flow or pumped. The unit includes air backwash (2.5 scfm/ft^2) and water backwash (15–18 gpm/ft^2). Sludge is withdrawn automatically from the clarifier through a sludge collection system. An influent-throttling valve that operates off the water level in the treatment plant controls plant flow. The plant is designed to operate unattended, requiring only replacement of treatment chemicals and adjustment of chemical dosage as raw water conditions change. Figure 25–4 is a drawing of a typical Reliant unit and Figure 25–5 is a photograph of a package Reliant unit.

CONTACT CLARIFICATION-FILTRATION PACKAGE SYSTEMS

A package system developed around 1980 and now available from several manufacturers incorporates a high-rate flocculation-clarifier unit with fixed or floating media followed by mixed or dual media filters. The unit is preceded by coagulant addition and high-energy mixing. The State of California and others have adopted the term *contact clarification-filtration* to identify this technology.

Studies in California have demonstrated effective treatment in conformance with the Surface Water Treatment Rule on waters where the median total coliform MPN is less than 500 per 100 mL and the turbidity is less than 15 NTU.[1] A substantial number of particle count and organism challenge studies have been completed in California, with treatment systems designed in conformance with this technology. These studies in California concluded that “additional pilot plant studies should not be necessary

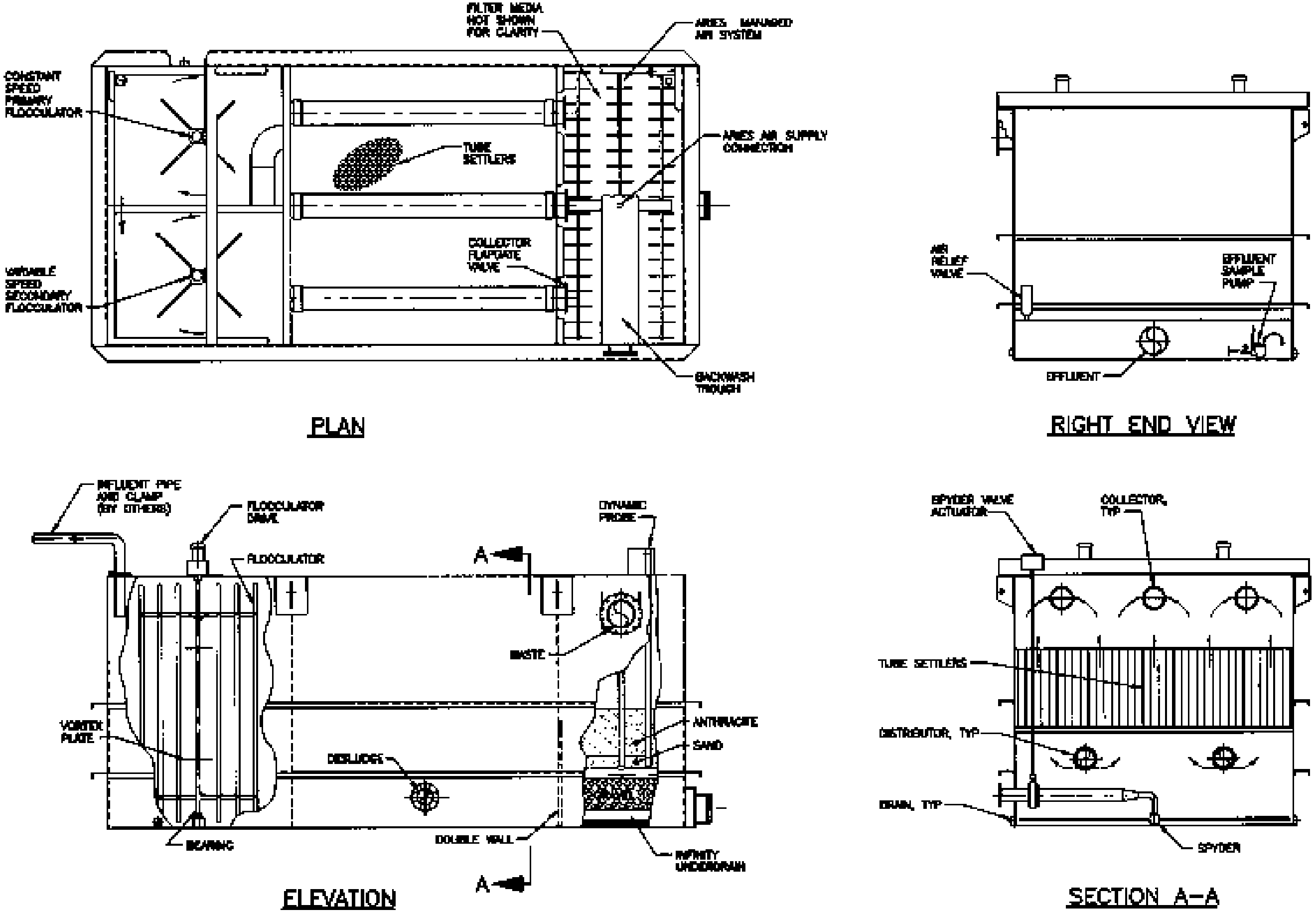


Fig. 25-4. Reliant conventional package plant installation (Courtesy of Roberts Water Technologies)

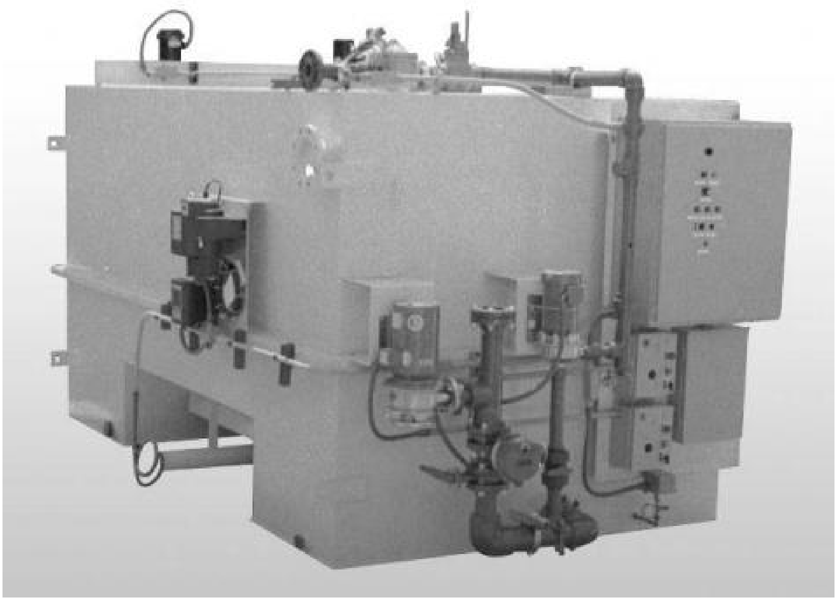


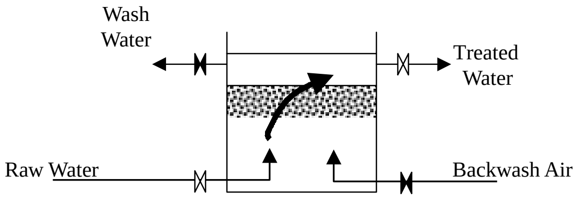
Fig. 25–5. Photograph of Reliant (Courtesy of Roberts Water Technologies)

except to ascertain the ability to deal with source specific water quality problems and identify the best coagulant and optimum dose.'' The systems qualified in California for approval without further studies are:

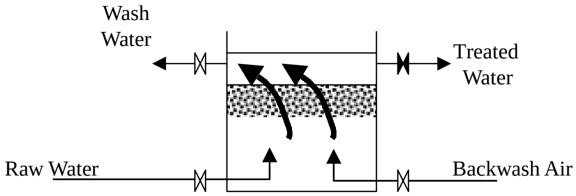
- Microfloc Trimate and Trident, manufactured by USFilter
- Pacer II, manufactured by Roberts Filter Group
- Advent Package Water Treatment System, manufactured by Infilco Degremont

The space-saving potential of the equipment is evident because the rise rate through the adsorption clarifier is generally twice the filtration rate (10 versus 5 gpm/ft^2). A contact clarifier therefore requires about one-half of the space of a typical high-rate filter.

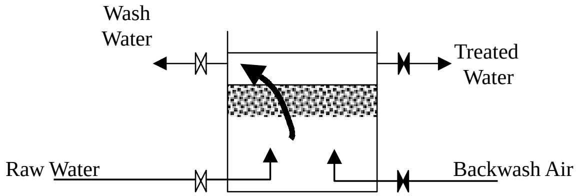
Cleaning of the contact clarifier is accomplished by flushing. A timer initiates the flush cycle, but the equipment also includes a pressure switch that monitors headloss across the adsorption media and can automatically initiate a flushing cycle if required. Figure 25–6 illustrates the operation of a typical contact clarifier flushing cycle. When a cycle is initiated, the plant effluent valve closes, causing the water level to rise in the plant as the influent flow continues. When the water level reaches a predetermined level, a switch causes the influent valve to close. Air in the valve opening is distributed through perforated laterals beneath the adsorption media. This causes an immediate expansion in the media, and a vigorous scrubbing action takes place. Influent feed is opened again and dislodged solids are then hydraulically flushed out of the top of the contact clarifier to waste. Flushing frequency may vary, depending upon influent water



Normal Operation



Backwash Operation



Flushing Operation

Fig. 25-6. Backwash cycle for typical contact clarifier

quality. Typically, the controls are set up to initiate a flushing cycle every 4 to 8 hours. Unlike conventional filters, complete cleaning of the adsorption clarifier is not required, as the majority of solids are removed by the violent agitation provided during the first minutes of the flush cycle. Also, more efficient performance of the adsorption clarifier occurs if some residual solids are left on the media.

The mixed-media filter is backwashed in a manner similar to that of a conventional filter. Although the filter may not necessarily be backwashed each time the adsorption clarifier is flushed, the equipment is designed to ensure that a backwash cycle is always preceded by a flushing cycle.

USFilter—Microfloc Trimite and Trident Systems

The Trimite and Trident package water treatment plant designs utilize an upflow filter of high- density plastic bead media, termed an ‘‘adsorption clarifier,’’ followed by a mixed-media filter for final polishing. The adsorption clarifier replaces the flocculation and settling processes and results in an extremely compact unit. Figure 25–7 is a flow diagram of the typical Trimite and Trident Water System manufactured by USFilter,

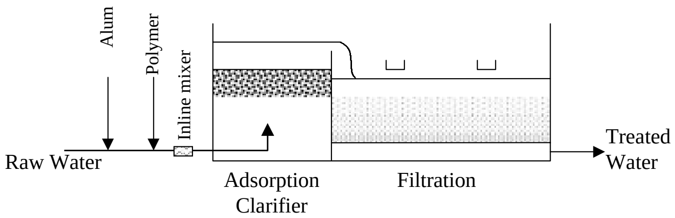


Fig. 25-7. Flow diagram of a adsorption clarifier package plant

illustrating the various operating cycles. During operation, chemically coagulated water is introduced into the bottom of the adsorption clarifier compartment, where it passes upward through a bed of buoyant adsorption media. The adsorption clarifier combines the processes of coagulation, flocculation, and settling into one unit process.

In passing through the adsorption media, the chemically coagulated water is subjected to: (1) mixing, (2) contact flocculation, and (3) clarification. At operating flow rates, the mixing intensity, defined by the mean temporal velocity gradient value G, ranges from 150 to 300 $\sec^{-1}$. Flocculation is accomplished by turbulence as water passes through the adsorption media, and is enhanced by contact with flocculated solids attached to the media. Estimates of the mixing parameter Gt in the adsorption clarifier range from about 1×10^4 to 3×10^4, depending on flow rate and the rate of headloss development.

Turbidity removal in the adsorption clarifiers is accomplished by adsorption of the coagulated, flocculated solids on the surfaces of the adsorption media and on previously attached solids. The adsorption clarifier provides excellent pretreatment, which frequently is better than the performance achievable with complete flocculation and settling processes. Turbidity removal in this stage ranges up to 95 percent.

The material used for the adsorption media was selected as a result of experimentation with various materials, all with a specific gravity of less than 1 and of various sizes and shapes. The buoyant media is retained in the adsorption clarifier by a screen over the compartment.

Two style units are available, with capacities ranging from 350 to 4,200 gpm. The Trimite (Fig. 25–8) models are designed for flows from 50 to 350 gpm per unit, and the Trident (Fig. 25–9) for flows 200 to 1,400 gpm per unit. Multiple units are used for higher flows.

Roberts Water Technologies Inc.—PACER II System

Pacer II units are also available in both steel and stainless-steel construction. Package Pacer II units are available with flow rates up to 350 gpm, and standard modular units are available with design flow rates of up to 1,400 gpm. Following is an operational description of the Pacer II system.

The Pacer II Dual Treatment System is an effective surface or groundwater treatment process capable of producing high-quality filtered water in varying raw-water conditions. The Pacer provides multiple barriers of protection through the combination of contact clarification and deep bed filtration. This combination provides a high level of particle conditioning that has been proven through extensive testing to achieve greater than three-log reductions of both *Giardia* and *Cryptosporidium*.

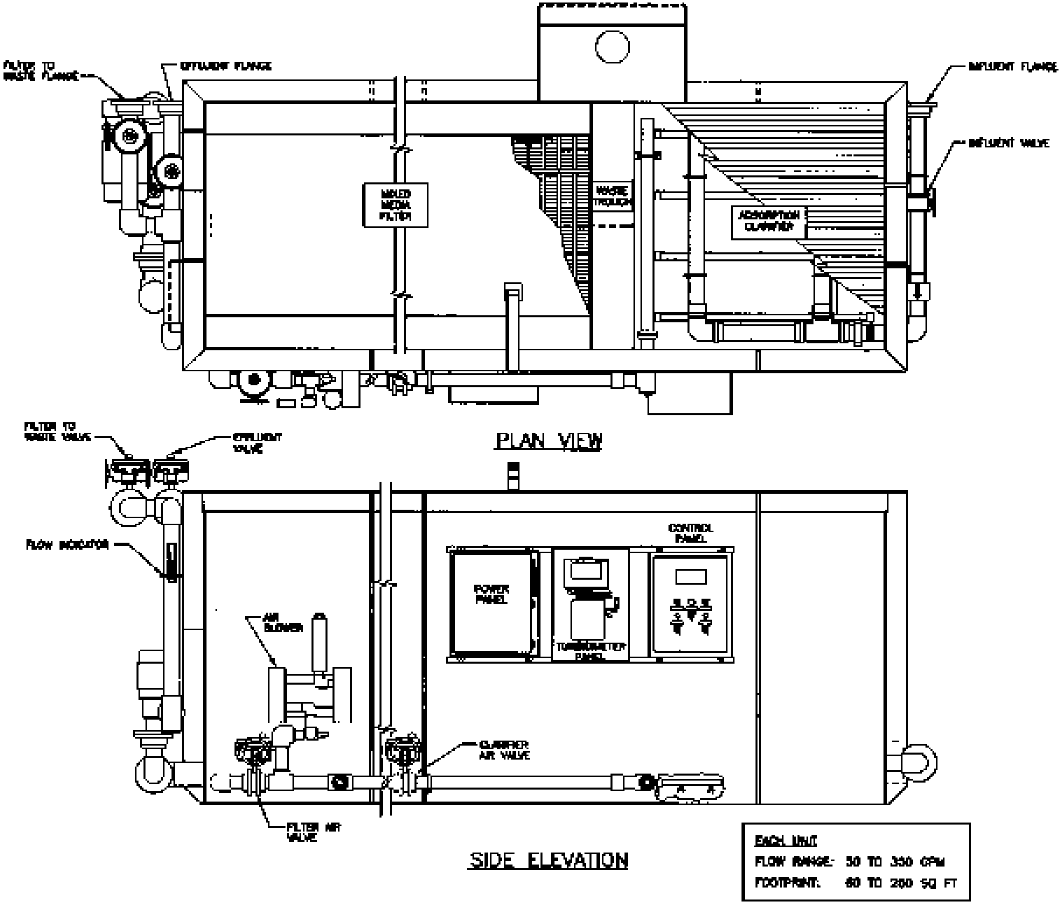


Fig. 25-8. Layout for trimite adsorption clarifier package plant (Courtesy of USFilter)

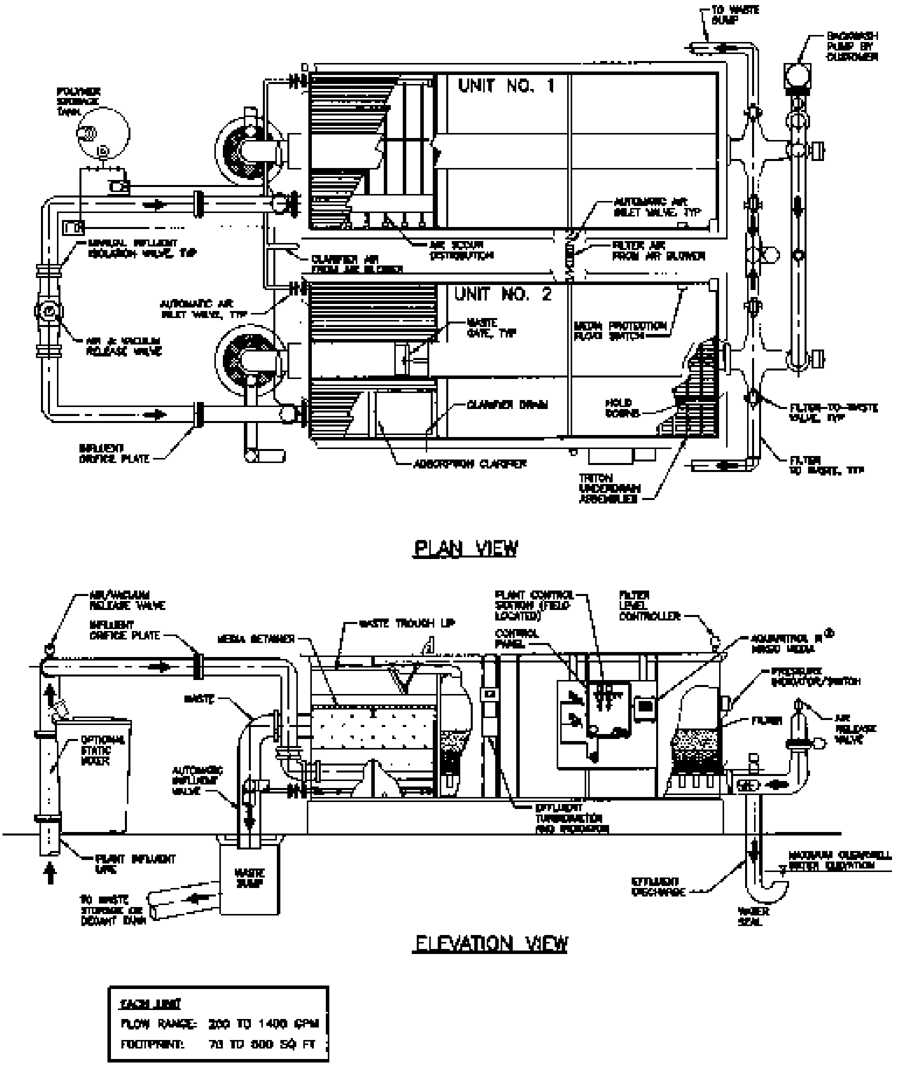


Fig. 25-9. Layout for Trident adsorption clarifier package plant (Courtesy of USFilter)

The first stage, the ContaClarifier, is a granular, nonbuoyant media bed that is stable under all service flow conditions. Chemically dosed water is introduced to the bottom of the clarifier section and flows upward through the bed. The flow around the irregularly shaped media particles causes the intimate mixing and interparticle collisions needed for floc formation. The flocculation process is enhanced by contact with the previously retained solids. Effective flocculation in combination with straining and attachment mechanisms in the media achieves clarification with up to 95 percent solids reduction.

The clarified and conditioned water is transferred to the second stage of the treatment process, the polishing filter. The conventional trimedia filter profile consisting of a coarse anthracite layer over silica sand and fine garnet was used at each plant. The polishing filter can consistently produce finished water quality below 0.1 NTU with low particle counts because of the highly effective conditioning for filtration of the Pacer II process.

ContaClarifier bed washing can be activated manually, or automatically at an elapsed time interval, or when headloss reaches a predetermined set point. Air agitation, along with normal operational raw-water upflow, scrubs the ContaCarifier media. Raw-water flow continues for a short period afterward, flushing the remaining solids to the waste line.

The backwash cycle incorporates a separate air wash step using the Aries Managed Air System for the most effective cleaning. The influent valve is closed, the filter is allowed to drain until the water level is about six inches above the filter media, and the Aries system is turned on. The scouring action developed by the Aries breaks the accumulated solids free from the filter media particles, ensuring effective cleansing at minimum backwash water use. The filter is air washed at about 2–3 cfm/ft^2 for 3–4 minutes. The backwash flow is then established for refill and maintained for 5–8 minutes at a rate of at least 15–18 gpm/ft^2. Flow is then reestablished through the clarifier and the unit begins the filter to waste cycle. This continues for 5–10 minutes until the effluent turbidity is below acceptable limits. The total backwash cycle time is 20–30 minutes.

Figure 25–10 is a drawing of a typical Pacer II unit, and Figure 25–11 is a photograph of a Pacer II 700 gpm unit.

Advent Packaged Water Treatment System

The Advent package water treatment unit by Infilco Degremont combines contact adsorption clarification with gravity filtration and is available for applications from 125 gpm to 1,400 gpm (see Figs. 25–12 and 25–13). In general, contact adsorption clarification applications require a fairly stable raw water source with low amounts of color and maximum turbidities between 20 and 40 NTU for efficient operation. After the raw water is coagulated, a coagulant aid polymer is typically introduced prior to entering the upflow contact adsorption clarifier. The raw water is evenly distributed at 10 gpm/sf via nozzles up through a 30-inch-deep monomedia. Additional flocculation occurs within the media and particles are trapped. Raw water flow is not interrupted during cleaning of the media, and air scour is added to agitate the media and release trapped particles periodically.

Clarified water is collected in a trough above the upflow media and is distributed to the downflow gravity filter, which is rated at 5 gpm/sf, and surface wash is optional depending on regulatory requirements or designer preferences. Media configurations differ per application.

The package treatment system is simple to operate and maintain. All functions of the system are automated with the exception of adjustment of clarifier flush rate, filter backwash rate, and/or air scour rate for seasonal water temperature change. All clarifiers and filters in the system are automatically cleaned and returned to service when headlosses reach preset adjustable levels, elapsed time, and/or high effluent turbidity. Controls are provided to facilitate manual initiation of automatic clarifier and/or filter cleaning cycles. The control system allows only one clarifier or filter to clean at a time, and provides queuing logic, to automatically schedule backwashing of multiple units. The unit is typically designed for separate air scour and water backwashing.

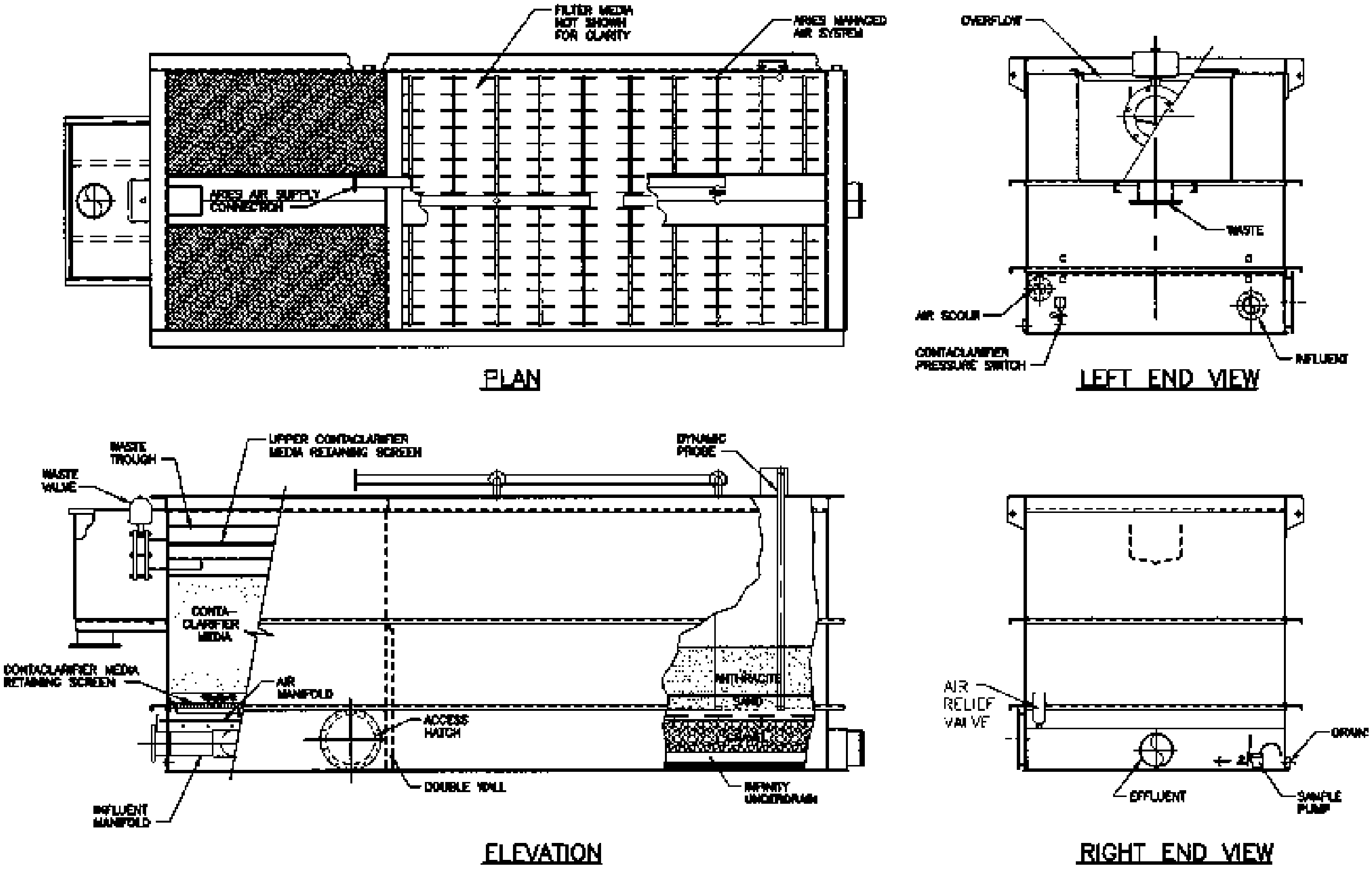


Fig. 25-10. Layout for Pacer II ContaClarifier package plant (Courtesy of Roberts Water Technologies)

Fig. 25-11. Photo of 700-gpm Pacer II package plant (Courtesy of Roberts Water Technologies)

Backwash water and air is distributed through a nozzle underdrain at suitable rates per individual application.

HIGH-RATE CLARIFICATION-FILTRATION PACKAGE SYSTEMS

High-rate clarification using ballasted flocculation and sludge blanket reactors has been combined into package systems. Two systems are described below:

- Actiflo package plant, manufactured by USFilter
- PulsaPak® Package Water Treatment System, manufactured by Infilco Degremont, Inc.

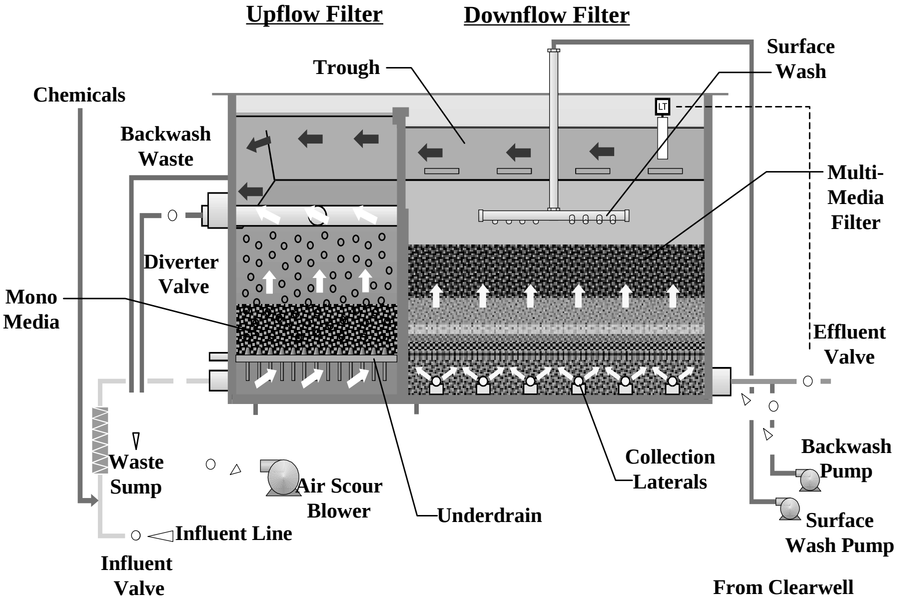


Fig. 25–12. Depiction of Advent water treatment system (Courtesy of Infilco Degremont, Inc.)

Fig. 25–13. Photo of Advent package plant (Courtesy of Infilco Degremont, Inc.)

USFilter / Kruger—Actifloc Package Plants

The Actifloc package plant is based on the ballasted flocculation process described in Chapter 11. It is manufactured as a stand-alone high-rate clarification unit (Actiflo ballasted flocculation) or coupled with a mixed media filter to provide a complete package treatment system. The package plant includes the following unit processes (see Fig. 25–14 for a schematic of the package plant):

- Coagulation using alum or ferric
- Injection systems where a polymer and sand is mixed with the coagulated water
- Maturation to provide mixing for flocculation
- Settling for the fully ballasted particles. Clarified water passes through lamella settling tubes to improve hydraulics and performance
- Sand/floc recirculation with a hydrocyclone for separating the sand and floc particles
- Mixed media filter, complete with backwash pumping and air scour

The Actifloc system is designed for mixing times of 1 to 2 min and maturation times of 4 to 6 min. Filters are operated at 4 to 6 gpm/sf^2.

Complete Actifloc package plants with or without filters are typically designed as available for flow ranges from 0.25 to 2 mgd/tank. Figures 25–15 and 25–16 show a typical package plant. Footprint is 100 to 560 ft^2/tank, including filters.

Infilco Degremont, Inc.—PulsaPak® Package Water Treatment System

Infilco Degremont engineers and provides a package high-rate pulsed sludge blanket clarification/gravity filtration package plant called the PulsaPak®. The PulsaPak® consists of the widely accepted Pulsator® technology and operates in a manner very similar to that of the Superpulsator by Infilco Degremont, Inc., described in Chapter 11. The PulsaPak® package plant includes the following unit processes (see Fig. 25–17 for a cutaway of the package plant):

Coagulation, flocculation, and sedimentation are provided in one unit by means of a pulsed sludge blanket flow regime.

Following chemical addition, a portion of the raw water is collected and released from a vacuum chamber in gentle pulsations. Each pulsation of raw water is evenly distributed along the unit's floor and is mixed with a suspended blanket of previously formed floc.

Lamella tubes are placed in the clarifier above the sludge blanket to remove fine floc and allow higher loading rates than those of a conventional sedimentation process.

Sludge is collected internally and removed by gravity from the clarifier periodically (about twice an hour).

Dual gravity filters are provided for filtration following clarification. The filters can accommodate up to 42 inches of media and are designed for simultaneous or separate air and water backwashing. The standard design is equipped with a nozzle underdrain.

The PulsaPak® package plant is designed for upflow velocities of 1.6 to 1.8 gpm/ft^2. PulsaPak® package plants are available for flow ranges from 125 to 700 gpm. Fig. 25–18 shows a typical package plant.

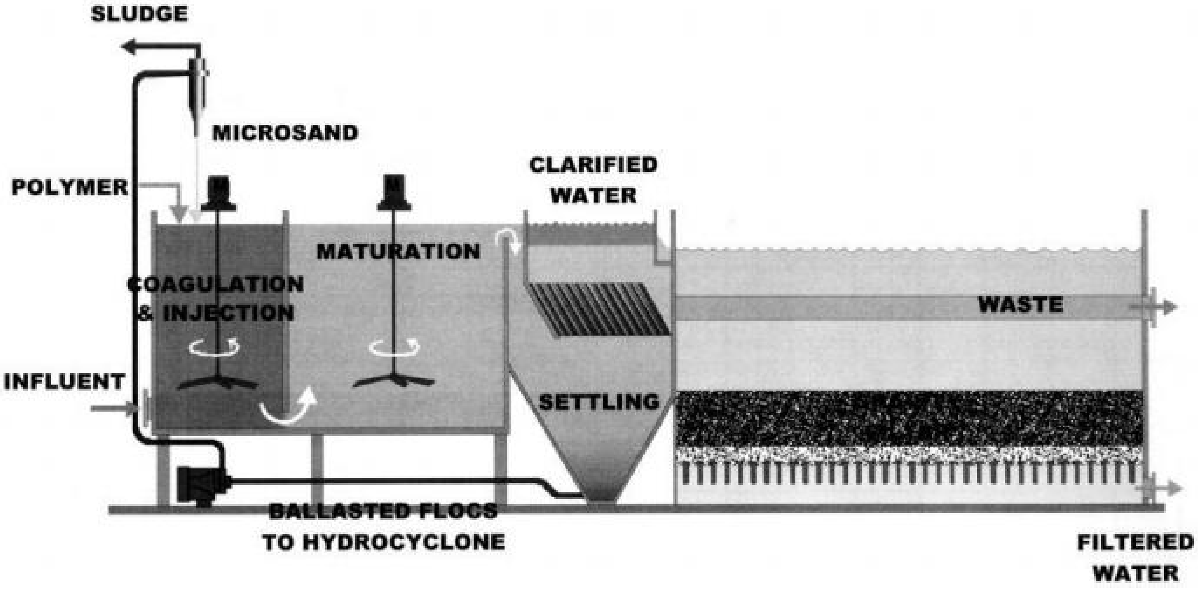
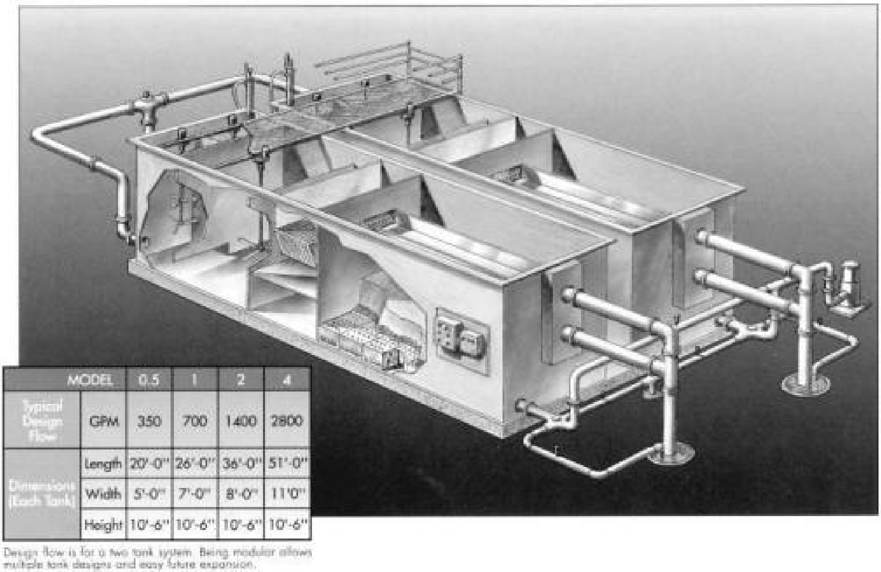


Fig. 25–14. Process schematic of Actifloc package water treatment plant (Courtesy of USFilter/Kruger)



MODEL		0.5	1	2	4
Typical Design Flow	GPM	350	700	1400	2800
Dimensions (Each Tank)	Length	20'-0"	26'-0"	36'-0"	51'-0"
	Width	5'-0"	7'-0"	8'-0"	11'0"
	Height	10'-6"	10'-6"	10'-6"	10'-6"

Design flow is for a two tank system. Being modular allows multiple tank designs and easy future expansion.

Fig. 25–15. Schematic of Actifloc package water treatment plant setup (Courtesy of USFilter/Kruger)

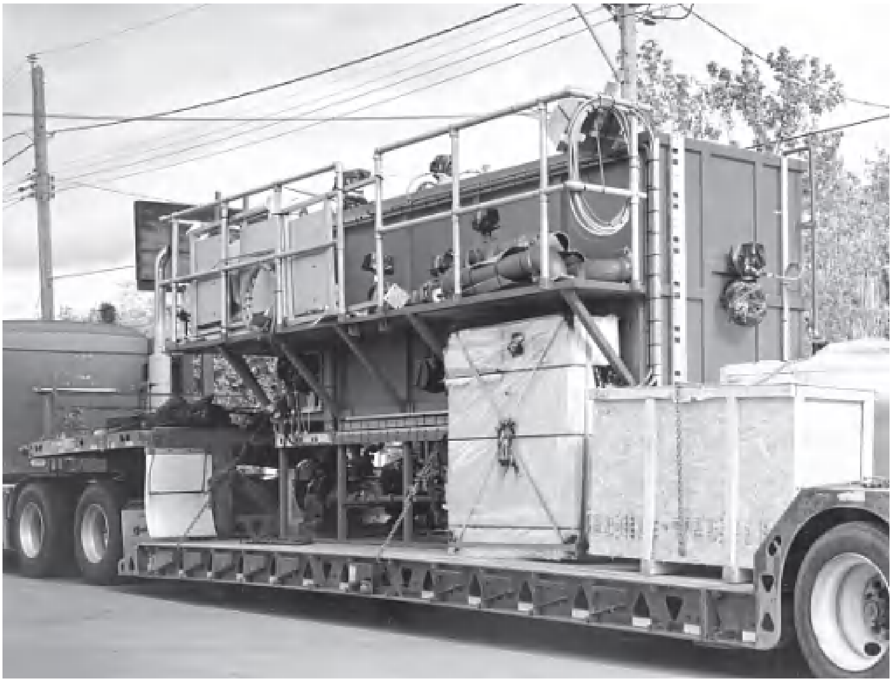


Fig. 25–16. Photo of Actiflo package water treatment plant (without filter) being transported to Cass County, Missouri (Courtesy of USFilter/Kruger)

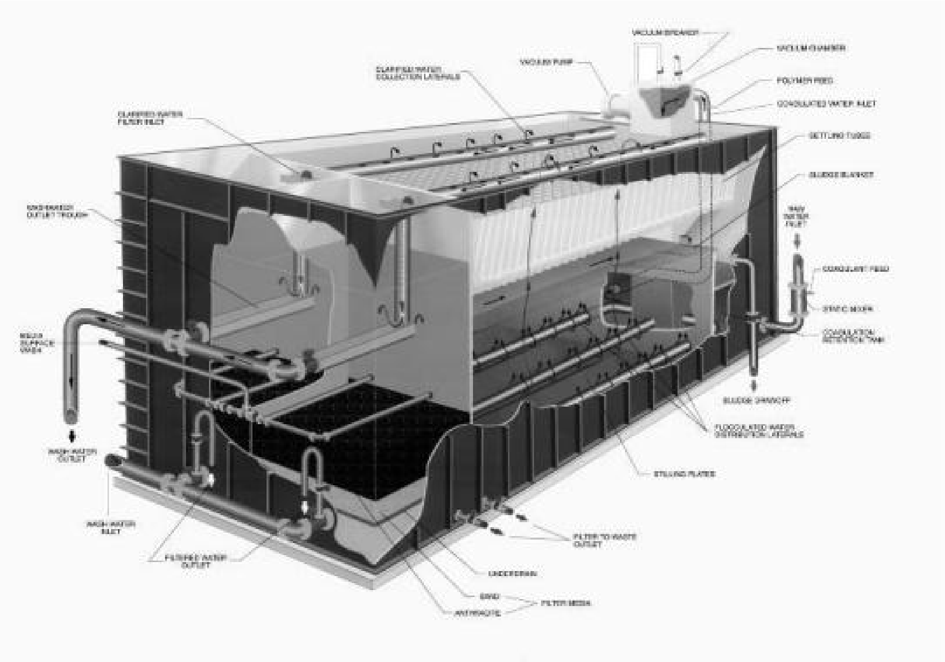


Fig. 25–17. Layout schematic of PulsaPak package water treatment plant (Courtesy of Infilco Degremont, Inc.)

Fig. 25–18. Photo of PulsaPak package water treatment plant. (Courtesy of Infilco Degremont, Inc.)

APPLICATION CRITERIA AND REQUIREMENTS

Before selecting a package system for a particular application, a potential user must be certain that it can produce the required quality and quantity of water from the proposed raw-water supply. Package systems characteristically have limitations (especially those employing high-rate unit processes) related to the quality of the raw-water supply. These limitations must be recognized when one is considering a package system. For example, such factors as low raw-water temperature, high or flashy turbidity, excessive color, or atypical coagulant dosages (higher than expected based upon normal turbidity levels) may influence the selection and rating of a particular package system. The manufacturer's nameplate capacity of a modular unit may have to be downrated or a larger unit selected to handle difficult treatment conditions. Water supplies of consistently high turbidity (greater than 200 NTU) may require presedimentation prior to treatment in a modular unit. A misapplication can occur if the user is not informed of particular equipment limitations for a given treatment requirement.

It is recommended that all records of raw-water quality be reviewed to determine the full range of treatment conditions to be expected before a particular-capacity package system is selected. Especially valuable are laboratory analyses of representative raw-water supplies to provide information critical to a proper application. Under certain conditions, on-site pilot tests may be justified and warranted to verify the suitability of a modular unit. This is especially important because many of the new modular unit designs employ high-rate, short-detention-time unit processes that require close control in order to perform effectively. Advance information on the quality of the proposed raw water supply and its treatment characteristics helps to ensure a successful installation.

OPERATIONAL CONSIDERATIONS

Equally important to the success of modular unit applications is the quality of plant operation. Regardless of the size of the facility, if operating personnel do not possess an adequate understanding of the process and equipment they are responsible for operating, production of safe and palatable finished water may be a hit-or-miss proposition. Some manufacturers have incorporated automatic controls such as effluent turbidity monitors that shut down the plant when the turbidity of the filtered water exceeds a preset limit. This fail-safe device (assuming that it is not bypassed by the operator) ensures that if the plant produces any water at all, it meets a given turbidity standard. An effluent turbidity control device that will shut the system down under certain preset conditions is required in any package treatment system.

Package systems from most manufacturers have controls that automate the chemical feed system to maintain a specified finished water turbidity. This is advantageous where plants do not have full-time operators and raw-water conditions change frequently. The control system is based on a microprocessor-based system that receives plant influent and effluent turbidities as input, and adjusts chemical dosages to optimum levels. Filter effluent turbidity is compared to the plant set point turbidity (for example, 0.1 NTU), and coagulant and coagulant aid dosages are adjusted based upon the deviation observed. If the effluent turbidity is 5 percent higher than the set point turbidity, the dosages are proportionately increased. Likewise, if the turbidity is less than the set point, the coagulant dosage is reduced, thus reducing chemical consumption and related operating costs. The unit can be paced to flow and pH levels, and will adjust chemical dosages of coagulant and pH control chemicals accordingly.

No matter what control systems are used, the burden of producing a safe drinking water rests with the operator. In this regard, there is no substitute for a well-trained operator who has the necessary skills and dedication to operate the equipment properly.

Most package system manufacturers' equipment manuals include at least brief sections on operating principles, methods for establishing proper chemical dosages, instructions for operating the equipment, and troubleshooting guides. An individual who studies these basic instructions and receives a comprehensive start-up and operator training session from the manufacturer's start-up technicians should be able to operate the equipment easily and satisfactorily.

These services are vital to the successful performance of a package water treatment plant and should be a requirement of the package system manufacturer. The engineer designing a package system should specify that start-up and training services be provided by the manufacturer, and also should consider requiring the manufacturer to visit the plant at 6-month and 1-year intervals after start-up to adjust the equipment, review operations, and retrain operating personnel. Further, this program should be ongoing, and funds should be budgeted every year for at least one revisit by the manufacturer.

REFERENCE

1. "California Surface Water Treatment Alternative Filtration Technology Demonstration Report," Edited by R. H. Sakaji, November 23, 1998.

CHAPTER 26

Plant Hydraulics

INTRODUCTION

Plant hydraulics refers to the conveyance of water through the various components in a water treatment plant. Components may include pipes, channels, appurtenances such as valves, gates, flow meters, etc., and the process structures. If the headloss caused by a particular plant component or process unit is not accurately predicted, the plant's and/or unit's operation may be adversely affected or entirely inhibited. Conversely, if the predicted headloss through each plant component or process unit is too conservative or exaggerated, the cost of plant operation may be needlessly increased by higher pumping costs.

Analyzing water treatment plant hydraulics involves numerous elements, each having its own predictive equations to compute headloss through the element. Totaling the headloss through each of the elements provides the head needed to convey water through the plant. The designer should be aware of the principles involved in the formulas so that the limitations of the formulas are recognized and the appropriate formula is used for that application. Because hydraulics is not an exact science, the designer should exercise judgment in the application of hydraulics principles, and the selection of suitable safety factors when computing the headloss.

This chapter will discuss the general design considerations necessary to calculate hydraulic headloss, and also will review the typical hydraulic applications encountered in water treatment plant design. The theoretical development of hydraulic concepts is not discussed in this book and the reader is referred to the references included at the end of this chapter for that information.

GENERAL DESIGN CONSIDERATIONS

Capacity

The initial task in determining plant hydraulics is to determine the headloss through each plant component. Once the design capacity of the plant is determined, the maximum and minimum flows anticipated for each unit should be defined. Considerations for determining the maximum flow should be based on how the plant or component is controlled, as the maximum flow is often greater than the plant capacity. For instance, if a particular unit is controlled by a pump, the maximum flow to that unit would be the pump's maximum capacity, which might not be the same as the plant capacity.

There may also be recycle flows within the plant that would affect the maximum flow to a unit. For instance, a backwash decant recycle system may operate on level control and return to a clarifier periodically at a constant rate. Because this may occur at a time when maximum flow is being experienced by the clarifier, the recycle flow would have to be considered in the hydraulic design of the clarifier. Consideration should be given to such temporary peak conditions when determining the headloss through a particular unit.

When multiple units operate in parallel and one is out of service for maintenance, the remaining units must be capable of hydraulically handling the flow without major disruption to the plant performance. This would apply to all related appurtenances such as feed pipes, valves, and so on.

Units, such as filters, that normally are taken out of service for backwashing should be capable of handling the flows with some units out of service for backwashing, as well as allowing for some units to be out of service for repair or maintenance.

Available Head

Each water plant has unique parameters affecting the available head. These parameters may influence the design of a process, the plant layout, the type of plant control, and the method of water transport. The available head will determine the cost of the headloss. In mountainous or hilly terrain it may be feasible to design a plant without the need for pumping the main flow. In such cases the cost of headloss is negligible. In fact, excessive available head may be used to generate power.

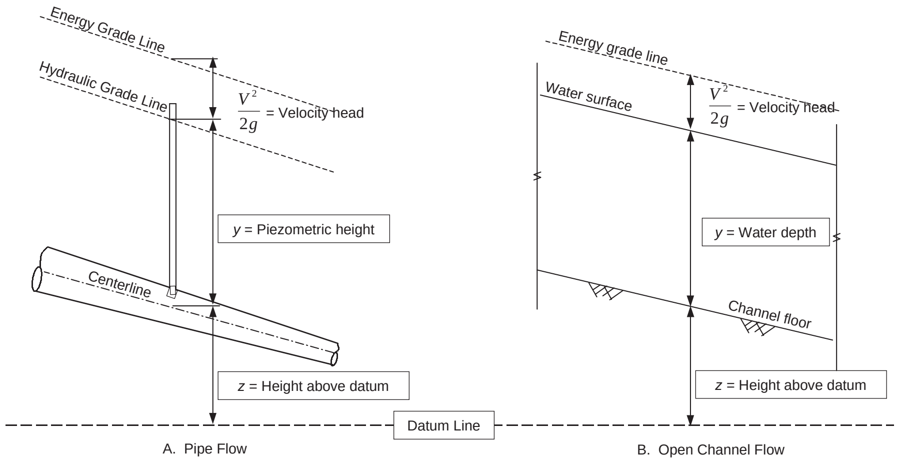
In flat areas or in the treatment of well water, additional head will result in increased operating costs for pumping. At a power cost of $0.075/kWh, and a wire-to-water efficiency of 70 percent, it costs $123/year to pump 1 mgd (3.78 ML/d) 1 foot (0.31 m). At 8 percent interest over a 20-year design life, the present worth cost is approximately $1,200. Therefore, a 50-mgd (200-ML/d) plant with 3 feet (0.91 m) of wasted head results in an equivalent additional present-worth cost of approximately $180,000.

When an existing water plant is expanded or upgraded, the available head is known accurately. Additional new processes in parallel with the existing processes must be designed to match the available head.

COMPONENT DESIGN CONSIDERATIONS

Hydraulic factors that must be addressed include headloss in pipes, hydraulic elements within the conveyance system, and special hydraulic conditions associated with certain treatment processes. The special problems include flow distribution and manifolds, density currents, and to a lesser degree, wind-inducted impacts. These are discussed in the following paragraphs.

There are two types of flow encountered in water plant design: pipe flow and open channel flow. Pipe flow occurs when a conduit flows full, and open channel flow occurs when the flow has a free surface subject to atmospheric pressure. Figure 26–1 illustrates the two types of flow and their components. The hydraulic grade line for pipe flow is represented by the water level in the piezometer (vertical) tubes shown in Figure



A. Pipe Flow

B. Open Channel Flow

Total Head: $H = z + y + \frac{V^2}{2g}$

Headloss between Point 1 and Point 2, H_L

$$H_L = \left(z_2 + y_2 + \frac{V_1}{2g}\right) - \left(z_1 + y_1 + \frac{V_1}{2g}\right)$$

Fig. 26–1. Two types of flow and their components (*Source:* Reference 1)

26–1A. The water levels in the tubes are maintained by pressure exerted by the water in each section of pipe. The hydraulic grade line in an open channel coincides with the water surface, provided a uniform velocity distribution is maintained, and is shown in Figure 26–1B.

The energy grade line, or energy line, represents the total energy in the flow at a particular section. When referred to an arbitrary datum line as illustrated in Figure 26–1, the energy line is the sum of the pipe centerline height (z) (or channel bottom height), the piezometric height (y), and the velocity head ($V^2/2g$). The loss of energy that results when the liquid flows from one point to another is the headloss (H_L).

The hydraulic analysis computes the headlosses (H_L) of all the separate hydraulic elements. The total then represents the elevation to which the water must be pumped in order to allow for gravity flow through the plant. The remainder of this chapter describes and presents the individual hydraulic elements and the equations that are used to compute headloss.

Piping Systems

The headloss in piping systems includes the pipe losses and the losses caused by various hydraulic components in the piping system. The hydraulic components include the following:

- Bends
- Entrance
- Increasers and reducers

- Valves and gates
- Branches
- Orifice
- Venturis

Pipe Flow

The resistance to liquid flow in a pipe results in friction headloss or friction pressure loss. The resistance is caused by turbulence occurring along pipe walls from interior pipe roughness and viscous shear stresses within the liquid. The amount of headloss for a given pipe system depends on several factors:

- Size of pipe
- Pipe interior surface roughness
- Pipe length
- Liquid viscosity

The headloss in pipe due to friction can be approximated by several formulas, including Hazen-Williams, Darcy-Weisbach, or Manning's. The Hazen-Williams formula is:

$$V = 1.318 C_{\mathrm{HW}} R_h^{0.63} S^{0.54} \tag{26–1}$$

$$V\ 0.8949 C_{\mathrm{HW}} R_h^{0.63} S^{0.54} \qquad \text{(metric)} \tag{26–2}$$

where:

V = velocity, ft/sec (m/s)
C_{HW} = Hazen-Williams roughness coefficient, diminsionless
R_h = hydraulic radius, ft (m)
S = energy slope, ft/ft (m/m)

Table 26–1 lists typical values of C_{HW} for pipes of various materials and pipe condition.

The Hazen-Williams formula can also be expressed in a more convenient form as:

$$H_{\mathrm{HW}} = \frac{10.44 L q^{1.85}}{C_{\mathrm{HW}}} \tag{26–3}$$

$$H_{\mathrm{HW}} = 0.1334\, \frac{66.86 L}{C_{\mathrm{HW}}}\, \frac{q^{1.85}}{D^{4.8655}} \qquad \text{metric} \tag{26–4}$$

where:

H_{HW} = headloss, ft (m)
L = length of pipe, ft (m)
C_{HW} = Hazen-Williams roughness coefficient, diminsionless
q = flow, gpm (1/s)
D = pipe diameter, ft (m)

TABLE 26–1. Typical Roughness Values

	Hazen-Williams, C (Unitless)		Darcy-Weisbach, E (Millifeet)	
Pipe Material	Range	Typical Design Value	Range	Typical Design Value
Asbestos cement	140–160	140		0.005
Brass	120–140	130		0.005
Cast iron	80–150	140		0.008
Concrete				
Steel form	85–150	120	1–10	6
Centrifugally spun	90–150	130		1.2
Copper	120–150	130		0.005
Corrugated metal		60		150
Galvanized iron	100–130	120		0.5
Plastic	120–150	140		0.005
Steel	125–140	130	0.1–0.3	0.2

The Darcy-Weisbach formula is:

$$H_{\mathrm{DW}} = f\,\frac{LV^2}{D2g} \qquad (26\text{–}5)$$

where:

H_{DW} = headloss, ft (m)
f = roughness coefficient, which varies with pipe sizes, roughness, velocity, and kinematic viscosity, dimensionless
D = diameter of pipe, ft (m)
L = length of pipe, ft (m)
V = average pipe velocity, ft/sec (m/s)
g = acceleration of gravity, 32.174 ft/sec^2

The value of f is expressed as a function of the Reynolds Number ($\mathrm{Re} = VD/V$, where V is the kinematic viscosity) and can be obtained from the Moody diagram (Fig. 26–2).[3]

The Manning formula is:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (26\text{–}6)$$

$$V = \frac{1.0}{n} R^{2/3} S^{1/2} \quad \text{(metric)} \qquad (26\text{–}7)$$

where:

V = velocity
n = Manning roughness coefficient, dimensionless
R = hydraulic radius, ft (m)
= cross-sectional area of liquid divided by wetted perimeter
S = energy slope, ft/ft (m/m)

Table 26–2 lists typical values of n, for closed conduits flowing partly full for various pipe materials.

The Hazen-Williams formula is the most commonly used formula for water plant hydraulic calculations. Tabulated values of headloss per 1,000 feet can be found in Williams and Hazen.[4] The headloss is given for various pipe sizes, C_{HW} values, and flows, which allows simple calculation of the expected friction losses.

The Darcy-Weisbach equation is sometimes used because the headloss is expressed in terms of a constant times the velocity head, as are many of the other system losses.

Although the Manning formula is more typically used with open channel flow, it can be applied to pipes flowing full.

Hydraulic Component Head Losses

In calculating the headloss in a pressure pipe system, the headloss for each component of the system is determined in terms of the velocity head as shown in Figure 26–3. The headloss can be defined by the following equation:

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Fig. 26–2. Moody diagram (*Source*: Reference 5) (From King, H. W., and Brater, E. F. *Handbook of Hydraulics*, 5th ed. Copyright © 1973. Reproduced with permission of The McGraw Hill Companies.)

TABLE 26–2. Values of the Manning Roughness Coefficient

	Range		Use in Design	
Kind of Conduit	Best	Worst	From	To
Closed Conduit Flow				
Clean uncoated cast-iron pipe	0.011	0.015	0.013	0.015
Clean coated cast-iron pipe	0.010	0.014	0.012	0.014
Dirty or tuberculated cast-iron pipe	0.015	0.035		
Riveted steel pipe	0.013	0.017	0.015	0.017
Loc-bar and welded pipe	0.010	0.013	0.012	0.013
Galvanized-iron pipe	0.012	0.017	0.015	0.017
Brass and glass pipe	0.009	0.013		
Wood-stave pipe	0.010	0.014		
Wood-stave pipe, small diameter			0.011	0.012
Wood-stave pipe, large diameter			0.012	0.013
Concrete pipe	0.010	0.017		
Concrete pipe with rough joints			0.016	0.017
Concrete pipe, ‘‘dry mix,’’ rough forms			0.015	0.016
Concrete pipe, ‘‘wet mix,’’ steel forms			0.012	0.014
Concrete pipe, very smooth			0.011	0.012
Vitrified sewer pipe	0.010	0.017	0.013	0.015
Common clay drainage tile	0.011	0.017	0.012	0.014
Corrugated metal (2⅔ × ½)	0.023	0.026		
Corrugated metal (3 × 1 and 6 × 1)	0.026	0.029		
Corrugated metal (6 × 2 structural plate)	0.030	0.033		
Rock, unlined	0.038	0.041		
Enameled steel	0.009	0.010		
Open Channel Flow				
Uncoated cast-iron pipe	0.012	0.015		
Coated cast-iron pipe	0.011		0.012	0.013
Commercial wrought-iron pipe, black	0.012	0.015		
Commercial wrought-iron pipe, galvanized	0.013	0.017		
Smooth brass and glass pipe	0.009	0.013		
Smooth lockbar and welded ‘‘OD’’ pipe	0.010		0.011	0.013
Riveted and spiral steel pipe	0.013		0.015	0.017
Vitrified sewer pipe	0.010	0.017	0.013	
Common clay drainage tile	0.011	0.017	0.012	0.014
Glazed brickwork	0.011	0.015		0.013
Brick in cement mortar; brick sewers	0.012	0.017		0.015
Neat cement surfaces	0.010	0.013		
Cement mortar surfaces	0.011	0.015		0.013
Concrete pipe	0.012	0.016		0.015
Wood stave pipe	0.010	0.013		
Plank flumes:				
Planned	0.010	0.014	0.012	
Unplanned	0.011	0.015	0.013	
With battens	0.012		0.015	
Concrete-lined channels	0.012	0.018	0.014	0.016
Cement rubble surface	0.017	0.030		
Dry-rubble surface	0.025	0.035		
Dressed-ashlar surface	0.013	0.017		
Semicircular metal flumes, smooth	0.011	0.015		

TABLE 26–2. (*Continued*)

Kind of Conduit	Range		Use in Design	
	Best	Worst	From	To
Semicircular metal flumes, corrugated	0.0225	0.030		
Canals and ditches:				
Earth, straight and uniform	0.017	0.025		0.0225
Rock cuts, smooth and uniform	0.025	0.035		0.033
Rock cuts, jagged and irregular	0.035			
Winding sluggish canals	0.0225	0.030	0.0275	
Dredged earth channels	0.025	0.033		
Canals with rough stony beds, weeds on earth banks	0.025	0.040		0.035
Earth bottom, rubble sides	0.028	0.035	0.030	0.033
Natural stream channels:				
(1) Clean, straight bank, full stage, no rifts or deep pools	0.025	0.033		
(2) Same as (1), but some weeds and stones	0.030	0.040		
(3) Winding, some pools and shoals, clean	0.033	0.045		
(4) Same as (3), lower stages, more ineffective slope and sections	0.040	0.055		
(5) Same as (3), some weeds and stones	0.035	0.050		
(6) Same as (4), stony sections	0.045	0.060		
(7) Sluggish river reaches, rather weedy or with very deep pools	0.050	0.080		
(8) Very weedy reaches	0.075	0.150		

Source: Adapted from reference 5.

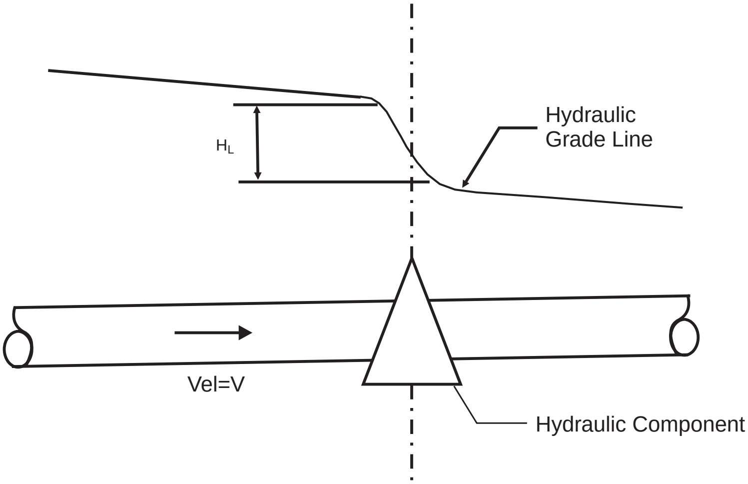

Fig. 26–3. Headloss in a pressure pipe system

$$H_L = K_L \frac{V^2}{2g} \tag{26–8}$$

where:

H_L = headloss, ft (m)
K_L = headloss coefficient, dimensionless
V = velocity, ft/sec (m/s)
g = Acceleration of gravity
= 32.174 ft/sec^2 (9.82 m/s^2)

Table 26–3 lists typical values of losses for a variety of hydraulic components in terms of velocity head. These losses are in addition to pipe friction losses.

A simple example can be used to illustrate the concepts based on a typical piping system to a clarifier, as shown in Figure 26–4.

Entrance

The entrance configuration is commonly a pipe in a tank wall, such as a reservoir outlet box, chemical mixing box, flow split box, junction box, and so forth. The intake may be a sluice gate, a simple sharp-edged entrance, or a streamlined bell-mouthed entrance. The designer must decide which type to use, depending upon upstream head, the importance and cost of headloss, velocities, and downstream condition. If headloss is critical and must be kept to a minimum, the radius of the bell-mouthed entrance should be $D/7$, with D = diameter of the pipe downstream.

The entrance may include a sluice gate or slide gate that is used for isolation of downstream units. For example, if the gate is large enough that it does not constrict the opening, the headloss coefficient (K_L) can be estimated to be approximately 2.0 times the velocity head. If the sluice gate is partially submerged, the K_L value will be approximately 2.5.

The values of K_L vary within the published literature, and median values are shown in Table 26–3. The designer should adopt a set of K_L values for each condition and use them in a consistent manner.

Increasing and Reducing Components

Headloss in pipes that are enlarged or expanded can be approximated by:

$$H_L = K_L \frac{V_1^2 - V_2^2}{2g} \tag{26–9}$$

where:

V_1 = velocity at the entrance to the enlargement, ft/sec (m/s)
V_2 = velocity at the exit from the enlargement, ft/sec (m/s).

In this case, values for K_L depend on the expansion rate, as shown in Figure 26–5. (Fig. 26–5 also includes coefficients for sudden contractions.)

TABLE 26-3. Special Losses of Head (K_L) in Terms of $V^2/2g$

Losses in Pipes, Pipe Fittings, and Valves	
Appurtenance, Alphabetically	K_L
BENDS	
90° Elbow	*Smooth–Rough*
Flanged, regular	0.21–0.30
Flanged, long radius	0.14–0.23
90° Bend	
Screwed, short radius	.9
Screwed, midum radius	.75
Screwed, long radius elbow	.60
Intersection of two cylinders	
(not rounded, e.g., welded pipe)	1.25–1.8
45° Elbow	
Screwed, regular	0.30–0.42
Flanged, long radius	0.18–0.20
Flanged, regular	0.20–0.30
General rule to use ¾ of loss for 90° bend of same radius	
22½° Bend	
Use ½ of loss for 90° bend of same radius	
Standard 45° bend (4–18″)	0.20–0.30
Obtuse-Angled	
Deflection of pipe (θ) $<$ 90°	$1.5\left[\frac{\theta}{90°}\right]^2$
Return Bend	
Flanged, regular	0.38
Flanged, long radius	0.25
Screwed, regular	2.2
Any Bend (*except as above*)	
θ = angle of bend	$0.25\sqrt{\frac{\theta}{90°}}$
Wye Branches or 45° Laterals	
Use ¾ of the loss for a tee, or	1.0
Tee	
Standard, bifurcating	1.50–1.80
Standard, 90° turn	1.80
Standard, run of tee	0.60
Reducing, run of tee of ½	0.90
Reducing, run of tee of ¼	0.75
Use losses for 90° bend with zero radius	

TABLE 26-3. (*Continued*)

Losses in Pipes, Pipe Fittings, and Valves		
Appurtenance, Alphabetically	K_L	
	K_s (Smooth Surface)	K_r (Rough Surface)
Miter Bends		
5° deflection angle	0.016	0.024
10°	0.034	0.044
15°	0.042	0.062
22.5°	0.066	0.154
30°	0.130	0.165
45°	0.236	0.320
60°	0.471	0.684
90°	1.129	1.265
CONDUITS, Closed Pipes or Open Channels		
90° Bends (Velocity 2 to 6 ft/sec)		
0.0 R	1.0–1.4	
0.25 R	0.5–0.6	
0.50 R	0.3–0.4	
$1'$–$8'$ R	0.2–0.3	
$10'$ R	0.3–0.35	
$15'$ R	0.4–0.5	
$20'$ R	0.5–0.6	
$25'$ R	0.55–0.65	
CONTRACTOR, SUDDEN		
$d/D = 1/4$	0.42	
$d/D = 1/2$	0.33	
$d/D = 3/4$	0.19	
ENLARGEMENT, SUDDEN (due to turbulence)		
(V_1 = downstream velocity)		
Sharp-cornered outlet	$1.0(V_2^2/2g - V_1^2/2g)$	
Bell-mounted outlet	$0.1(V_2^2/2g - V_1^2/2g)$	
In terms of velocity of small end		
$d/D = 1/4$	0.92	
$d/D = 1/2$	0.56	
$d/D = 3/4$	0.19	
ENTRANCE	0.83–1.0	
Entrance Losses	0.5	
Pipe projecting into tank (Borda entrance)	0.23	
End of pipe flush with tank	0.04	
Slightly rounded		
Bell-mouthed		
GATES	2.5	
Sluice	0.5	
Submerged port in 12″ wall		
Contraction in a conduit	0.2	
Width equal to conduit width and without top submergence	1.80	
Shear		
Wide open (orifice)		

TABLE 26-3. (*Continued*)

Losses in Pipes, Pipe Fittings, and Valves	
Appurtenance, Alphabetically	K_L
INCREASERS	
(V_1 = velocity of small end)	$0.25(V_1^2/2g - V_2^2/2g)$
Bushing or Coupling	$1.4(V_2^2/2g - V_1^2/2g)$
OBSTRUCTIONS IN PIPES Where A_1/A_0 = ratio of area of pipe to area of opening in obstruction	Note: Values of coefficients below are for the corresponding A_1/A_0 ratios listed (in terms of pipe velocities)
A_1/A_0	Coeffs.
1.05	0.10
1.1	0.19
1.2	0.42
1.4	0.96
1.6	1.54
1.8	2.17
2.0	2.70
2.2	3.27
2.5	4.00
3.0	5.06
4.0	6.75
5.0	8.01
6.0	9.4
7.0	10.4
8.0	11.3
9.0	12.5
10.0	13.5
OPEN CHANNELS	
Sudden Contraction or Inlet Losses	
(In terms of downstream velocity)	
Sharp-cornered entrance	$0.5(V_1^2/2g - V_2^2/2g)$
Round-cornered entrance	$0.25(V_1^2/2g - V_2^2/2g)$
Bell-mouthed entrance	$0.05(V_1^2/2g - V_2^2/2g)$
Turns Around Baffles	3,2
ORIFICE METERS	
Orifice to Pipe Diameter Ratio	
(In terms of pipe velocities)	
1:4 (0.15)	4.8
1:3 (0.33)	2.5
1:2 (0.50)	1.0
2:3 (0.67	0.4
3:4 (0.75)	0.24
OUTLET	
(V_1 = velocity in pipe)	
Outlet	
From pipe into still water or atmosphere (free discharge)	1.0
From pipe to well	$0.9(V_1^2/2g - V_2^2/2g)$
Bell-mouthed outlet	$0.1(V_1^2/2g - V_2^2/2g)$

TABLE 26-3. (*Continued*)

Losses in Pipes, Pipe Fittings, and Valves

Appurtenance, Alphabetically	K_L
REDUCERS	
(Velocity of small end)	
Ordinary	0.25
Bell-mouthed	0.10
Standard	0.04
Bushing or coupling	0.05–2.0
VALVES	
Angle	
Wide open	2.1–2.0
Butterfly	
Fully open	0.30
$\theta = 10°$	0.46
$\theta = 20°$	1.38
$\theta = 30°$	3.6
$\theta = 40°$	10
$\theta = 50°$	31
θ = 60°	94
$\theta = 70°$	320
$\theta = 80°$	1750
Check	
Horizontal lift type	8–12
Ball type	65–70
Swing check	0.6–2.3
Swing check (fully open)	2.5
Diaphragm	
Fully open	2.3
¾ open	2.6
½ open	4.3
¼ open	21.0
Foot	1.5
Gate	
Fully open	0.19
¼ closed	1.15
½ closed	5.6
¾ closed	24.0
Globe	
Fully open	10.0
Plug, Screwed	
(¼ turn from closed to fully open)	
Fully open	0.77
99% open	0.86
98% open	0.95
95% open	1.45
90% open	2.86
80% open	9.6
70% open	28.0
Plug, Globe, or Stop. 600 psi	
Fully open	4.0
¾ open	4.6
½ open	6.4
¼ open	780.0
"Y" or Blow-Off	2.9

TABLE 26–3. (*Continued*)

Losses in Pipes, Pipe Fittings, and Valves

Appurtenance, Alphabetically	K_L	
VENTURI METERS		
The loss of head occurs mostly in, and downstream from the throat.		
(Losses are in terms of throat velocities.)		
Loss Between Upstream End and Throat	0.03–0.06	
Total Loss through Meter for:	>0.5 Diam. of Pipe.	0.33 to 0.5 Diam. of Pipe
Total Angle of Divergence = +5°	1/7	1/10
Total Angle of Divergence = +15°	1/3	1/16
Long Tube		
(Upstream angle of 10.5° and a downstream angle of 2.5°)		
Pipe Diameter		
6″	0.135	
10″	0.126	
16″	0.122	
20″	0.119	
24″	0.116	
30″	0.113	
36″	0.111	
42″	0.110	
48″	0.109	
60″	0.107	
Throat-to-Inlet diameter Ratio in Terms of Inlet (pipe) Velocity	1.0–1.2	
(1:3) 0.33	0.44–0.52	
(1:2) 0.50	0.25–0.30	
(2:3) 0.67	0.20–0.23	
(3:4) 0.75	0.129	
Eccentric or Flat Invert		
Concentric		
(Throat-to-Inlet Diameter Ratio)	0.27	
4/12 = .33	0.18	
6/12 = .50	1.43	
7/12 = .58	0.14	
8/12 = .67	0.135	
9/12 = .75	0.283	
Eccentric or Flat Invert		

(From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems*. Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

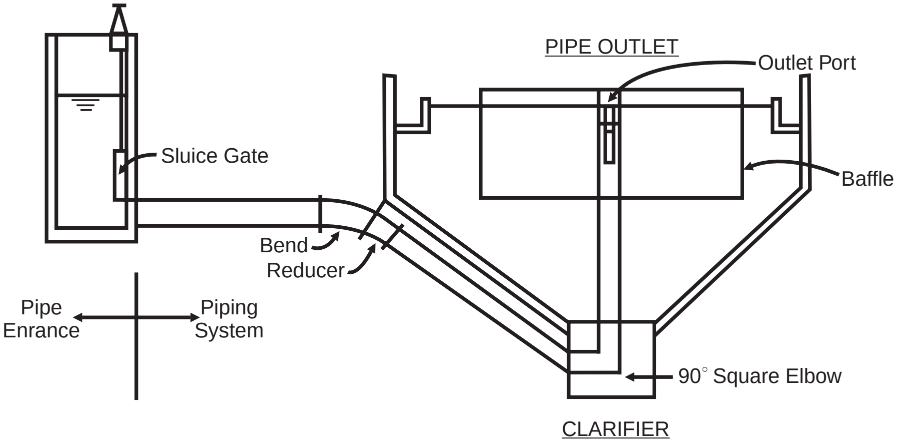

Fig. 26–4. A typical piping system

Miller[7] developed curves for sudden contractions and expansions, which are shown in Figures 26–6 and 26–7, respectively. The loss coefficients are based on the mean velocity bead in the smaller pipe.

The K_L value for a sudden large increase is approximately 1.0. Sudden enlargements can be effective head reducers, and they can also withstand cavitation without significant vibration and damage to the system because the cavitation will take place in the fluid. Figure 26–8 illustrates the fluid action in a sudden enlargement.[6] The diameter ratio of a sudden enlargement controls the severity of the headloss and cavitation potential. The angle of enlargement should be kept below 4 degrees for minimal headloss and cavitation.

Orifices

Orifices are used in pipelines to create a headloss and/or measure the flow rate. They are often used in filter underdrain design to create a headloss during the backwash cycle to achieve an even distribution of backwash water to filter media. The headloss can be approximated using equation 8, where K_L values are based on the pipe-to-orifice area ratios, which typically range from 0.9 to 0.15. Figure 26–9 illustrates K_L values based on area ratios. If an orifice is used, the cavitation potential should be investigated.

Nozzles

Nozzle headloss can be defined in a manner similar to that for orifices. The headloss coefficient values (K_L) range from 0.15 to more than 100 for area ratios from 0.8 to 0.1. Figure 26–9 illustrates K_L values based on area ratios for nozzles. Nozzles, like orifices, should be checked for cavitation potential.

Venturi

Headlosses for Venturi tubes are determined in a manner similar to those for orifices and nozzles. The headloss in Venturi tubes is typically lower than those of orifices and

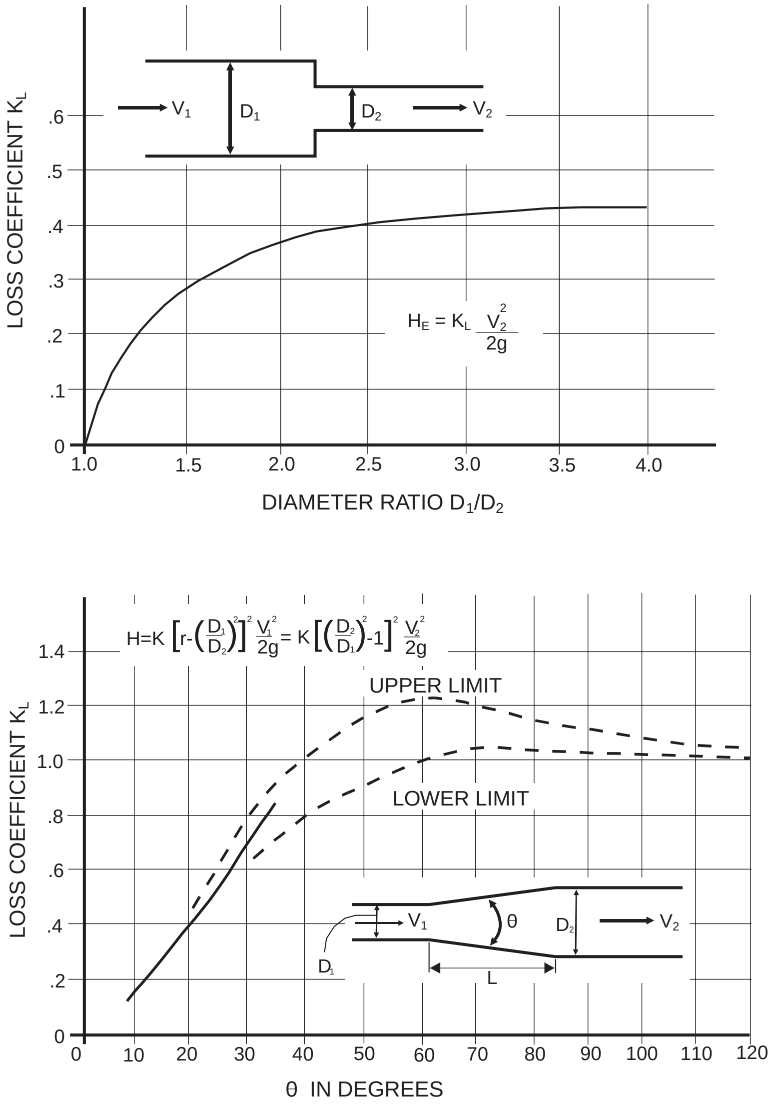

Fig. 26–5. Headloss coefficients for contractions and enlargements (*Source:* Reference 6; courtesy of the Hydraulic Institute)

nozzles for the flow-measuring Venturi tubes. Typical K_L values vary from 0 to 10 for area ratios of 0.8 to 0.1 for recovery angles from 5 to 7 degrees. Figure 26–10 illustrates the range of K_L values based on area ratios and recovery angles.

Miller[7] developed similar curves for a variety of differential flow meters. These are shown in Figure 26–11.

Image Not Available

Fig. 26–6. Abrupt contraction loss coefficients (based on velocity in A_2) (*Source:* Reference 6; courtesy of the Hydraulic Institute)

Manifolds

Manifolds are typically used in filter underdrains and for filter influent and effluent gallery piping. The manifold may be defined as a series of lateral pipes that connect to a main pipe. There are two types of manifolds: dividing flow and combining flow. Combining flow manifolds act differently from dividing flow manifolds. The headloss

Image Not Available

Fig. 26–7. Abrupt expansion loss coefficient (based on velocity in A_1) (*Source:* Reference 6; courtesy of the Hydraulic Institute)

Image Not Available

Fig. 26–8. Flow action in sudden enlargement (*Source:* Reference 6; courtesy of the Hydraulic Institute)

in the main pipe of a combining flow manifold is caused by the mixing of flows, and is expressed as a headloss coefficient times the downstream velocity head. Blaisdell and Mansen[8] have verified the following equation for the headloss coefficient in the main pipe caused by a junction.

$$K_j = 2\,\frac{Q_B}{Q_M} = \left(1 + 2\,\frac{A_M}{A_B}\cos\theta\right)\left(\frac{Q_B}{Q_M}\right)^2 \qquad (26\text{–}10)$$

where:

A_M = area of the downstream main pipe, sq ft (m^2)
Q_M = flow of the downstream main pipe, cu ft/sec (m^3/s)
A_B = area of the branch or lateral pipe, sq ft (m^2)
Q_B = flow of the branch or lateral pipe, cu ft/sec (m^3/s)
θ = angle of convergence between the main and branch pipes, degrees

When manifolds are used for filter underdrain systems, they serve as combining flow manifolds under normal operation, and dividing flow manifolds under backwashing conditions. Filter inlet manifolds and backwash line manifolds are dividing flow types, while filter effluent manifolds are combining flow manifolds. Dividing flow manifolds are often used to distribute flow to a series of successive lateral outlet ports. Dividing flow manifolds commonly discharge to filters or sedimentation basins, or distribute backwash water to filter bottoms.

Although it has been common practice to attempt to distribute flow evenly in this manner, this is not recommended without the use of a control device. Studies have been performed to attempt to define the mechanics of dividing flow manifolds for the purpose of determining the flow splits.[9,10] It can be concluded from these analyses that it is not practical to divide the flow by manifold construction only. It is recommended

Image Not Available

Fig. 26–9. Loss coefficients for orifices and nozzles (*Source:* Reference 6; courtesy of the Hydraulic Institute)

Image Not Available

Fig. 26–10. Headloss coefficients for Venturi tubes (*Source:* Reference 6; courtesy of the Hydraulic Institute)

that a control device such as an orifice or throttling valve be used to impart sufficient headloss in the laterals to equalize the imbalanced manifold losses. For example, in a five-lateral manifold the flow split between the first and last lateral may be 14 and 25 percent, respectively. A control device would be necessary to throttle lateral 5 sufficiently to reduce its flow to 20 percent, while allowing lateral 1 to increase to 20 percent.

The manifold should be analyzed for all extreme flow conditions of its operation. The operating flows may vary considerably and result in significant difference in headloss. The manifold should be sufficiently large to minimize velocities and thereby reduce inaccuracies in the calculation.

Dividing and Combining Flows

Miller[7] evaluated combining and dividing flows and presented a series of curves to estimate the headloss coefficient. Based on the flow arrangements shown in Figure 26–12, the loss coefficients are defined as:

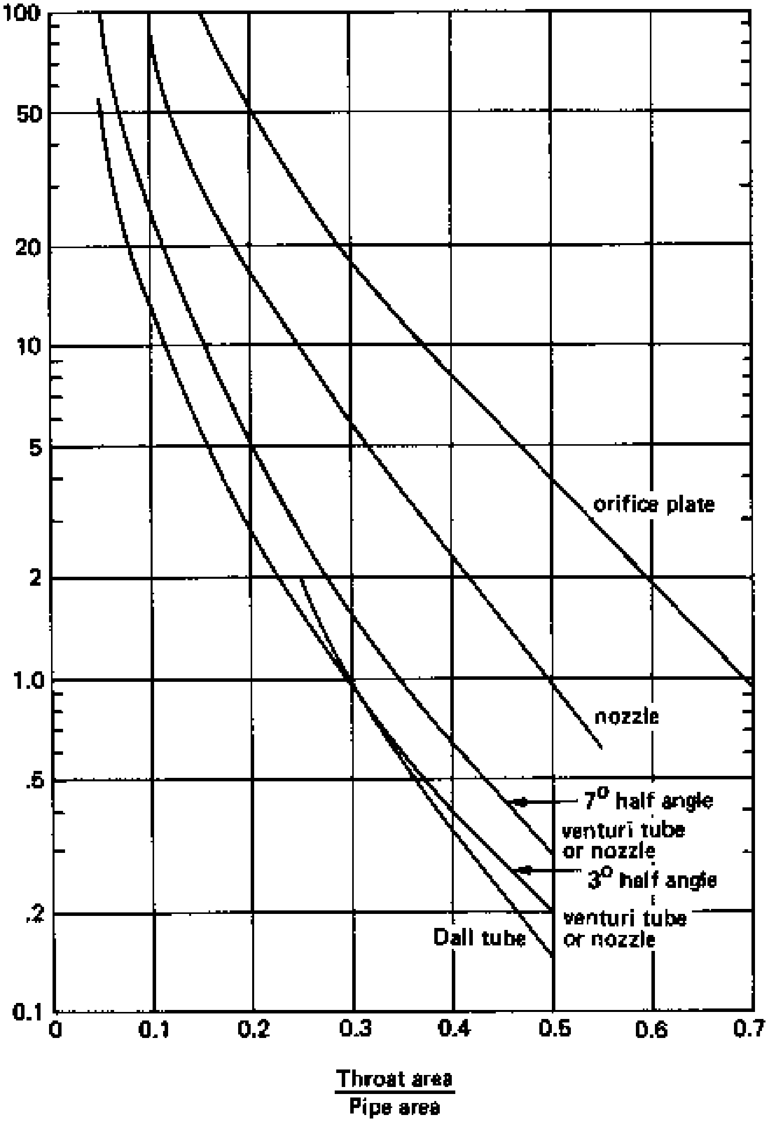

Fig. 26–11. Loss coefficients for differential flowmeters (*Source:* Reference 7; courtesy of the BHR Group Limited)

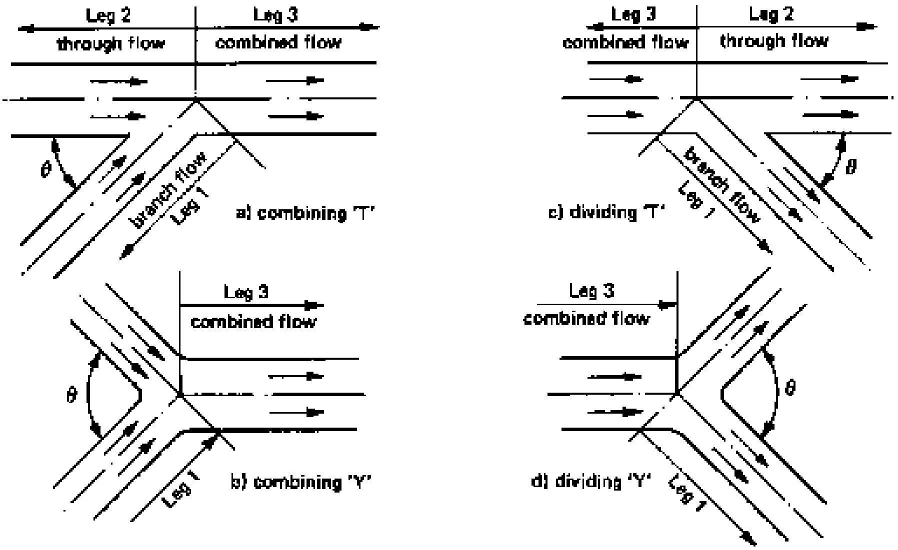

Fig. 26–12. Geometric parameters for combining and dividing junctions (Courtesy of the BHR Group Limited)

Combining flow:

$$k_{13} = \frac{(U_1^2/2g + h_1) - (U_3^2/2g + h_3)}{U_3^2/2g} \tag{26–11}$$

$$k_{23} = \frac{(U_2^2/2g + h_2) - (U_3^2/2g + h_3)}{U_3^2/2g} \tag{26–12}$$

Dividing flow:

$$k_{31} = \frac{(U_3^2/2g + h^3) - (U_1^2/2g + h_1)}{U_3^2/2g} \tag{26–13}$$

$$k_{32} = \frac{(U_3^2/2g + h_3) - (U_2^2/2g + h_2)}{U_3^2/2g} \tag{26–14}$$

The loss coefficient curves developed by Miller[7] are reproduced here as Figures 26–13 to 26–24 (pages 911–922).

Outlets

The outlets to a piping system typically involve a control structure or a basin. Basins, such as flocculation basins, clarifiers, filters, and wet wells, will result in the loss of all velocity head available in the pipe system. The headloss coefficient is simply 1.0.

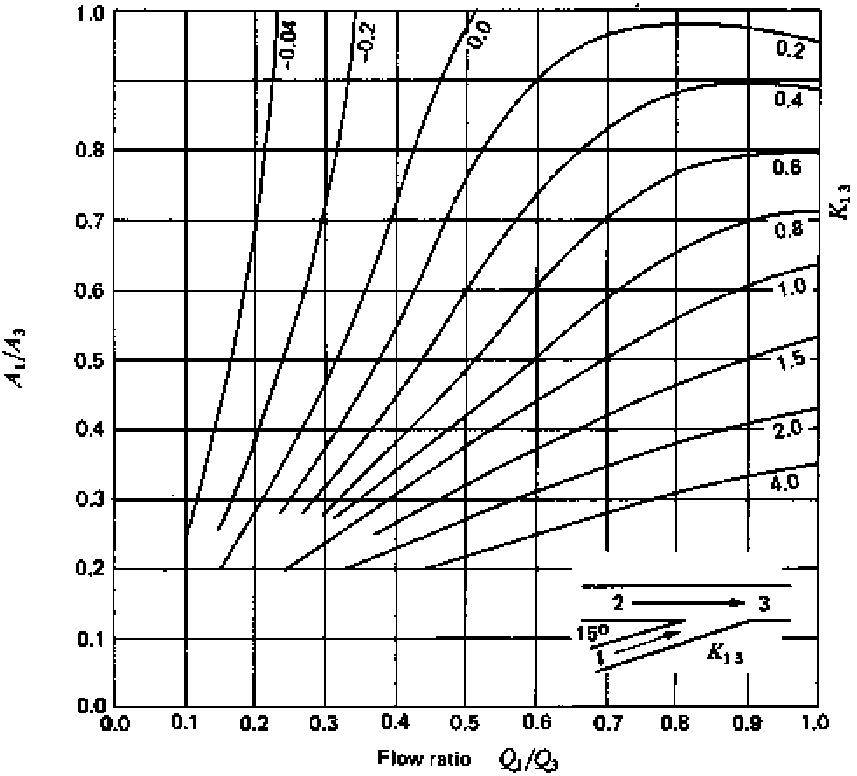

Fig. 26–13. Combining flow—branch angle 15°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

Control structures may include such devices as gates, butterfly valves, fixed cone valves, tube valves, needle valves, or hollow-jet valves. The headloss attributable to a control device at the end of a conduit that discharges to atmosphere can be given as:[11]

$$H_c = \left(\frac{1}{C^2} - 1\right)\frac{V^2}{2g} \tag{26–15}$$

where:

H_c = headloss of control device, ft (m)
C = coefficient of discharge
V = pipe velocity, ft/sec (m/s)
g = acceleration of gravity, 32.174 ft/sec^2 (9.81 m/s^2)

The coefficient of discharge, C, varies with the type of control opening. Equation 26–15 expressed in terms of a headloss coefficient (K_L) would be:

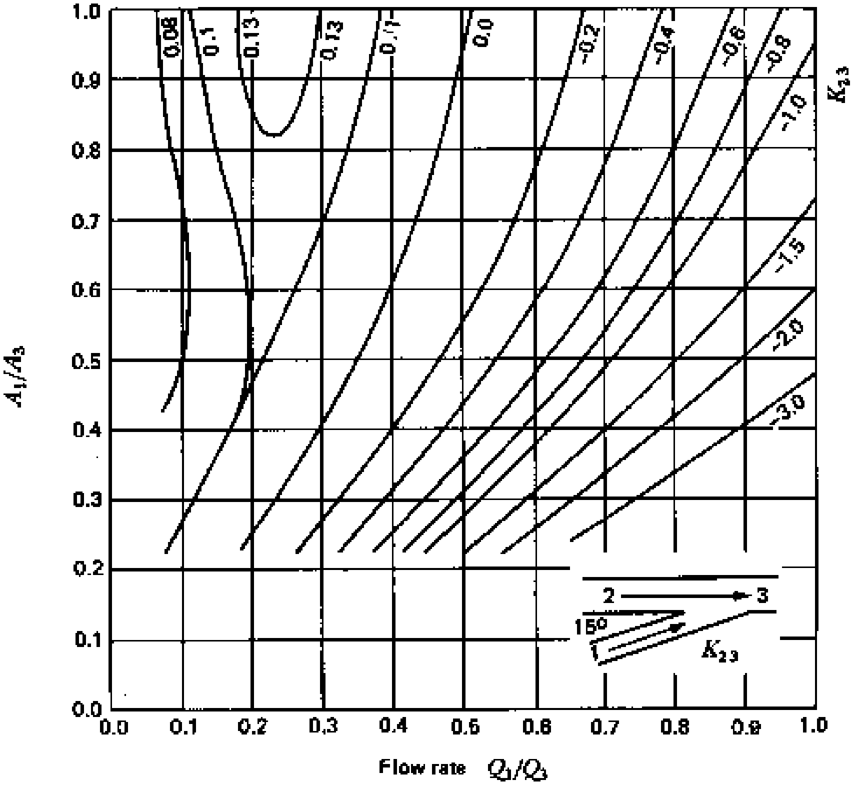

Fig. 26–14. Combining flow—branch angle 15°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

$$H_C = K_L \frac{V^2}{2g} \tag{26–16}$$

Various headloss and discharge coefficients for several types of outlet control devices are given in Table 26–4 (page 922).

Valves

Many different types of valves are used in water treatment plants. Some are used to isolate components, but others to modulate control flows. The designs of these valves differ between manufacturers, and in some instances, valves of different sizes from the same manufacturer may even differ in design. Therefore, to obtain accurate values for the loss coefficient for a particular valve, the manufacturer of that valve should be consulted. However, Miller[7] has developed the loss coefficient for several different types of valve, and these are reproduced here as Figures 26–25 to 26–29 (pages 923–926).

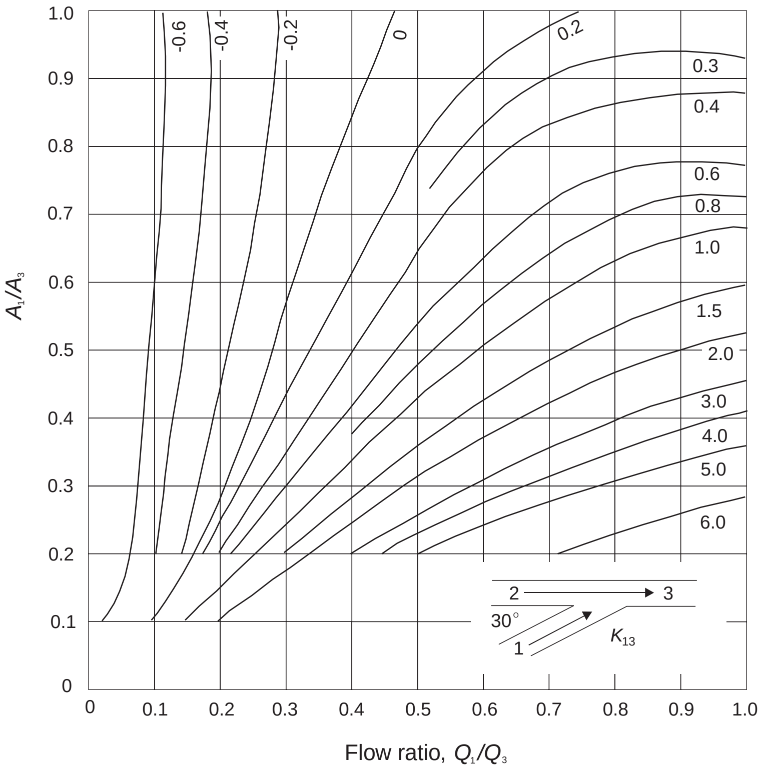

Fig. 26–15. Combining flow—branch angle 30°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

Throttling and Control Valves

Valves used to control the flow rate by throttling need to be sized correctly for satisfactory operation. Some valves are better suited for throttling than others. For better control, an ideal control valve will tend to have uniform effect on flow throughout its movement, which is difficult to achieve. Figure 26–30 (page 927) shows the flow coefficient for a variety of valves. The flow coefficient, C_v, represents the flow of water in gallons per minute with a 1-psi pressure drop through the valve.

The objective for sizing a throttling valve is to limit its operation or movement to a zone where the control is stable and repeatable. Butterfly valves are not an ideal throttling valve, but nevertheless, they are commonly used for filter effluent flow control and other throttling applications. The effective control zone for a butterfly valve is within the range of 25° to 70° open. To properly size a butterfly valve for throttling water flow rate, the following formula is used:

$$Q = C_v \sqrt{\Delta p} \qquad (26\text{–}17)$$

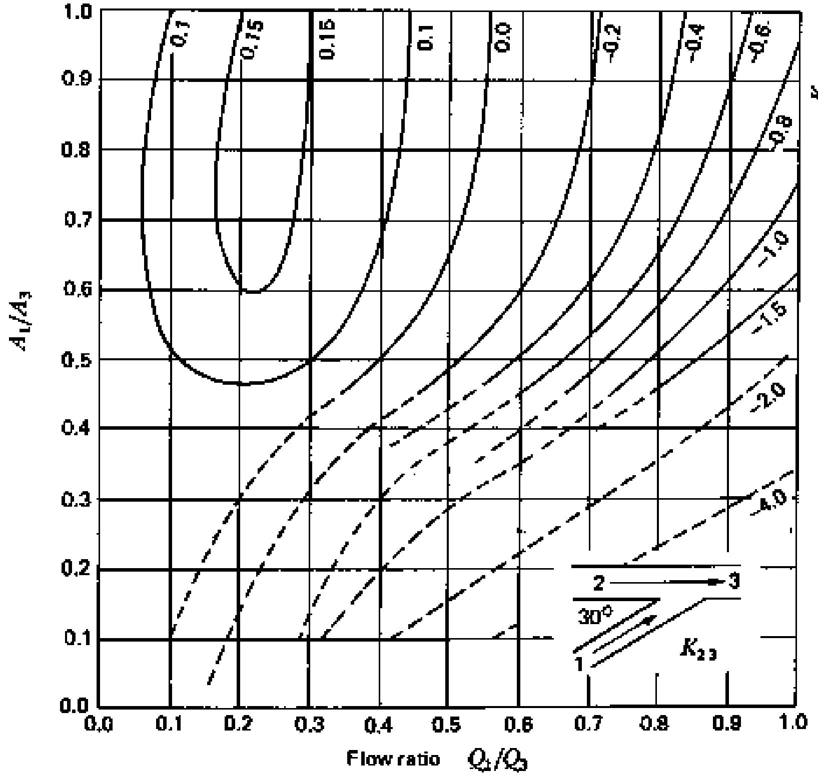

Fig. 26–16. Combining flow—branch angle 30°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

where:

Q = flow rate, gallons per minute.
Δp = pressure drop across the valve, psi
C_v = flow coefficient

The conditions desired to be controlled are usually well defined. The flow rate range that the user wishes to control and the variation in upstream and downstream water level that will be experienced can be defined. From these known conditions, the range in C_v values are calculated. The range of calculated C_v values then are compared to the C_v values for various valve sizes to enable the user to select the appropriate valve size.

If the control valves are going to dissipate a high head, then cavitation may be experienced. Care must be taken to avoid operation of valves under conditions of cavitation.

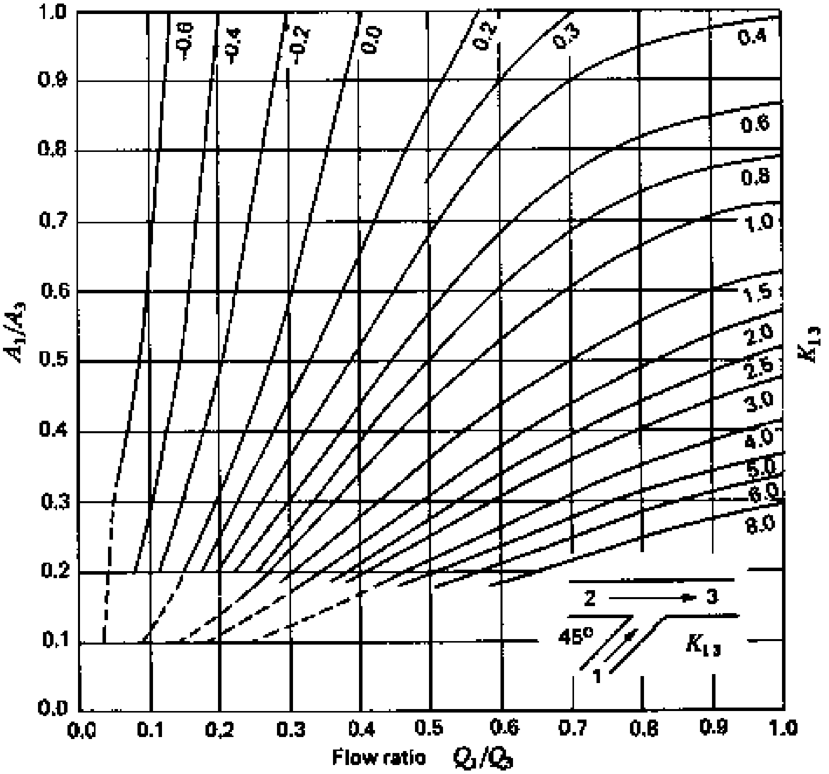

Fig. 26–17. Combining flow—branch angle 45°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

Cavitation

This section gives a general discussion of cavitation. For a more comprehensive treatment of the topic, see References 12 and 13.

Cavitation occurs when the dynamic conditions in a piping system may cause the static pressure to fall to the vapor pressure. Under these conditions, vapor bubbles, or "cavities," form. As these "cavities" pass into a downstream zone with a higher static pressure, the cavities collapse. The collapsing bubbles or cavities are termed cavitation, and are responsible for the characteristic noises associated with cavitation. Cavitation may:[7]

- Restrict flow
- Cause erosion and failure
- Generate unacceptable noise
- Cause instabilities within the system

There are four broad categories of corrosion:

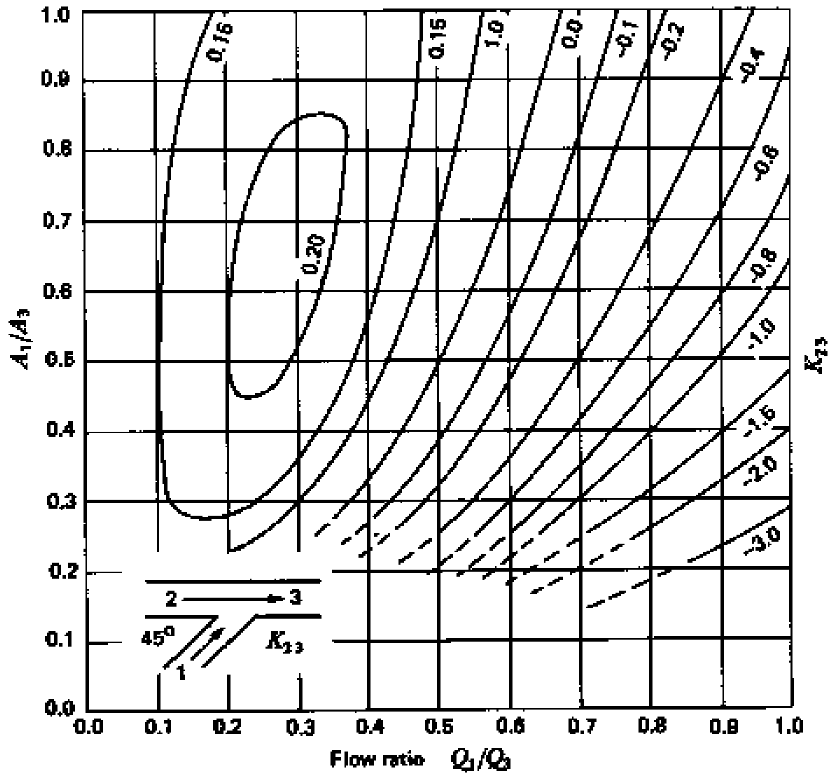

Fig. 26–18. Combining flow—branch angle 45°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

- Bulk cavitation
- Flow curvature cavitation
- Surface roughness cavitation
- Turbulence cavitation

For a detailed discussion of cavitation, the reader is referred to Miller.[7] For this text, the equations and curves presented by Miller are reproduced here.

Cavitation of Butterfly Valves The incipient velocity (U_{ir}) and critical velocity (U_{cr}) are shown in Figure 26–31 (page 928). The base conditions are a valve diameter of 0.321 m and an upstream head minus the vapor pressure head of 50 m. To correct these for other valve sizes and head use:

$$U_i \text{ or } U_c = C_1 U_r \left[\frac{h_u - h_{fv}}{50}\right]^{0.39} \tag{26–18}$$

The value for C_1 is shown in Figure 26–32.

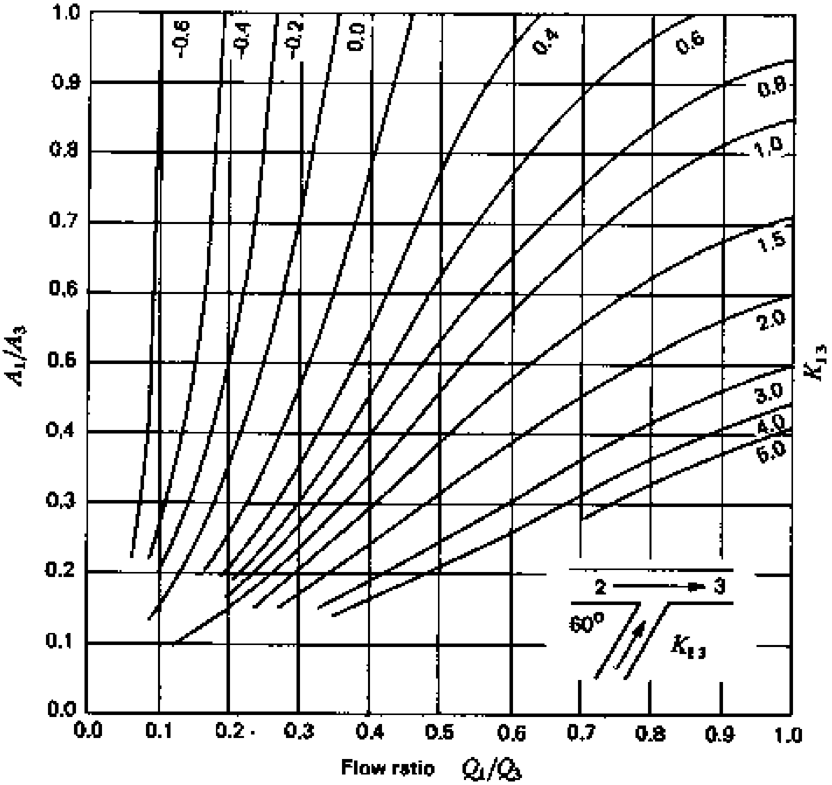

Fig. 26–19. Combining flow—branch angle 60°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

Cavitation of Ball Valves The critical cavitation velocities, U_{cr}, for ball valves are shown in Figure 26–33 (page 929). The velocities are corrected to other head conditions by using:

$$U_c = U_{cr}\left[\frac{h_u - h_v}{50}\right]^{0.4} \tag{26–19}$$

Cavitation of Orifices The mean velocities associated with incipient, critical, incipient damage, and choking cavitation are shown in Figure 26–34 (page 929).[7] The base conditions are a pipe diameter, *D*, of 0.076 m, an upstream head of 63 m gage and a vapor head, h_v, of −8.6 m gage.

To correct the velocities to other conditions, use the following equation:

$$U_i \text{ or } U_c = C_1 U_r \left[\frac{h_u h_v}{71.6}\right]^{0.5} \tag{26–20}$$

where C_1 is taken from Figure 26–35 (page 930).

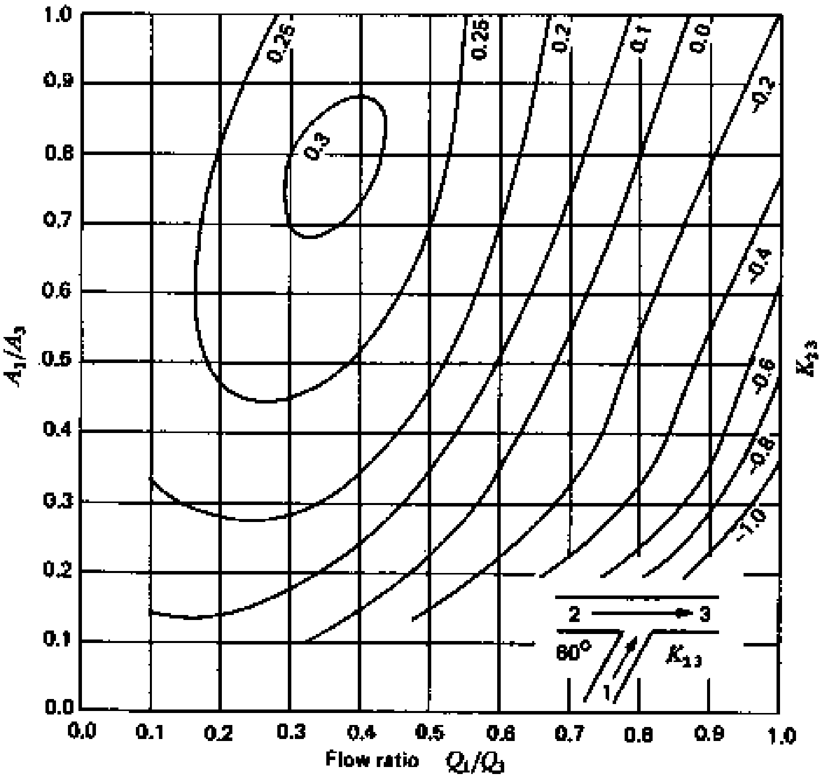

Fig. 26–20. Combining flow—branch angle 60°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

Another method of evaluating valve cavitation is presented in the following sections. The cavitation index for valves has been defined as:[14]

$$\sigma = \frac{P_D - P_V}{P_U - P_D} \tag{26–21}$$

where:

P_D = absolute pressure downstream of element, psia
P_U = absolute pressure upstream of element, psia
P_V = vapor pressure of liquid, psi

The properties of water are tabulated in Appendix E.

Cavitation can also occur in areas where the velocity is high and the pressure is low, such as in a nearly closed valve, which may occur at a control element (valve, gate, etc.). When water passes a flow constriction, such as a throttling valve, the velocity of flow increases, accompanied by a reduction in pressure. If the downstream pressure falls below the vapor pressure of the water, vapor bubbles will develop, and cavitation occurs.

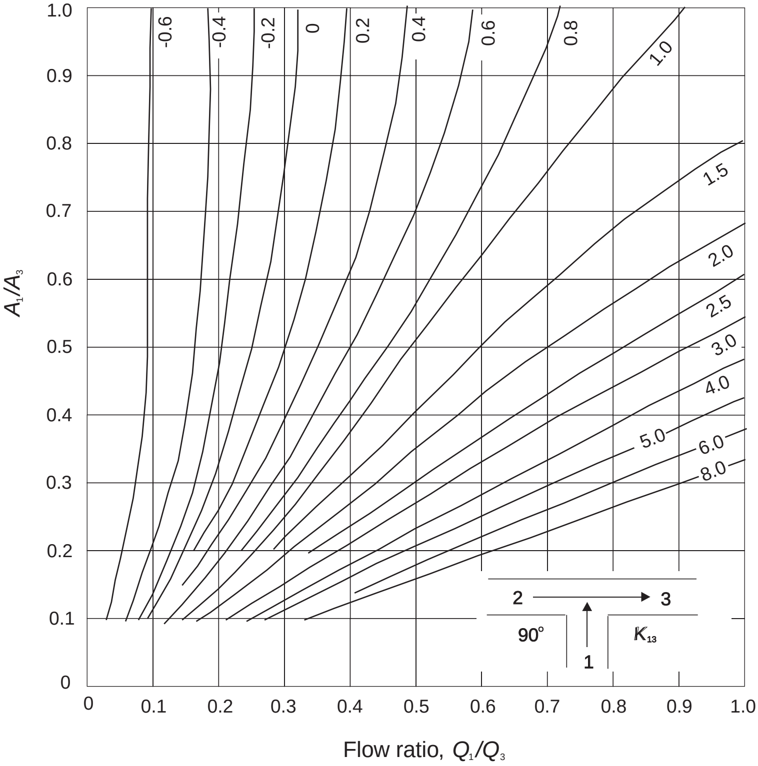

Fig. 26–21. Combining flow—branch angle 90°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

To avoid cavitation, the cavitation index, should exceed the values in Table 26–5 (page 930) or Table 26–6 (page 931). For example, consider a single-seated globe valve and the following conditions:

Fluid:	Water
Temperature:	70°F
Vapor Pressure, P_v:	0.363 psi
Upstream Pressure, P_U:	180 psi
Downstream Pressure, P_D:	50 psi

The value for σ is

$$\sigma = \frac{P_D - P_v}{P_U - P_D} = \frac{50 - 0.36}{180 - 50} = 0.38$$

The value attained indicates that cavitation may be a problem and the use of an alternate valve may be necessary. These values provide only a general guideline based on earlier work for potential cavitation problems and should be used with caution. The

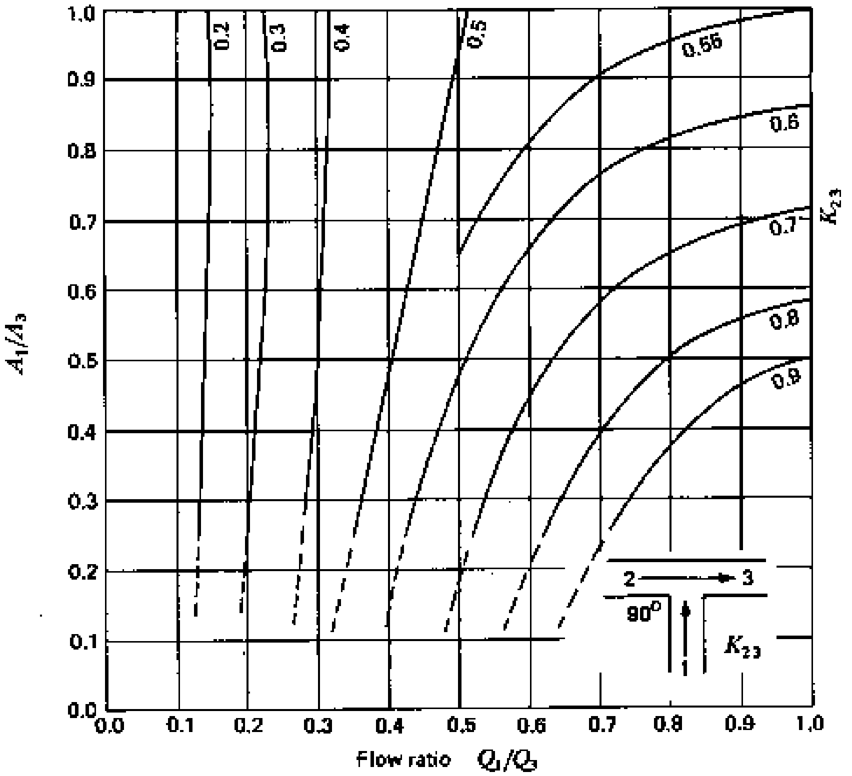

Fig. 26–22. Combining flow—branch angle 90°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

onset of cavitation depends on the pressure, size, opening, and manufacturing of valves and pumps. For a more comprehensive discussion, see References 12 and 13.

Pump Cavitation

Pump cavitation occurs when the absolute pressure at the inlet of the pump drops below the vapor pressure of the liquid being pumped. When the liquid or water enters the eye of the impeller, the velocity increases and is accompanied by a drop in pressure. If the pressure drops below the vapor pressure of the water, the water will vaporize and the pumped liquid will consist of water and bubbles and/or vapor pockets. As the water moves through the pump (impeller), it reaches an area of higher pressure and the bubbles/vapor pockets collapse. If this collapse occurs on a solid boundary, which in this case is the impeller, severe damage will occur and the pump will lose capacity.

Thoma's cavitation number is:[15,16]

$$\sigma = \frac{KN_s^{4/3}}{10^6} \tag{26–22}$$

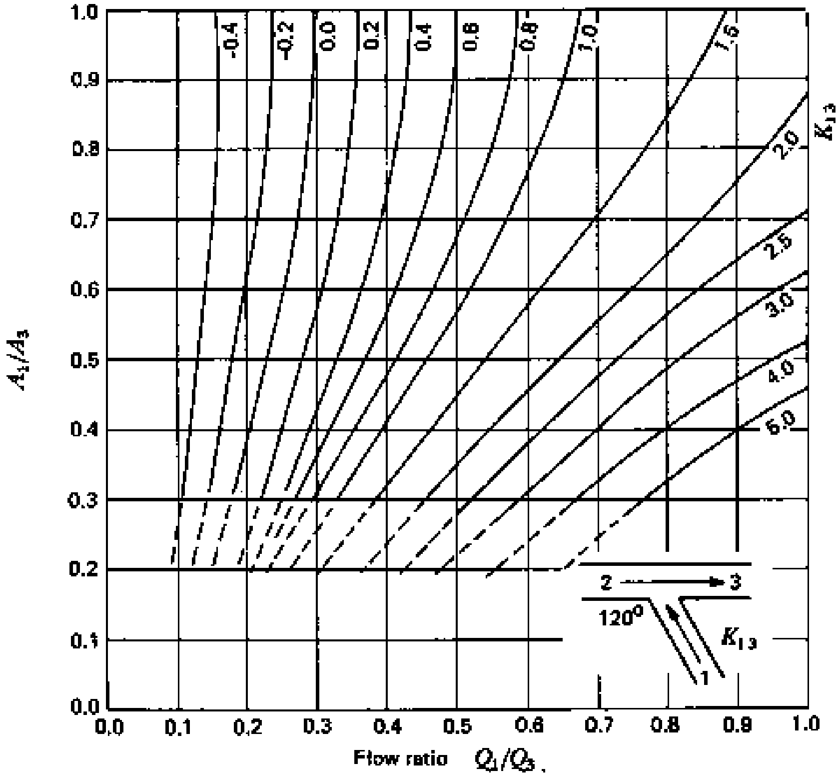

Fig. 26–23. Combining flow—branch angle 120°—loss coefficient K_{13} (*Source:* Reference 7; courtesy of the BHR Group Limited)

where:

σ = Thoma's constant
K = constant for a pump type (single end suction)
= 6.3 (for n in English units) (range of 4.3 to 9.4)
N_s = specific speed (in English units)

$$= \frac{\text{rpm}\sqrt{Q}}{H^{3/4}}$$

rpm = revolution per minute
Q = flow rate, gpm
H = pumping pressure, ft

The cavitation index provides an indication whether the conditions under which the pump is operating will cause the pump to cavitate. In water treatment plant design practice, conditions seldom will require a rigorous analysis of cavitation, but there are occasionally situations that may require an analysis. Rutschi presents an equation based

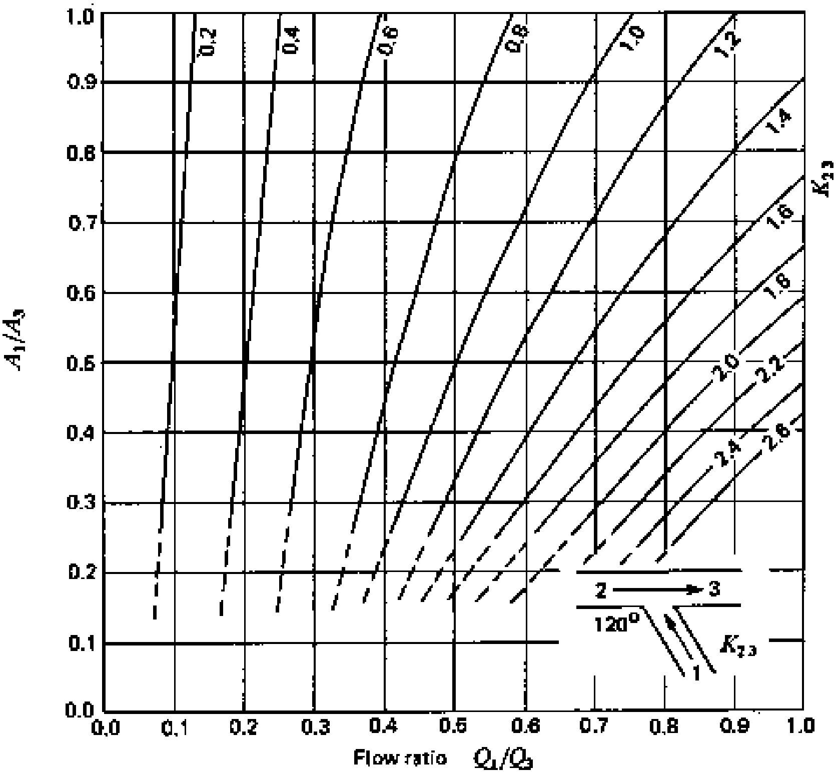

Fig. 26–24. Combining flow—branch angle 120°—loss coefficient K_{23} (*Source:* Reference 7; courtesy of the BHR Group Limited)

TABLE 26–4. Headloss and Discharge Coefficients

Type of Gate or Valve	Headloss Coefficient*
Sluice gate	0.2 to 0.8
Gate valve	10 to 0.20
Butterfly valve	0.16 to 0.5
Cone valve	0.04
Angle Valve	1.8 to 5
Globe Valve	4 to 10
Plug valve	0.5 to 1
Check valve (swing)	0.6 to 2.5
Diaphragm or pinch valve	0.2 to 2.3
Ball valve	0.04

*Based on valves fully open.

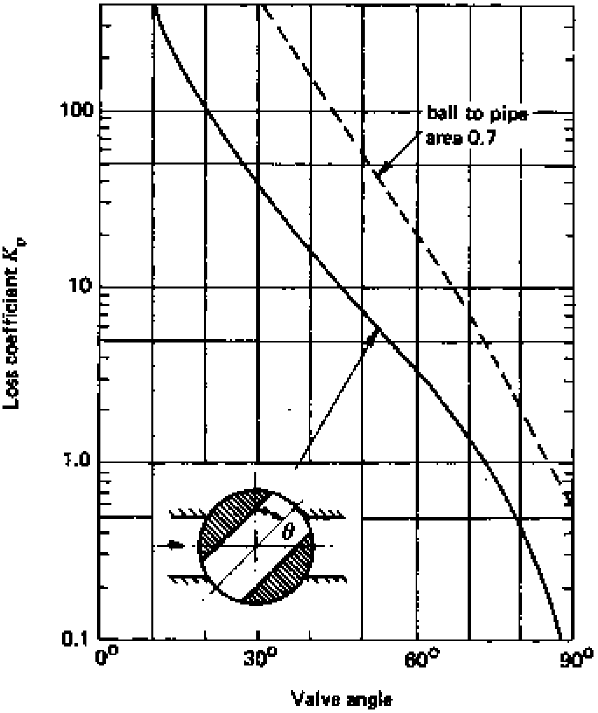

Fig. 26–25. Ball valve loss coefficients (*Source:* Reference 7; courtesy of the BHR Group Limited)

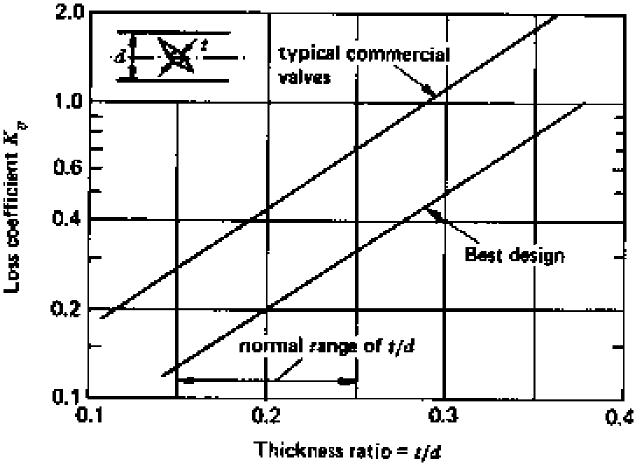

Fig. 26–26. Loss coefficients for fully open butterfly valves (*Source:* Reference 7; courtesy of the BHR Group Limited)

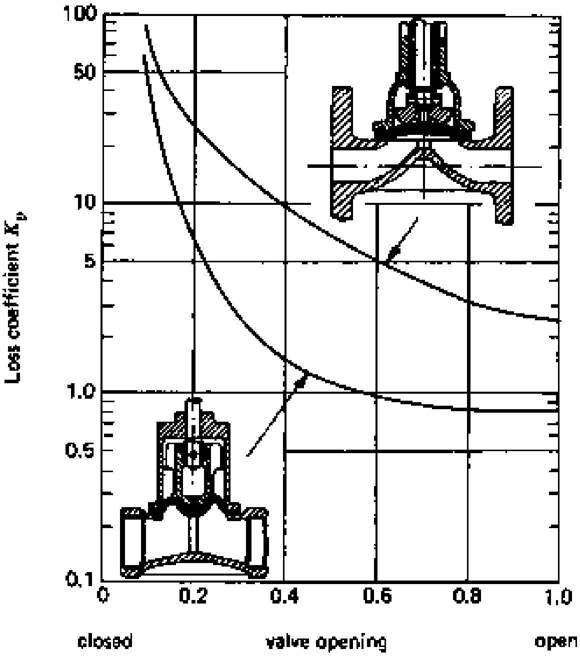

Fig. 26–27. Diagrams valve loss coefficients (*Source:* Reference 7; courtesy of the BHR Group Limited)

on net positive suction head that is related to Thoma's cavitation constant as follows:[17]

$$\sigma = \frac{\text{NPSH}_i}{H_T} \tag{26–23}$$

where:

NPSH_i = net positive suction head at point of cavitation inception, feet
H_T = total dynamic head, feet

By combining these two equations, a method for estimating the NPSH can be determined:

$$\text{NPSH}_i = \frac{H_T K \cdot N_s^{4/3}}{10^6} \tag{26–24}$$

The value obtained for NPSH_i is then compared to the NPSHA and NPSHR to determine if a problem may occur.

For centrifugal pumps, the following conditions should be avoided:[1]

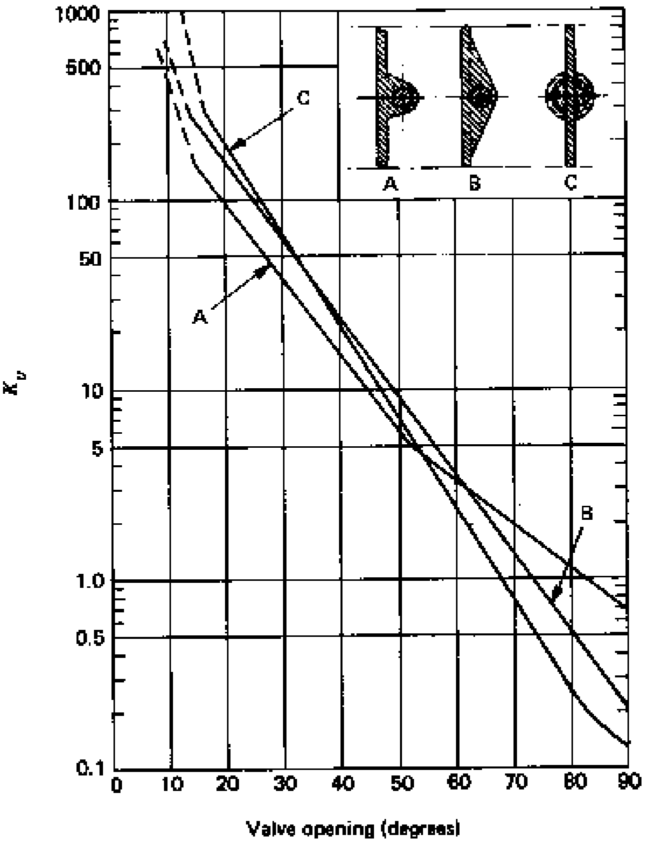

Fig. 26–28. Loss coefficients for butterfly valves (*Source:* Reference 7; courtesy of the BHR Group Limited)

- Heads much lower than head at peak efficiency of pump
- Capacity much higher than capacity at peak efficiency of pump
- Suction lift higher or positive head lower than that recommended by manufacturer
- Liquid temperatures higher than that used to design the system
- Speeds higher than manufacturer’s recommendations

The simplest method of avoiding cavitation in pumps is to make sure that the NPSHA is at least 2 to 5 feet larger than the NPSHR.

Open Channels

Open channel flow is used to convey water through many of the water plant processes. Open channel flow occurs in basins, conveyance channels, split boxes, flumes, launders or troughs, and over weirs. Under most circumstances, the flow can be assumed to

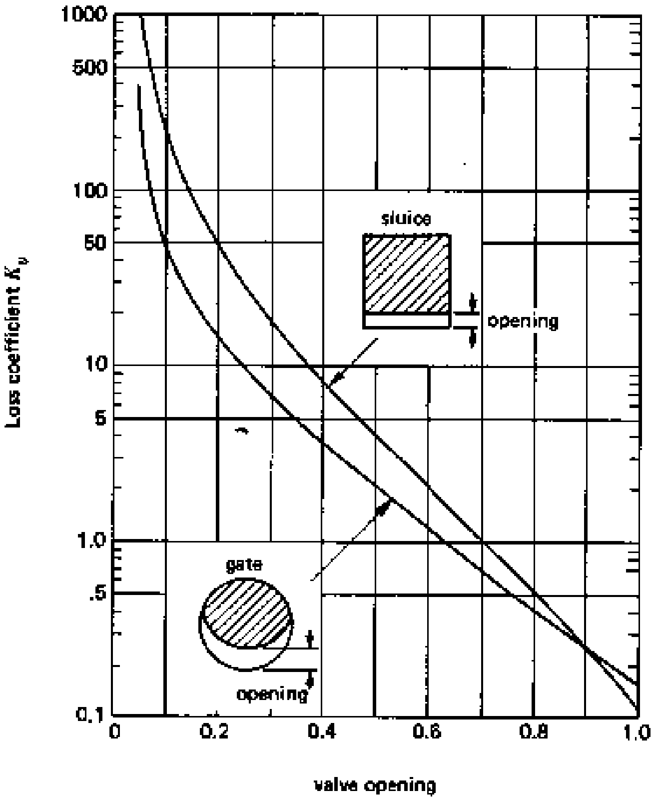

Fig. 26–29. Gate and sluice valve loss coefficients (seat area = pipe area) (*Source:* Reference 7; courtesy of the BHR Group Limited)

approach parallel flow with a uniform velocity distribution. The types of flow typically encountered approach steady varied flow, or spatially varied (nonuniform) flow. Figure 26–36 (page 931) illustrates the typical types of flow. Although these types rarely are accurately achieved, they provide reasonable approximations in most circumstances. There may be special circumstances in a plant process or design configuration resulting in other types of flow. In these cases, a more detailed investigation of the open channel hydraulics is warranted than is presented here. Several references listed in this chapter may be used.[1,18–21]

Conveyance Channels

Open flow conveyance channels may be pipes flowing partially full or concrete channels of various shapes. These are termed *artificial flow channels,* as opposed to natural flow channels. Natural channels may also be encountered, particularly in plant influent raw-water channels. Most applications for conveyance channels approach uniform flow conditions.

By far, the most widely used formula for computations involving open channel flow is the Manning formula, introduced in 1889 by Robert Manning and later modified to its present form, as presented in Equation 26–6.

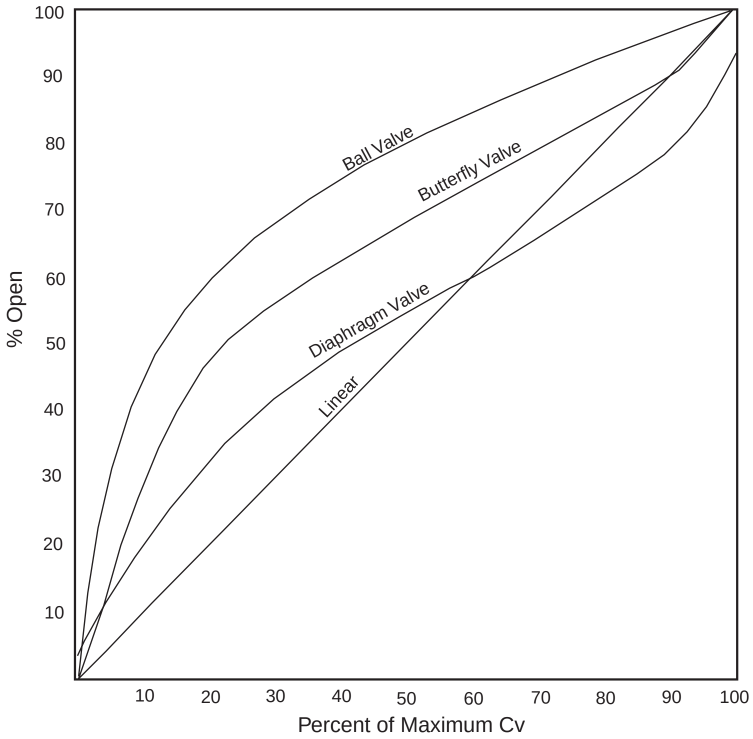

Fig. 26–30. Flow characteristic of valves in partially open position (Courtesy of ITT Industries)

Values of *n* may be obtained from Table 26–2. Manning's formula may be related to channel discharge by using the continuity equation:

$$Q = VA \qquad (26\text{–}25)$$

where:

Q = discharge, cu ft/sec (m^3/s)
A = cross-sectional area, sq ft (m^2)
V = mean velocity, ft/sec (m/s)

Thus:

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2} \qquad (26\text{–}26)$$

$$Q = \frac{1.0}{n} AR^{2/3}S^{1/2)} \qquad \text{(metric)} \qquad (26\text{–}27)$$

The calculation of headloss in open channels is basically the determination of the water surface elevations. These elevations are controlled by either upstream, down-

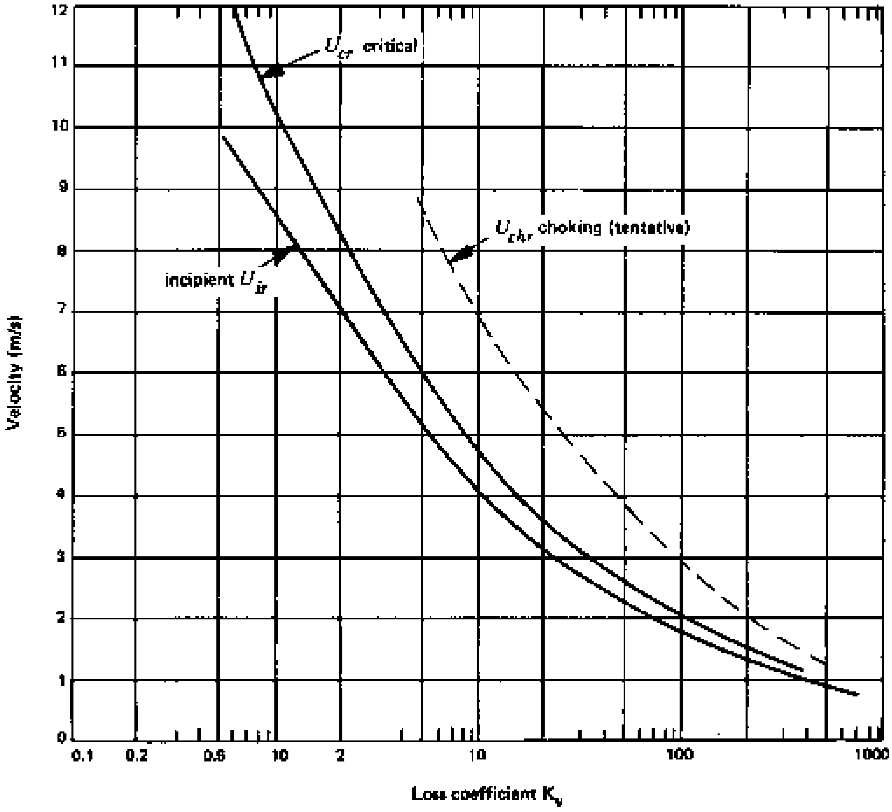

Fig. 26–31. Cavitation velocities for butterfly valves (*Source:* Reference 7; courtesy of the BHR Group Limited)

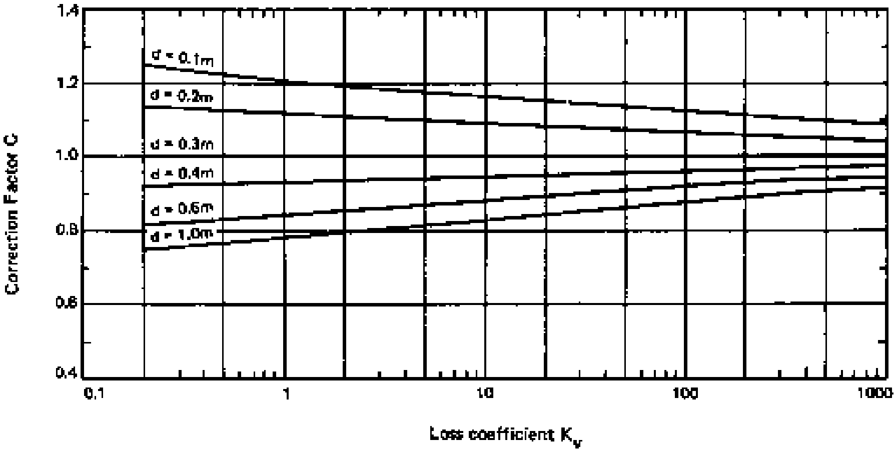

Fig. 26–32. Correction factors for valve size (*Source:* Reference 7; courtesy of the BHR Group Limited)

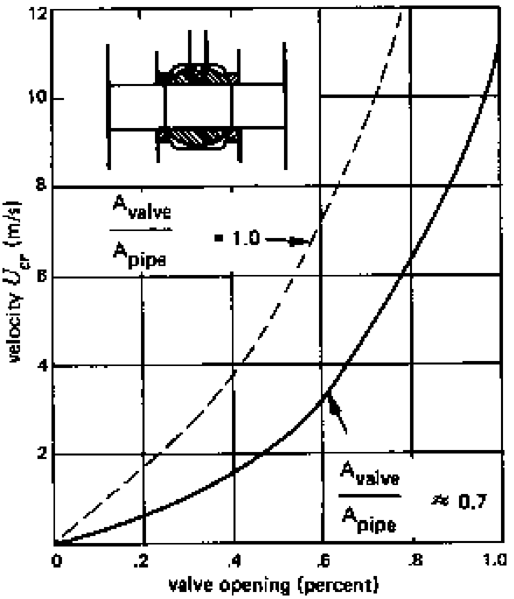

Fig. 26–33. Critical cavitation velocities for ball valves (*Source:* Reference 7; courtesy of the BHR Group Limited)

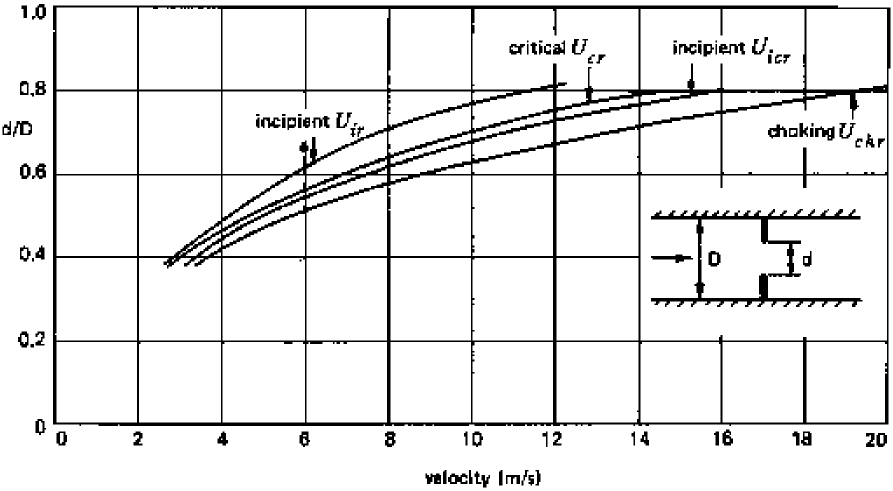

Fig. 26–34. Cavitation velocities for sharp-edged orifices (*Source:* Reference 7; courtesy of the BHR Group Limited)

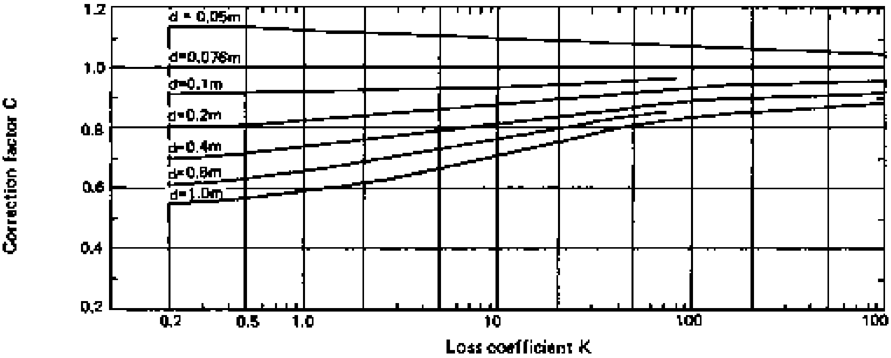

Fig. 26–35. Correction factors for orifice size (*Source:* Reference 7; courtesy of the BHR Group Limited)

stream, or artificial conditions. Mild or flat slopes result in subcritical flow, and surface profiles are generally controlled by downstream or artificial conditions. Steep slopes result in supercritical flow, and surface profiles are generally controlled by upstream or artificial conditions. Steep slopes and supercritical flows are not typically encountered in water treatment plant conveyance means, and should be avoided.

It may be necessary to calculate the water profile in a pipe flowing partially full. Since this is classified as open channel flow, the relation between the hydraulic elements as computed by Manning's equation must be known. Figure 26–37 illustrates the relationships between the hydraulic elements of a circular pipe.

King and Brater's *Handbook of Hydraulics*[5] contains tables (reprinted here as Table 26–7 and 26–8 on pages 933–934) that can be used to calculate the depth of flow in a pipe flowing partially full, as the following example shows.

Example: Determine the depth of flow in a 36-inch-diameter cast-iron pipe flowing partially full with a slope of 0.010, an n value of 0.013, and a flow of 20 cu ft/sec (0.566 m^3/s).

Step 1. Calculate K' in the equation:

$$Q = \left[\frac{K'}{n}\right] d^{8/3} S^{1/2} \tag{26–28}$$

TABLE 26–5. Range of Critical Cavitation Indices

Item	Value	Comments
Gate valve	1.0 to 1.8	Depending on opening
Globe valve	0.45 to 0.8	Depending on opening
Butterfly valve	0.9 to 2.2	Depending on opening
Butterfly valve, vented	0.6 to 0.8	Depending on opening
Cone valve	0.6 to 4.3	Depending on opening
Ball valve	1.8 to 4.5	Depending on opening
Needle valve	0.6 to 2.1	Depending on opening and valve design

Source: Reference 11. (Printed with permission of J. P. Tullis, Fort Collins, CO)

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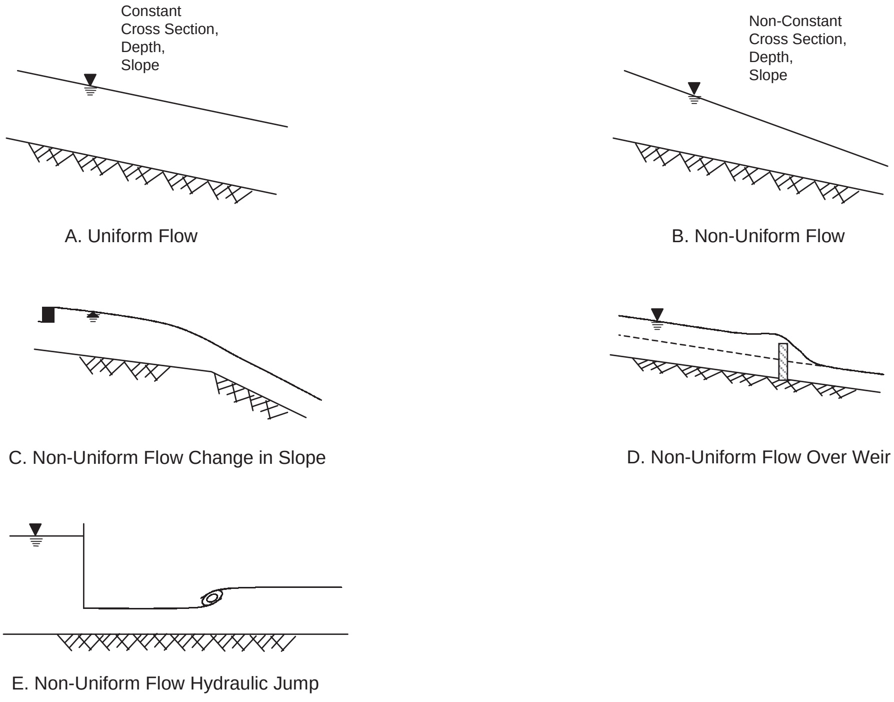
Rearranging and substituting:

$$K' = \frac{nQ}{d^{8/3}\,S^{1/2}} = \frac{0.013(2.0)}{(3)^{8/3}(0.01)^{1/2}} = 0.0139$$

Step 2. Determine the depth of flow:
Find D/d value for K′ = 0.0139:



A. Uniform Flow

B. Non-Uniform Flow

C. Non-Uniform Flow Change in Slope

D. Non-Uniform Flow Over Weir

E. Non-Uniform Flow Hydraulic Jump

Fig. 26–36. Various types of open-channel flow (*Source:* Reference 1)

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Fig. 26–37. Hydraulic elements for circular pipes (From King, H. W., and Brater, E. F., *Handbook of Hydraulics.* Copyright © 1976. Reproduced with permission of The McGraw-Hill Companies.)

$$D/d = 0.12$$

Therefore, the depth of flow is:

$$(D/d) \times d = 0.12 \times 36 = 4.32 \text{ inches or } 0.36 \text{ foot } (110 \text{ mm})$$

If desired, the velocity can then be obtained using Figure 26–37. The required pipe size can also be calculated to match a given flow line if desired.

Flow Distribution

Flow distribution channels are often used to distribute water to parallel processes, such as clarifiers or flocculation basins. The distribution can be achieved using submerged orifices or overflow weirs. The flow in the channels is termed spatially, nonuniform flow. Studies by Camp and Graber[22] evaluated flow distribution. By making some simplifying assumptions, they derived the following equation:

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$$\frac{h_o}{h_o + h_f} = \frac{1}{M^2} \tag{26–29}$$

where:

h_o = headloss through orifice
h_f = frictional loss between orifices
M = ratio of discharges of the two orifices

A typical design value is to keep the flows within 5 percent of each other. Then, substituting in Equation 26–29, $h_o = 10h_f$. The ratio of 10 × manifold headloss for the orifice headloss has been used by many engineers in designing distribution systems. However, more recent studies have shown that the discharge coefficient for each orifice changes with square of the downstream velocity in the manifold, and the above ‘‘rule of thumb’’ is not accurate.

Chao and Trussell[23] derived an equation to estimate the coefficient of discharge for rectangular side weirs as follows:

$$C_w = 0.611x \left(1 - \frac{3F_u^2}{F_u^2 + 2}\right)^{1/2} \qquad \text{for } F_u < 1.0 \tag{26–30}$$

where:

F_u = Froude number of channel flow upstream of the weir
$= V_u/gh_u$
V_u = Local average velocity, ft/sec
h_u = Depth of flow in channel, ft

Chao and Trussell[23] included a plot of the coefficient of discharge, as shown in Figure 26–38. This clearly shows that C_W changes dramatically with the Froude number, and must be considered if an accurate flow split is desired. Typically, one of three methods are used to achieve the flow split: These are:

- Construct a tapered channel.
- Construct a wide feed channel having virtually no headloss (this may result in solids deposition unless means are provided to prevent this).
- Provide modulating weirs.

FLOW MEASUREMENT—OPEN CHANNEL

Weirs

The headloss through a basin, whether it is a flow split basin, flocculation basin, or settling basin, is generally controlled by weirs at the outlet. Weirs are also used for flow splitting and flow measurement. The headloss over a weir depends on the type, length, and flow rate. There are a number of different types of weirs, as illustrated in Figure 26–39 (page 937). The headloss over a weir for given flows and weir shapes

Image Not Available

Fig. 26–38. Variation of C with F (*Source:* Reference 15; from Messina, J., *Pump Handbook,* 3d ed. Copyright © 2001. Reproduced with permission of The McGraw-Hill Companies.)

is widely published in hydraulic and water resource handbooks. The formula for the weir types illustrated in Figure 26–39 are as follows:[25]

- Rectangular—contracted weir:

$$Q = 3.33\left(L - \frac{nH}{10}\right)H^{3/2} \tag{26–31}$$

$$Q = 1.84\left(L - \frac{nH}{10}\right)H^{3/2} \qquad \text{(metric)} \tag{26–32}$$

- Rectangular—suppressed weir:

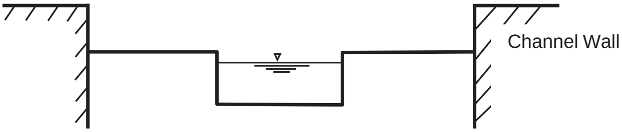
$$Q = 3.33\,LH^{3/2} \tag{26–33}$$

$$Q = 1.84\,LH^{3/2} \qquad \text{(metric)} \tag{26–34}$$

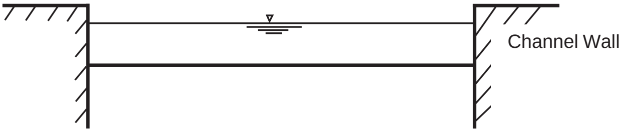
- Cipolletti weir:

$$Q = 3.367\,LH^{3/2} \tag{26–35}$$

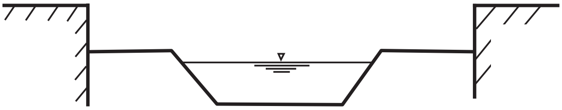
$$Q = 1.858\,LH^{3/2} \qquad \text{(metric)} \tag{26–36}$$



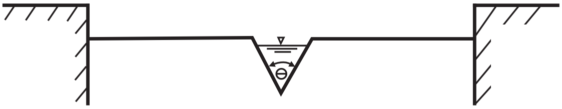
A. Rectangular - contracted



B. Rectangular - suppressed



C. Cipolletti



D. V - notch - $\theta = 90^\circ, 60^\circ, 45^\circ, 22\frac{1}{2}^\circ$

Fig. 26–39. Weir types

- V-notch weir:

$$Q = XH^{2.49} \tag{26–37}$$

where:

Q = discharge, cu ft/sec (m^3/s)
L = length of weir, ft (m)
H = head on weir crest, ft (m)
n = 1 or 2 for contractions at one or both ends of rectangular, contracted weir
X = 2.49 (1.34) for a 90° v-notch

$X = 1.443\ (0.778)$ for a = 60° v-notch
$X = 1.035\ (0.558)$ for a = 45° v-notch
$X = 0.497\ (0.268)$ for a = 22½° v-notch

The values in parentheses are for metric equivalents.

Broad-Crested Weir

Another weir that is sometimes used is a broad-crested weir. It is a length of elevated channel floor, as shown in Figure 26–40.

The total discharge over the weir is:

$$Q = 0.433\sqrt{2g}\left(\frac{d_1}{d_1 + h}\right)^{1/2} LH^{3/2} \tag{26–38}$$

where:

Q = flow rate, cfs
L = length of weir, ft
H = height of approach water above crest, ft
h = height of weir above channel, ft

As h goes from 0 to infinity, the equation simplifies to:

$$h = 0 \qquad Q = 3.47LH^{3/2} \tag{26–39}$$

$$h = \infty \qquad Q = 2.46LH \tag{26–40}$$

Based on actual measurements, the range of coefficient values is 3.05 to 2.67.[1]

Flumes

Flumes have been used to measure flows and to divide flows between downstream basins or processes. The flume most frequently used to measure flows is the Parshall

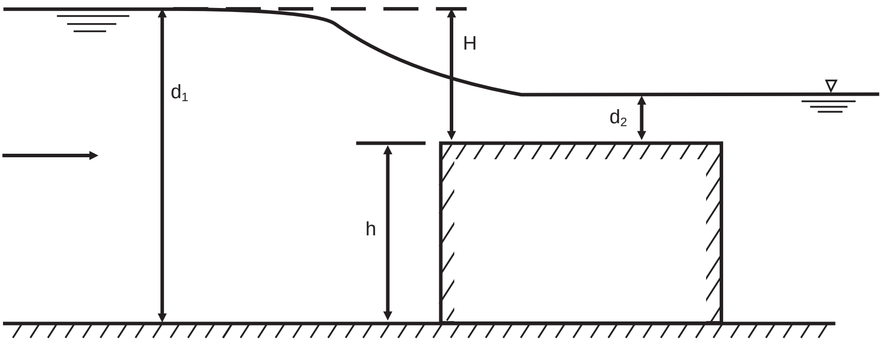


Fig. 26–40. Broad-crested weir

flume. Another flume that is sometimes used is the Cutthroat flume. These are briefly described in the following sections.

Parshall Flume The Parshall flume is a Venturi-type flume in which water passes through critical flow in the throat and has a hydraulic jump at the exit of the flume. The Parshall flume was developed in 1920. The flume was developed experimentally and requires that the water be evenly distributed across the channel and be without turbulence. Typically, an approach channel length of 10 × throat width is sufficient to provide the appropriate flow conditions.

The flow equation for free flow through a Parshall flume is:[25,26]

$$Q = kH^n \tag{26–41}$$

where:

Q = flow rate, mgd
H = depth of water, ft
k = constant (depends on throat width)
n = constant (depends on throat width)

The values of constants k and n for several flume widths are listed in Table 26–9.

Cutthroat Flume The cutthroat flume is a relatively simple flume with a flat bottom. However, it also needs to have an approach flow that is evenly distributed across the channel.

The flume is shown in Figure 26–41, and the flow rate equation is:

TABLE 26–9. Values for Parshall Flume Constants

Throat Width, in/ft	Flow in mgd		Throat Width, mm	Flow in m^3/hr	
	k	n		k	n
1 in	0.2184	1.550	25.4	217.3	1.550
2	0.4369	1.550	50.8	434.6	1.550
3	0.6411	1.547	76.2	635.5	1.547
6	1.331	1.580	152	137.2	1.580
9	1.984	1.530	229	1927	1.530
1 ft	2.585	1.522	305	2487	1.522
1½ ft	3.878	1.538	457	3803	1.538
2	5.170	1.550	610	5143	1.550
3	7.756	1.555	914	7863	1.568
4	10.34	1.578	1220	10630	1.578
5	12.93	1.587	1520	13440	1.587
6	15.51	1.595	1830	16280	1.595
8	20.68	1.607	2440	22010	1.607
10–50 ft	2.384 W + 1.616	1.60	3–15 M	8255 W + 1706	1.60

(Courtesy of Isco, Inc.)

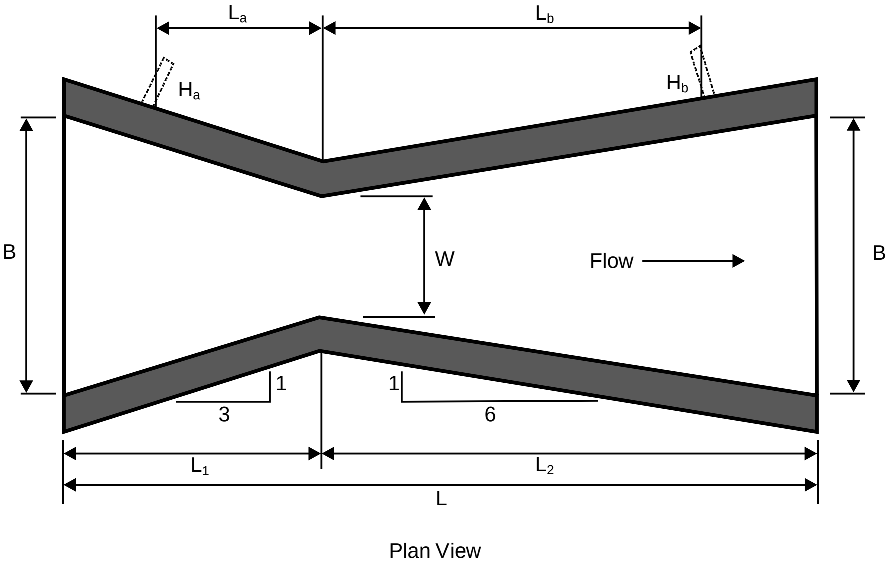


Fig. 26–41. Dimensional configuration of rectangular cutthroat flume (*Source:* Reference 25; illustration courtesy of Isco, Inc.)

$$Q = KW^{1.025}H^{n} \tag{26–42}$$

$$= CH^{n} \tag{26–43}$$

where:

Q = flow rate, mgd
k = free-flow coefficient
C = free-flow coefficient
W = throat width, W
H = water depth, ft
n = free-flow exponent

The coefficients are shown in Table 26–10.

FLOW MEASUREMENT—CLOSED CONDUITS

Conduit Flow Elements

There are several types of devices used to measure flow in pipes. These include differential pressure units, positive volume displacement units, direct discharge measurement units, and velocity-area measurement units. There are several items to consider before selecting the device to use, as follows:

TABLE 26–10. Coefficients and Exponents for Free Flow Through Cutthroat Flumes with Head in Feet

			cfs		gpm		mgd	
Length (ft)	Width (ft)	n_1	K	C	K	C	K	C
9.00	1.000	1.560	3.500	3.500	1571	1571	2.262	2.262
9.00	2,000	1.560	3,500	7.122	1571	3197	2.262	4.603
9.00	4,000	1,560	3,500	14.49	1571	6505	2,262	9.367
9.00	6.000	1.560	3.500	21.96	1571	9857	2.262	14.19
4.50	0.250	1.720	3.980	0.9611	1786	431.3	2.572	0.6212
4.50	0.500	1.720	3.980	1.956	1786	877.8	2.572	1.264
4.50	1.000	1.720	3.980	3.980	1786	1786	2.572	2.572
4.50	2.000	1.720	3.980	8.099	1786	3635	2.572	5.234
3.00	0.167	1.840	4.500	0.7171	2020	321.9	2.908	0.4635
3.00	0.333	1.840	4.500	1.459	2020	655.0	2.908	0.9432
3.00	0.667	1.840	4.500	2.970	2020	1333	2.908	1.919
3.00	1.333	1.840	4.500	6.043	2020	2712	2.908	3.906
1.50	0.083	2.000	6.400	0.5012	2872	2224.9	4.136	0.3239
1.50	0.167	2.000	6.400	1.020	2872	457.7	4.136	0.6592
1.50	0.333	2.000	6.400	2.076	2872	1896	4.136	2.730
1.50	0.667	2.000	6.400	4.224	2872	1896	4.136	2.730

(Courtesy of Isco, Inc.)

- What is the fluid being measured?
- What is the flow range (minimum, average and maximum)?
- Is the fluid likely to be contaminated?
- What accuracy of flow measurement is required?
- Is the fluid corrosive?
- What is the available headloss that can be tolerated?

After these questions have been answered, one of the following meters can be selected. For these meters, typically they should have a length of about 10 pipe diameters upstream of the meter and three to five pipe diameters downstream of the flow element.

Differential Pressure Meters

Differential pressure meters include the Venturi-type meters such as the Dall tube, Venturi insert, and conventional Venturi. They can handle a flow range of 10:1 and have an accuracy of ±1 percent. For best results, the flow range should be less than 10:1. These flow tubes should be installed such that they are always full of the fluid, and do not permit a negative pressure to occur at the throat at maximum flow. These units are typically used for filter rate control and raw-water and finished-water metering.

Orifice Meters

Another differential device is the orifice plate, described earlier in this chapter. Their flow accuracy is rated at ±2 percent with a flow range of only 4:1. However, these units create a high headloss, ranging from 40 percent of the differential pressure to as much as 90 for a small throat diameter to pipe diameter. These are sometimes used for air flow measurement and occasionally for water. But due to their high headloss, they are not favored. Other meters that are frequently used in water treatment plants are described below.

Propeller and Turbine Meters

These units operate through the fluid causing a propeller or turbine to rotate. As they rotate, a mechanism measures the rotation and sends out electric pulses. These flow devices have an accuracy of about ±2 percent and a flow range of 10:1. For these units to operate efficiently, the minimum velocity should be about 1 ft/sec. These meters are typically used on clean, uncontaminated water for billing purposes for customer supplies.

Magnetic Flow Meters

These devices operate by the water changing a magnetic field as it passes through the unit. The change in magnetic fluid creates a voltage that is proportional to the velocity through the meter, and through continuity, the flow rate is computed ($Q = V \cdot A$). These meters have an accuracy of less than 1 percent for higher velocities (3 to 20 ft/sec). At velocities less than 3 ft/sec, the accuracy deteriorates. These meters have traditionally been used to measure solids bearing fluids such as residual waste flows, but they are now also being used more widely in the potable water industry.

Ultrasonic Flow Meters

These meters operate by measuring the time it takes for an electronic pulse to cross a pipe, against the flow of liquid. The two styles are the Doppler type and the transit type. The transit type can operate on clean water, whereas the Doppler type usually is used on solids bearing fluids. Transit meters are much more expensive, but do provide improved accuracy. The Doppler meter has an accuracy of about 2 percent and the transit type unit is ±1 percent.

Rotameters (Variable Area Flow Meters)

Rotameters have a tapered tube containing a float. These units are mounted vertically, with the wide end at the top. Water flows up through the tube, causing the float to move upward. As the rate of flow increases, the float rises farther up the tube. The height of the float is proportional to flow. It can be used for either gas or liquid flow. The flow range is about 10:1, and it has an accuracy of ±2 percent. This unit is typically used for low flow rates, such as chemical solutions, or seal water flow. A typical range of flow is up to 150 gph for liquids and up to 500 scfm for gases. These meters are not affected by the piping arrangement upstream of the meter.

Baffles

Some basins contain a number of baffles that are used for mixing. When the flow velocity is not negligible, the headloss through the basin must be considered. Experimental tests have determined that the K_L value to be used in Equation 26–8 is approximately 3.3. This was determined for 180-degree bends in flow.

Occasionally, the baffles are of the over-and-under-flow type. In these cases, the weir can be used for flow over a baffle and the orifice equation for the flow under the baffle.

Launder and Troughs

The flow in the clarifier launder and backwash trough is considered spatially varied flow or nonuniform flow. To determine the maximum water level in a launder, the desired solution is the upstream head in the launder at the farthest point from the discharge point, as shown in Figure 26–42. The general formula is:

$$H_L = \sqrt{(2h_c^3/h_o) + \left(h_o - \frac{1}{3}iL\right)^2} - \frac{2}{3}iL \qquad (26\text{–}44)$$

where:

H_L = headloss, ft (m)
h_c = critical depth, ft (m) = $(Q^2/gb^2)^{1/3}$ for rectangular channels
L = length of channel, ft (m)

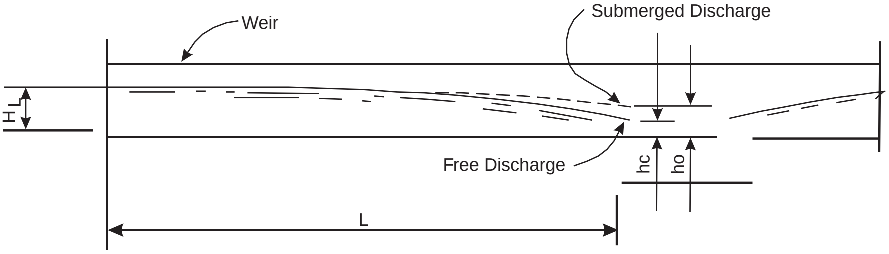


Fig. 26–42. Flow in launder

h_o = depth at discharge location, ft (m)
i = launder slope, ft/ft (m/m)
g = acceleration of gravity, 32.174 ft/sec^2 (9.81 m/s^2)
b = channel width, ft (m)
Q = discharge, cu ft/sec (m^3/s)

When the channel is level, as most clarifier launders are, $i = 0$, the formula may be reduced to:

$$H_L = \sqrt{(2h_c^3/h_o) + h_o^2} \tag{26–45}$$

If the discharge is free, that is, if $h_o = h_c$, then the equation may be further reduced:

$$\begin{aligned} H_L &= \sqrt{3h_c^2} \\ &= h_c\sqrt{3} \\ &= 1.732h_c \end{aligned} \tag{26–46}$$

Effluent launders should be sized to provide at least 4 inches (0.10 m) freeboard from the bottom of the V-notches at the upstream end at the peak flow conditions.

Filter washwater troughs act identically to clarifier launders, and the same criteria apply. A rectangular washwater trough with free discharge can be sized by the following equation, by substituting the value for h_c into equation 26–46:

$$H_o = \sqrt{3}\left[\frac{Q^2}{gb^2}\right]^{1/3} \tag{26–47}$$

where:

b = width of launder, ft (m).

FILTERS

The determination of headloss through filters is made for both filter operation and filter backwashing. The operational headloss consists of two components: headloss through a clean bed and the headloss allowed between filter runs before backwashing. The headloss allowed between backwashes can be chosen on the basis of several factors, including cost of available headloss, filter media design, and filter breakthrough. Typical design headloss valves range from 5 to 10 feet (1.524 to 3.048 m).

The headloss through a clean filter bed can be calculated through the use of a number of equations, including the Carmen-Koseny, Fair-Hatch, and Rose equations, which are given below for reference. However, unless a unique filter system is designed and headloss is critical, the benefit of these equations is questionable. Typical clean filter beds and underdrain systems, whether mixed- or dual-media, generally impart 1 to 1½ feet (0.31 to 0.46 m) of headloss. Specific media manufacturers or suppliers should be consulted when headloss values are determined.

The Carmen-Kozeny equation, for use with uniform-size media, is:

$$\frac{H_L}{l} = f\left(\frac{1}{\phi d}\right)\left(\frac{1-\varepsilon}{\varepsilon^3}\right)\left(\frac{V^2}{g}\right) \tag{26–48}$$

where:

H_L = headloss, ft (m)
l = depth of media, ft (m)
ϕ = particle shape factor, dimensionless
= 1.0 for sphere
= 0.28 for mica flakes
= 0.82 for rounded sand
= 0.73 for angular sand
d = particle diameter, ft (m)
ε = bed porosity, dimensionless
V = filtration rate of water, ft/sec (m/s)
g = acceleration of gravity, 32.174 ft/sec^2)
f = friction factor, dimensionless

$$f = 150\,\frac{1-\varepsilon}{\text{Re}} + 1.75$$

Re = Reynolds number, dimensionless $\frac{\phi w \rho}{\mu}$
$\overline{V}s$ = average superficial velocity through an empty bed, ft/sec (m/s)
ρ = density, slugs/cu ft (kg/m^3)
μ = dynamic viscosity, lb · sec/sq ft (N · s/m^2)

The Fair-Hatch equation for nonuniform media is:

$$\frac{H_L}{l} = \frac{K}{g}\,\frac{\mu V(1-\varepsilon)^2}{\varepsilon^3}\left(\frac{6}{\phi}\right)^2 \sum_{i=1}^{n} \frac{P_i}{d_{pi}^2} \tag{26–49}$$

where:

H_L = headloss, ft (m)
l = depth of media, ft (m)
K = Kozeny constant, ≈ 5

μ = dynamic viscosity, lb/sec/ft^2 (N · s/m^2)
ρ = density, slugs/cu ft (kg/m^3)
g = acceleration of gravity, 32.174 ft/sec^2 (9.81 m/s)
V = filtration rate of water, ft/sec (m/s)
ε = bed porosity, dimensionless
ϕ = shape factor, dimensionless
P_i = sand fraction within adjacent sieve sizes, by weight
d_p = geometric mean size of adjacent sieve sizes, ft (m) = $[(d_1)(d_2)]^{1/2}$

The Rose equation for uniform size media is:

$$\frac{H_L}{l} = f\left(\frac{1}{\phi d}\right)\left(\frac{1}{\varepsilon^4}\right)\left(\frac{V^2}{g}\right) \tag{26–50}$$

where:

H_L = headloss, ft (m)
l = depth of media, ft (m)
ϕ = shape factor, dimensionless
d = particle diameter, ft (m)
ε = porosity, dimensionless
g = acceleration of gravity, 31.174 ft/sec^2 (9.81 m/s)
V = filtration rate of water, ft/sec (m/s)
P_i = sand fraction within adjacent sieve sizes, by weight
d_p = geometric mean size, ft (m) = $[(d_i)(d_2)]^{1/2}$
f = friction factor, dimensionless
= 1.067 C_D

$$C_D = \frac{24}{R^e} + \frac{3}{\sqrt{\text{Re}}} + 0.34 \quad \text{for transitional flow}$$

$$C_D = \frac{24}{\text{Re}} \quad \text{for laminar flow}$$

$$\text{Re} = \text{Reynolds number} = \frac{\phi d V \rho}{\mu}$$

The headloss through a filter bed during backwash can be calculated as simply the sum of headlosses of components of the backwash system:

$$H_L = h_b + h_g + h_u + h_f \tag{26–51}$$

where:

H_L = total headloss, ft (m)
h_b = headloss through the expanded bed, ft (m)
h_g = headloss through the gravel support bed, ft (m)
h_u = headloss through the underdrain system, ft (m)
h_f = headloss through the remaining backwash system, ft (m)

The headloss through the underdrain and backwash system can be determined from previous discussions in this chapter. As in the clean filter bed, the headloss through the expanded bed and gravel support system is more difficult to calculate. Typical values are 2 to 5 feet (0.61 to 15.24 m), but the bed and gravel support headloss may be calculated by the following formula:

Expanded bed headloss:

$$h_b = l_b(1 - \varepsilon)(S_s - 1) \tag{26–52}$$

where:

h_b = headloss through the expanded bed, ft (m)
l_b = depth of bed (at rest), ft (m)
ε = porosity of bed, dimensionless
S_s = specific gravity of the filter media, dimensionless

Gravel support bed headloss:

$$h_g = 200 l_g \left(\frac{V\mu}{\rho g \phi^2 d_{10}^2}\right)\left(\frac{(1 - \varepsilon_g)^2}{\varepsilon_g^3}\right) \tag{26–53}$$

where:

h_g = headloss through the gravel support bed, ft (m)
l_g = depth of bed, ft (m)
V = backwash rate, ft/sec (m/s)
μ = dynamic viscosity, slugs/ft-sec (n s/m^2)
p = density, slugs/cu ft (kg/m^3)
g = acceleration of gravity, 32.174 ft/sec^2 (9.81 m/s)
ϕ = shape factor of gravel, dimensionless
d_{10} = size of gravel of which 10 percent by weight is less than, ft (m)
ε_g = porosity of gravel bed, dimensionless

Filter manufacturers should be consulted to assist in headloss determinations.

PRESSURE SURGE

A pressure surge (also called water hammer) occurs when the fluid flow momentum is changed. The magnitude of the forces resulting from pressure surges can be extremely large and may result in pipeline failures in extreme cases. A detailed discussion of this subject is presented in Parmakian[27] or Karassic.[15]

A common type of pressure surge happens when pumping equipment is stopped. The sudden cessation of flow results in the water attempting to continue to move without the driving force of the pump. A negative pressure results that moves along the pipeline at the speed of sound in the fluid (4,700 ft/sec for water), causing collapsing pressures in the pipe. When this pressure reaches the discharge reservoir, the entire length of pipe is subject to collapsing pressures, and the fluid velocity is zero.

Because the pressure in the pipe is less than that in the reservoir, water flows into the line, setting up a positive wave front traveling back toward the pump. Typically, the pump discharge is provided with a check valve to prevent backflow through the pump. The positive wave reaches the valve, and the kinetic energy is converted to static pressure, which is the pressure surge.

In order for the full magnitude of the water hammer pressure wave to propagate, the check valve must be closed instantly. Because closing a valve instantly is virtually impossible, the pressure of the wave created may be less than the full wave pressure that would occur for instantaneous valve closure. However, if the valve is completely closed before the first positive pressure wave returns to it (the travel time of the wave exceeds the closure time of the valve), the pressure in the pipe will continuously increase, and the pressure wave created will be the same as for instantaneous valve closure. Therefore, when the valve closure time is equal to or less than $2L/a$ [where L is pipe length, ft (m), and a is pressure wave velocity, ft/sec (m/s)], the wave pressure will be the same as for instantaneous valve closure. When the valve closure time is longer than $2L/a$, the pressure wave will be less than for instantaneous valve closure. Having the valve closure time longer than $2L/a$ will allow fluid flow through the pump. However, restarting a pump that is rotating in the reverse direction can result in coupling or shaft failure.

The velocity of the water hammer wave and the increase in pressure due to water hammer may be obtained from the following equations, where the valve closure or stoppage of flow due to stopping the pump occurs in less than $2L/a$:

$$a = \frac{4{,}660}{\sqrt{1 + K_e D/Et}} \tag{26–54}$$

$$h = \frac{aV_0}{g} \tag{26–55}$$

where:

a = water hammer wave velocity (acoustic velocity), ft/sec
K_e = bulk modulus of elasticity of water, 300,000 psi
D = pipe diameter, in.
E = modulus of elasticity of pipe wall material, psi
t = thickness of pipe wall, in.
h = head due to water hammer (in excess of static head), ft
g = acceleration of gravity, 32.2 ft/sec^2
V_0 = velocity in pipe at initial steady-state conditions, ft/sec

The modulus-of-elasticity values for various types of pipes are

- Cast-iron pipe, about 1.35×10^7 psi
- Most steel pipe conduits, about 3.0×10^7 psi
- Concrete pipe or transite, about 3×10^6 psi
- Polyvinyl chloride (PVC), about 4.2×10^5 psi at 73°F
- Ductile iron, about 2.4×10^7 psi

Pipes of composite materials, such as concrete-lined steel cylinder pipe, will have wave velocities more nearly those of the inner material.

From the previous description of wave propagation for instantaneous valve closure, it is apparent that if the valve were not completely closed by the end of the cycle $2L/a$, the intensity of the pressure wave reflected from the valve would be suppressed. The maximum intensity of the pressure wave for a valve closure time of longer than $2L/a$ may be obtained by nomograph charts.[19]

Typical solutions to reducing pressure surge magnitude include:

- *Surge tank.* The top of the surge tank must be above the hydraulic grade line. The surge tank accumulates water in response to the pressure surge and supplies water to moderate the negative pressure.
- *Air chamber.* The lower portion of the air chamber contains water, and the upper portion contains compressed air. Like the surge tank, the air chamber provides water and a place to accumulate water during a surge.
- *Slow-closing check valve.* By a slow closing of the pump check valve over a period of more than one cycle (i.e., more than $2L/a$), the water hammer pressure is moderated.
- *Surge suppressor*. A surge suppressor consists of a pilot-operated valve that opens quickly upon pump outage through the use of a solenoid valve. The valve releases water from the pipeline and is closed later at a slower rate by using the pipeline pressure acting on a diaphragm valve.
- *Pump control valve*. Typically a ball valve with an air/oil or all-oil hydraulic operator is used for the pump check valve. The opening and closing speeds of the valve are adjustable, and limit switches on the valve are normally wired into the control circuit of the pump. The valve acts as a slow-closing check valve in power failure modes, and in normal operation the valve is 90–95 percent closed when the pump is stopped.

TRACER STUDIES

As part of the Safe Drinking Water Act, there is a requirement to determine the CT times for disinfection purposes. In order to determine this value accurately, a tracer study is performed to determine the contact time through the basins of interest. The contact time used, defined as T_{90}, represents the time in which 90 percent of the water passing through the system being tested is retained.

A single flow rate may not characterize the flow through the entire system. Therefore, tracer studies should be conducted at different flow rates. Ideally, tracer tests should be performed for at least four flow rates that span the entire range of flows. The flow rates should be separated by approximately equal intervals to span the range of operation—with one near average flow, two greater than average, and one less than average flow. The number of studies can be reduced based on site-specific restrictions and resources available. Systems with limited resources can conduct a minimum of one tracer test at the peak flow rate.

The water level in the basin being tested should be at or slightly below, but not above, the normal minimum operating level. For ozone contactors, tracer studies should be conducted for the range of mixing used in the process. Air or oxygen should be added in lieu of ozone to prevent degradation of the tracer. The flow rate of air or oxygen used for the contactor should be applied during the study to simulate actual operation. Tracer studies should then be conducted at several air/oxygen-to-water ratios to provide data for the complete range of ratios used at the plant.

Tracer Selection

Chloride and fluoride are the most common tracers used in drinking water plants (nontoxic and approved for potable water use). Rhodamine WT can be used, but it has several restrictions. However, Rhodamine B is not recommended by the EPA. Drawbacks of chloride and fluoride are that salts can induce density currents. Fluoride has the following limitations: It is difficult to detect at low levels, and many states impose a finished water limitation of 1 mg/L. The federal secondary and primary MCLS are 2 and 4 mg/L, respectively. EPA recommends that fluoride be used only in cases where feed equipment is already in place, for safety reasons.

Tracer Study Methods

It is recommended that at least 90 percent mass recovery for both methods be accomplished. One of the following two methods are generally used to perform tracer studies:[28]

- *Step-Dose Method*. Introduce tracer at constant dose until the concentration at the endpoint is steady-state. Frequently employed for drinking water applications. The results are directly used to determine the T_{90} value. Feed equipment is available

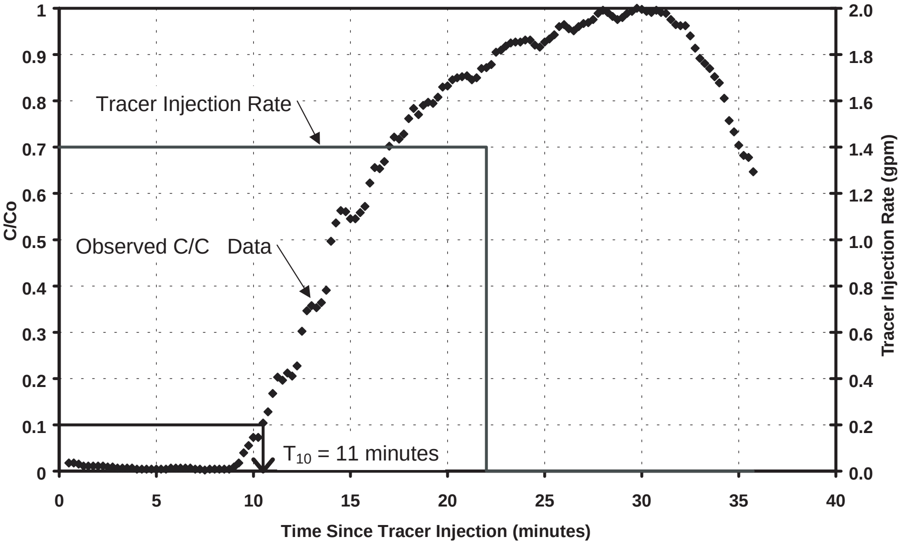


Fig. 26–43. Step-feed tracer curve

to provide a constant rate of application of the tracer, and data can be verified with concentration-time profile obtained from shutting off tracer.

- *Slug-Dose Method.* Introduce an instantaneous dose of tracer and take samples at the exit. Many samples are required to define concentration versus time profile. The slug-dose method is more complex than the step-dose method. The concentration and volume of instantaneous tracer dose must be carefully monitored to provide an adequate tracer profile at the effluent of the basin; concentration versus tracer profile cannot be used to directly determine T_{90} without further manipulation; and a mass balance is required to determine whether the tracer was completely recovered. This method is typically run for 2 times the theoretical detention time to capture all the tracer.

EPA recommends chloride doses on the order of 20 mg/L where background chloride levels are less than 10 mg/L in step-dose studies. A chloride dose of 10 to 20 mg/L is recommended in slug-dose tests. Fluoride concentrations as low as 1 to 1.5 mg/L are practical for step-dose tests when raw water fluoride levels are not significant. A fluoride dose of 1 to 2 mg/L is recommended for slug-dose tests. Typically, chloride is the tracer of choice.

Other Considerations

The tracer background concentration must be determined. The water must be sampled at the same locations as the tracer study.

Data Evaluation

The method of choice for evaluating the data is frequently a graphical plot of the value C/C_0 versus time. The background concentration of the tracer is determined and subtracted from the measured value of the tracer at the end of the basin. The ratio C/C_0 is then plotted against time as shown in Figure 26–43. The T_{10} value is read directly from the curve.

The slug-dose evaluation is similar, but the data is first converted to a form similar to the proceeding step-dose method. This method is described in detail in reference 28.

REFERENCES

1. Chow, Ven Te, *Open Channel Hydraulics*, McGraw-Hill, New York, 1959.
2. Westaway, C. R., and Loomis, A. W., eds., *Cameron Hydraulic Data*, Ingersoll Rand, Woodcliff Lake, NJ, 1981.
3. Moody, L. F., ‘‘Friction Factor for Pipe Flow,’’ *Trans. ASME,* Vol. 66, 1944.
4. Williams, G. S., and Hazen, A., *Hydraulic Tables,* 3rd ed., John Wiley, New York, 1920.
5. King, H. W., and Brater, E. F., *Handbook of Hydraulics*, 6th ed., McGraw-Hill, New York, 1976.
6. Hydraulic Institute, 9 Sylvan Way, Parsippany, NJ 07054 1973 267-9700, www.pumps.org.
7. Miller, D. S., ‘‘Internal Flow Systems,’’ *British Hydrodynamics Research Association,* 2nd ed., Cranfield, England, 1996.

8. Blaisdell, F. W., and Mansen, P. W., ‘‘Energy Loss of Pipe Junction,’’ *Journal of the Irrigation and Drainage Division*, ASCE, pp. 59–78, September 1967.
9. Hudson, H. E., Jr., *Water Clarification Process: Practical Design and Evaluation*, Van Nostrand Reinhold, New York, 1981.
10. McMann, J. S., ‘‘Mechanics of Manifold Flow,’’ *Trans. ASCE*, pp. 119–1103, 1954.
11. Ball, J. W., ‘‘Cavitation Design Criteria,’’ Control of Flow in Closed Conduits, *Proceedings of the Institute* held at Colorado State University, J. P. Tullis (ed.), August 1970.
12. Tullis, J. P., *Cavitation Guide for Control Valves.* U.S. Nuclear Regulatory Commission, NUREG/CR-6031, Washington, DC, April 1993.
13. Tullis, J. P., *Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients.* John Wiley & Sons, New York, 1989.
14. Skousen, P. L., *Valve Handbook*, McGraw-Hill, New York, 1998.
15. Karrassic, I. J., Krutzch, W. C., Fraser, W. H., and Messina, J. P., (eds.), *Pump Handbook*, 2nd ed., McGraw-Hill, New York, 1985.
16. Sanks, R. L., *Pumping Station Design*, 2nd ed., Butterworth-Heineman, Boston.
17. Rutschi, K., ‘‘Die fleiderer-saugzahl als gutegrad der saugfahigkeit von kreiselpumpen,’’ Schweizerische Bauzeitung, No. 12, Zurich, 1960.
18. ASCE Task Force, ‘‘Friction Factors in Open Channels,’’ *Journal of the Hydraulics Division*, ASCE, pp. 97–143, March 1963.
19. Daugherty, R. L., and Franzini, J. B., *Fluid Mechanics with Engineering Application*, 6th ed., McGraw-Hill, New York, 1965.
20. Henderson, F. M., *Open Channel Flow*, Macmillan, New York, 1966, p. 522.
21. Soliman, M. M., and Tinney, E. R., ‘‘Flow Around 180° Bends in Open Rectangular Channels,’’ *Journal of Hydraulics Division, ASCE*, pp. 893–908, July 1968.
22. Camp, T. R., and Graber, S. D., ‘‘Dispersion Conduits,’’ *J. San. Eng. Div.*, ASCE, Vol. 94, pp. 31–39, February 1968.
23. Chao, J. L., and Trussel, R. R., ‘‘Hydraulic Design for Flow Distribution Channels,’’ *ASCE, J. Env. Engineering*, Vol. 106, No. 2, p. 321, April 1980.
24. Skousen, Phillip L., *Valve Handbook*, McGraw-Hill, New York, 1997.
25. Grant, D. M., and Dawson, B. D., *Isco Open Channel Flow Measurement Handbook*, 5th ed., ISCO, Lincoln, NE, 1997.
26. U.S. Dept. of Interior, Bureau of Reclamation, *Water Measurement Manual*, Water Resources Technical Publication, 2nd ed., Denver, 1967.
27. Parmakian, John, ‘‘Waterhammer Analysis,’’ Dover Publications, New York, 1986.
28. USEPA. *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Works Systems Using Surface Water Sources.* Malcolm Pirnie, Inc. and HDR Engineering, Inc. for USEPA Science and Technology Branch. Contract No. 68-01-6989 (1990), printed by AWWA, 1991.

SUPPLEMENTARY READING

Hydraulic Institute, ANSI/HI 9.1–9.5-2000 American National Standard for Pumps—General Guidelines for Types, Definitions, Application, Sound Measurement and Decontamination, Parsippany, NJ, www.pumps.org.

Tullis, J. P., and Marachner, B. W., ‘‘Review of Cavitation Research or Valves,’’ *Journal of the Hydraulics Division, ASCE*, pp. 1–16, June 1968.

CHAPTER 27

Storage

INTRODUCTION

Finished-water storage is an essential element serving several functions in any large public water supply system.[1–3] It is primarily used to provide water to meet peak daily and emergency water system demands. Using storage to meet peak demands throughout the day reduces the required sizes and design capacities of other water system elements, such as sources of supply, treatment plants, pump stations, and transmission mains. Storage also helps to maintain uniform pressures throughout the service area. Finally, storage provides a reserve supply that can be drawn on during emergencies such as power outages, fires, and equipment failures. Storage volume requirements for these functions are typically classified as operating, equalizing, fire and/or emergency, and dead storage volumes. These individual volume components are considered in combination to determine the total volume of storage capacity that is required for any system.

CAPACITY

General

In all but the very smallest water service areas, the costs for constructing and operating water storage facilities are partially or totally offset by reduced costs for building and/or reductions in operating costs for other parts of the water system. Many state regulatory agencies have requirements for minimum distribution storage volumes. Adequate storage volumes also can serve to improve local fire insurance ratings and reduce fire insurance premiums. Capacity requirements of storage tanks comprise several component volumes:[4]

- *Operating Storage.* The volume of storage between the ‘‘pump on’’ and ‘‘pump off’’ levels in the tank is normally being used when the sources of supply pumps to the storage tank are off. The pumps supplying the tank will typically be started when the tank has drained to ‘‘pump on’’ level sensor.
- *Equalizing Storage.* This storage component is used when the source pump capacity is less than the peak system demands. The storage is needed so that water production facilities can operate at a relatively constant rate. The volume is determined

by the daily peak rates of water demand, compared to the average daily demand and source capacity.

- *Fire Storage.* The volume of water stored within the water system for fighting fires. The volume of storage required varies with the size of city and with the size, type, and classification of construction within the area served by the storage facility.
- *Emergency Storage.* This storage is used to provide water to the system during other unusual or emergency conditions. A typical example would be a prolonged source of supply failure. The volume needed depends upon the likelihood of an interruption of the source of supply and the time required making repairs or mobilizing an alternative source.
- *Dead Storage.* Storage in tanks or reservoirs that cannot be drawn out or used beneficially because of piping elevations or low pressures is dead storage. This volume is typically most significant in tall standpipe-type tanks where the bottom volume of storage is not usable due to low system pressures.
- *Storage Volume Dedicated to CT.* Finished water storage provided at water treatment plants in clearwells is sometimes used to meet the disinfection contact time or CT. In these cases, the amount of fixed volume used to meet the disinfection requirement should also be considered in the total volume and operational limits of the reservoir. Figure 27–1 illustrates the storage components of the total storage volume.

Operating Storage

Operating storage volume provides a volume of water to allow source and booster pumps to cycle over reasonable operating periods during normal system conditions. The volume is that contained in the storage reservoir between the lower ‘‘pump on’’ level and the higher ‘‘pump off’’ level. Typically these levels are set to optimize the cycle times of the pumps in the source of supply or booster pump station.

Other storage tanks may ‘‘float’’ on the transmission/distribution system hydraulics and not contain level sensors that control supply pumps. An altitude valve may be used to shut off flow to the tank at a preset maximum tank pressure or elevation. In these cases, the tank would not have any operating storage volume.

Equalizing Storage

Common practice in the United States is to provide a volume of equalizing storage so that the supply, treatment, and other water production facilities can operate at a uniform rate during the maximum day demand. Water use is greater during daylight hours—typically peaking in the mid-morning and early-evening hours. Water is withdrawn from storage during these peak demand hours of the day and is replenished during the minimum demands during the late-nighttime and early-morning hours.

Figure 27–2 illustrates the hourly variation in daily water use (diurnal variation) that might occur in a typical residential community on the day of maximum water use for the year. In this example, the maximum hourly rate of use is 175 percent of the average rate of use for the day, and the minimum hourly rate is 21 percent of the daily average. In this example, water is being withdrawn from storage between the hours of 7:00 A.M. and 10:00 P.M., and the reservoir is being refilled from 10:00 P.M. to 7:00 A.M. The equalizing storage volume required to supply hourly demands greater than the average hourly demand for the day is represented by the area under the demand

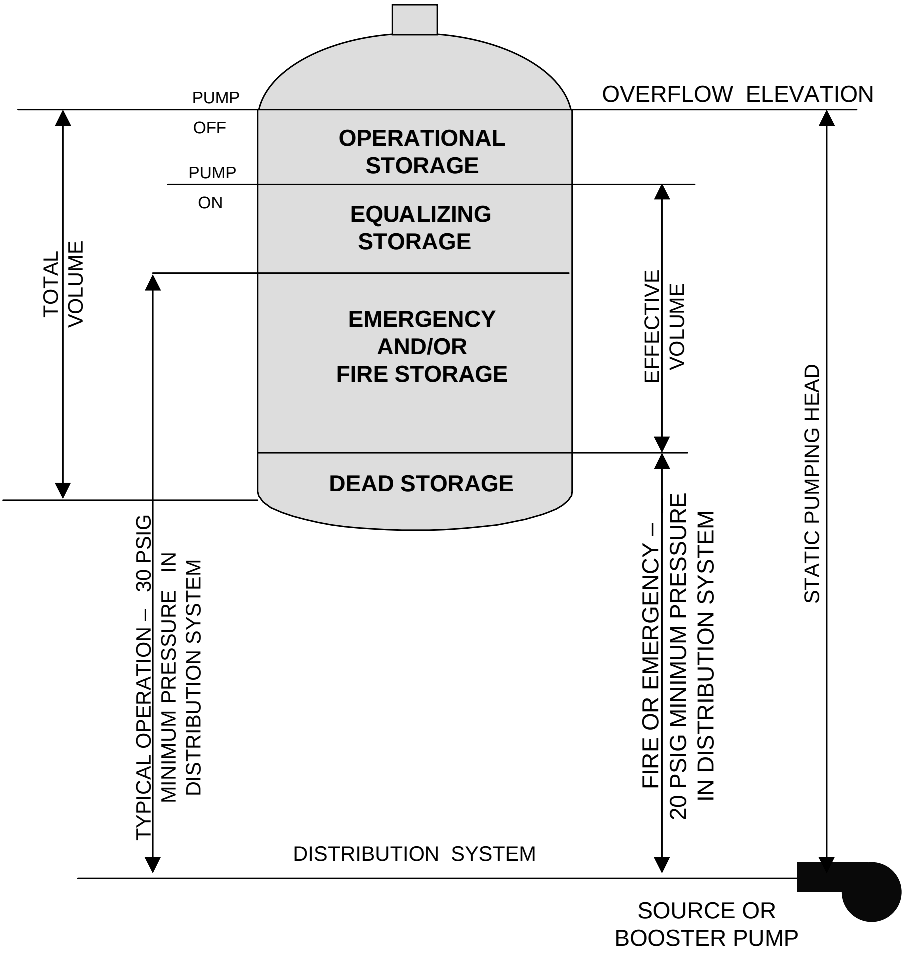


Fig. 27–1. Storage components (Courtesy of Washington State Health Department)

curve and above the line representing average hourly flow rate on the maximum day. Assuming that the source of supply meets the average flow demand during the maximum day, the shaded area below the average flow line would equal the unshaded area under the curve above the average flow line. With this diurnal demand curve, approximately 22 percent of the water volume used on the maximum day would need to be provided in flow equalization storage to meet peak demands. This is represented by the ratio of the shaded area to the total area below the average flow line. For example, if the maximum daily demand for this system were 10 MG (38 ML), then 0.22 ×10 MG = 2.2 MG (8.3 ML) of equalization storage would be required. This example calculation assumes that the source of supply operates for 24 hours per day at the average daily demand rate. Restricting water supply facilities to less than 24 hours of operation would require additional storage volume or a higher-capacity source of supply to meet peak demands.

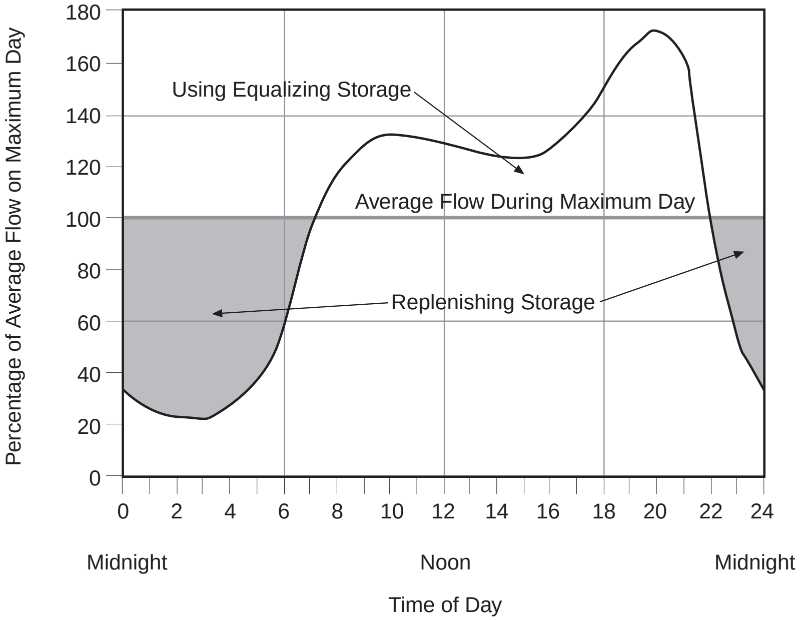


Fig. 27–2. Hourly water use variation

Typical Daily Use of Storage Volume

The shape of the diurnal curve of water demands will vary significantly between different cities because of differences in climates and local economies. A residential/light commercial area in the Northeast United States will have a significantly different demand pattern than a heavy industrial area or a resort area such as Las Vegas, Nevada. Local design data should be obtained for each water system to determine storage needs. However, with adequate source of supply capacity, equalizing storage of about 22 percent of the maximum day demand is typical for small residential areas in many parts of the county.

Fire Storage

Storage volume requirements for fire fighting should be determined based upon state and local fire districts and municipalities. Fire authorities often refer to the latest edition of the *Guide for Determination of Required Fire Flow*, published by the Insurance Services Office (ISO) to determine local fire flow requirements.

The typical minimum municipal fire flow requirement is 500 to 1,000 gpm (3–6 ML/d) for two hours for single family residential areas. Commercial and industrial areas fire flows can be as high as 8,000 gpm (40 ML/d) or more for eight hours or more. Typical fire storage requirements are given in Table 27–1. These storage capacities are given for illustrative purposes only. Actual capacity needs should be determined based upon local fire flow requirements.

TABLE 27-1. Typical Range of Storage Requirements for Fire Protection

Type of Development	Storage Volume, gallons	Storage Volume, ML
Low-density residential, 2 hr at 500 gpm	60,000	0.23
Built-up residential, 2 hr at 1,000 gpm	120,000	0.45
Light commercial, 4 hr at 2,000 gpm	480,000	1.8
Commercial, 4 hr at 4,000 gpm	960,000	3.6

Emergency Storage

The emergency storage volume needed depends upon the risk of interruption of one or more water sources and the length of time that may be needed to make repairs or connect to an alternative source. Unplanned interruptions of supply may occur as a result of well and or intake contamination, transmission main failures, or treatment plant, pump, or power failures. Typically, with only one source of supply, a minimum emergency storage volume would be enough to supply two days of average demand in the area served by the storage facility. If the area is served by multiple supply sources, this emergency volume could be reduced by the volume that could be provided if only the largest source were out of service. The risks of emergency source outages should be evaluated on a case-by-case basis.

Emergency storage can also provide additional water during planned source of supply interruptions. Planned interruptions may be a result of intentional shutdowns for inspection, maintenance, or improvement projects. The probable length of such planned outages should be estimated and scheduled to minimize impacts. The necessary emergency storage volume is then based upon the probable demands during this period.

Dead Storage

It is important to consider that although a storage tank contains a certain total volume, the bottom part of that volume may be unusable in the system. This is primarily because as the storage volume is drawn down toward the bottom the tank, the water pressure becomes inadequate to meet the needs of the system. This condition is especially true in standpipes where only the higher component of the storage volume is usable. Therefore, depending upon service conditions, this dead storage must be considered when determining total beneficial storage.

Total Storage

The total storage required is typically the sum of operating storage, equalizing storage, fire and/or emergency storage, and dead storage volumes. Rather than requiring both fire and emergency storage, some local fire and state agencies will allow the use of the larger of either fire or emergency storage volumes. These local agency requirements need to be determined on a case-by-case basis before designing a storage reservoir. As discussed earlier, these criteria apply where source(s) of supply are capable of meeting the maximum day demand so that storage reservoirs are refilled each day during daily low-demand periods.

NUMBER AND LOCATION OF RESERVOIRS

Under most conditions, the capital cost to construct a single reservoir to store a given volume of water is less than the cost to build several smaller reservoirs to store the same volume. However, the use of several smaller reservoirs to serve different parts of the system may make it possible to use smaller pipelines, and reduce pumping heads. In these cases, the overall system cost may be less with several smaller, well-placed reservoirs rather than a single large reservoir at a central location. Factors to consider in selecting the location of storage reservoirs are:

- Ground elevation
- Effects of location on pressure variations
- Location with respect to the sources of supply and the center of water demands
- Location with respect to pumping stations
- Other issues, such as foundation conditions, visual impacts, and other environmental impacts

The location with respect to water demands and pumping stations may influence the size of an area to be provided with two-directional flow during times of peak water demands. Meeting peak demands from opposite directions will increase the overall capacity of the distribution system. Two-directional flow in the main transmission lines can also increase the volume of water that they can deliver during peak demands. Developing a two-directional flow in a transmission main generally requires that the main source of supply pumping station be located at the opposite side of the center of the water system demand from primary storage facilities. Unfortunately, this arrangement can result in considerable differences in system pressures when source of supply pumps are on or off. Therefore many primary system reservoirs are located closer to the center of water demand. This more central location serves to minimize undesirable pressure variations, but also limits the area capable of receiving flow from two directions.

The hydraulic grade line necessary to provide service pressures to the area or zone being served determines the range of elevations of the reservoir. If the topography is such that the storage reservoir can be located on a hill at the necessary elevation, considerable savings in construction cost can be realized by reducing the height of the structure.

Water systems that receive part or all of their supply from a water treatment plant usually provide part of the necessary system storage at the treatment plant in a clearwell. If space is available and hydraulics and system hydraulics are favorable, the treatment plant is often also used for siting a significant storage reservoir.

TYPES OF STORAGE RESERVOIRS

Conventional storage reservoirs can be classified with respect to their configuration:

- Underground
- Ground level
- Elevated

An alternative to these conventional reservoirs, using a groundwater aquifer for storage, is commonly referred to as aquifer storage and recovery (ASR). ASR provide capacity to store a large volume of water during peak available supply periods to improve aquifer yields during peak seasonal demands.[5] Where groundwater conditions are favorable, ASR can provide a viable method of storing large volumes of water. Treated drinking water is stored in a suitable underground aquifer when surplus source water is available (typically during winter months in northern climates), then recovered when needed during peak demand periods (typically during summer months). Most of the applications of ASR have been for seasonal, long-term, or emergency storage applications.

Selection of the best type of conventional storage for a particular situation depends upon topography, foundation conditions, hydraulic grade lines, economics, freezing conditions, environmental and visual impacts, aviation hazards, and sabotage potential.

Where the bottom of a conventional storage reservoir is below normal ground surface, it should be placed above the groundwater table. Sewers, drains, standing water, and similar sources of contamination should be kept at least 50 feet (15.3 m) from the reservoir or as required by the local regulating agency. The top of a ground-level reservoir should be at least two feet (0.61 m) above flood level.

Large storage requirements may favor ground level, underground tanks, or aquifer storage. Siting a reservoir on a hill at the right location and elevation may provide the benefits of elevated storage at the cost of ground-level reservoirs. In level terrain, consideration should be given to standpipes, elevated tanks, or ground-level or underground tanks with provisions for booster pumping out of the reservoir to maintain system pressures. Elevated tanks are rarely sited in areas that experience significant seismic activity.

Standby emergency power would likely be required in booster-pumping stations to consider some ground-level reservoirs in level terrain as firm sources of water during emergency conditions. Standby emergency power can be in the form of a secondary independent power feed, engine-driven pumps, or an automated gas- or diesel-fueled generator.

Underground tanks may be used when it is necessary to conceal the tanks. This may be done for aesthetic reasons or to facilitate multiple uses of the site, such as for athletics or park activities. Figure 27–3 illustrates the combined use of a tank site for recreational purposes.

DESIGN CONSIDERATIONS FOR CONVENTIONAL STORAGE

General

Finished water storage reservoirs are typically constructed of steel, conventional reinforced concrete, post-tensioned concrete, or wrapped prestressed concrete. High-

Fig. 27–3. Combined use tank site (Courtesy of DYK, Inc.)

density polyethylene liners and floating covers are also used for some in-ground lined and covered reservoirs. The materials and designs used for finished water storage structures should provide stability and durability as well as protect the quality of the stored water.[6–10]

Protection

Covers Stored, finished water in the distribution system may be contaminated by birds or atmospheric fallout that falls into uncovered finished water storage tanks or reservoirs. Open reservoirs are also subject to pollution from activities such as bathing, fishing, deliberate contamination, and sources. All potable water reservoirs should be covered to protect the stored water against chance contamination and quality deterioration.

In addition to the public health protection, covers provide other benefits. They prevent development of tastes and odors by limiting or preventing algae growths. By excluding sunlight they reduce the amount of chlorine lost during storage and may eliminate the need for rechlorination of water leaving the reservoir. Covers reduce the frequency of reservoir cleaning, thereby reducing maintenance costs.

There are numerous ways to cover existing tanks and reservoirs. Covers may be fixed, or float on the water surface. Fixed covers can be made from reinforced concrete, steel, aluminum, or wood. Floating covers are usually made of high-density polyethylene. Fixed covers can be relatively flat (slope to drain), conical, or dome-shaped. The type of cover best suited for a particular installation can be selected on the basis of a number of factors:

- Location, size, shape, and materials used in construction of the existing reservoir
- Footing and other support conditions
- Snow, wind, and seismic loads
- Aesthetics

- Operations and maintenance restrictions
- Capital and maintenance costs, durability, and service life

The capital cost of covering existing open reservoirs has often been a deterrent. Some utilities have added treatment on the outflow of large, uncovered reservoirs in lieu of covering the reservoir. The acceptance of floating covers has substantially reduced costs of covering reservoirs. Floating covers are generally lower in first cost than rigid covers, require little maintenance, are resistant to seismic loads, and can be quickly installed. A floating cover can be installed on a reservoir of nearly any size or shape.[11,12] Refer to the latest edition of AWWA Manual M25, "Flexible-Membrane Covers and Linings for Potable-Water Reservoirs" for more information.

The all-aluminum geodesic dome is a frequently used cover for smaller circular reservoirs. The dome consists of a skeleton of aluminum trusses and a skin of aluminum panels. Although the aluminum geodesic dome has a higher initial cost than other covers, it also provides several advantages: a long service life, minimal maintenance, no painting, light weight, a bottom tension ring that transmits only vertical forces to the reservoir walls, easy reservoir access, fast construction while an existing reservoir is in service, and no interior columns or supports. Figure 27–4 illustrates the use of a geodesic dome cover on a water storage reservoir.

Protection from Trespassers Fencing, locks on access manholes, and other necessary precautions should be installed to deter trespassing, vandalism, and sabotage.

Drains Drains on water storage structures should, if possible, drain to daylight and not have direct connections to a sewer or storm drain. Drain surface outlets should be designed to prevent soil erosion downstream of the discharge. Although a drain con-

Fig. 27-4. Geodesic dome (Courtesy of Northwest Permastore Systems, Inc.)

nection to storm or sanitary sewer is not recommended, any such connection must be designed with cross-connection prevention, such as an air gap of at least two pipe diameters.

Overflow The overflow pipe of a water storage structure should be brought down near the ground surface and discharged over a drainage inlet structure or a splash plate to prevent erosion. Overflows should not be connected directly to a sewer or storm drain, and should adhere to the following guidelines:

- When an internal overflow pipe is used for elevated storage, it should be located in the access tube.
- The overflow of a ground-level structure should be high enough above the normal or graded ground surface to prevent the entrance of surface water.
- Overflow capacity should equal the maximum potential inlet flow.

Mixing and Circulation Water quality may deteriorate in a storage reservoir, especially if the reservoir is poorly mixed or the detention time for part or all of the water is excessive, allowing the residual disinfectant to decay. Poorly planned reservoirs experience thermal stratification, quality deterioration, and microbiological problems. The design of the inlet and outlet piping and the operation of the storage to ensure frequent replenishment of the contents are important considerations in storage design.[10] In some cases, it may be necessary to install internal baffles to promote the circulation of water through the reservoir.

Access Finished water storage structures should be designed with reasonably convenient access to the interior for cleaning, maintenance, and sampling. Manholes on scuttles above the waterline should be:

- Framed six inches (0.15 m) above the surface of the roof at the opening. On ground-level structures, manholes should be elevated above the top to prevent drainage from entering the manhole.
- Fitted with a solid watertight cover that overlaps the framed opening and extends down around the frame
- Hinged at one side and have a locking device

Vents Rigid storage reservoirs should be vented. The net open area vent area must be capable of allowing at least an air volume equivalent to the maximum potential flow water out of the reservoir. These vents should:

- Prevent the entrance of surface and rain water
- Be designed to prevent freezing
- Be screened to exclude birds, animals, and insects with noncorrodible screen and wire mesh. The screen should be designed to swing on hinges or collapse when clogged.
- Not be exposed or vulnerable to vandalism

Roof and Side Walls The roof and side walls of all structures should be watertight, with no openings except properly constructed vents, manholes, overflows, risers, drains, pump mountings, control ports, or piping for inflow and outflow. The following additional factors should be considered:

- Any pipes running through the roof or side wall of a finished water storage structure should be welded or properly gasketed in metal tanks, or connected to standard wall castings that are poured in place during the forming of the concrete structure. These wall castings should have flanges embedded in the concrete.
- Openings in a storage structure roof or top designed to accommodate control apparatus or pump columns should be curbed and sleeved with proper additional shielding to prevent surface water access.
- Valves and controls should be located outside the storage reservoir so that valve stems and similar projections will not pass through the roof or top of the reservoir and will not be subject to contamination by surface water.

Drainage for Roof or Cover The roof or cover of the storage structure should be well drained with a minimum 2 percent slope. Downspout pipes should not enter or pass through the reservoir. Parapets or similar features that would tend to hold water and snow on the roof are not recommended. Floating covers should be designed with perimeter systems to collect and drain or pump away rainwater.

Freezing All finished water storage structures and their appurtenances, especially the riser pipes, overflows, and vents, should be designed to prevent freezing. Consideration should be given to heating and/or insulation of exposed pipes and valves.[13]

Internal Walkways Every walkway over finished water in a storage structure should have a solid floor with raised edges so that shoe scrapings and dirt will not fall into the water.

Grading The area surrounding a ground-level structure should be graded in a manner that will prevent surface water from standing within 50 feet (15.2 m) of the structure.

Painting and Cathodic Protection The AWWA standards for painting exclude the use of paints that might add toxic materials to the stored water through leaching or other action. Proper protection should be given to metal surfaces by paints or other protective coatings, by cathodic protective devices, or by both. Use paint systems that are consistent with AWWA standards and are ANSI/NSF certified for potable water contact. Cathodic protection devices must be regularly inspected and maintained for satisfactory performance. Painting and maintenance of storage tanks are critical elements in ensuring public safety and ensuring a long tank service life.[14–22]

Exterior and interior coating systems for welded steel tanks and recoating of assembled bolted steel tanks are specified ANSI/AWWA Standard D102. The quality of application of any field-applied coating system is of critical importance in the long-term serviceability of the tank. Therefore, prior to selecting and specifying the coating systems, the designer must consider or define acceptable ambient conditions expected during coating application.

Factory-applied coating systems for bolted steel tanks should be in accordance with ANSI/AWWA Standard D103. This standard covers glass-fused-to-steel, thermoset liquid suspension, and thermoset powder coatings. Several manufacturers offer bolted steel tanks with a variety of coating systems. For example, glass-fused-to-steel tanks can be up to 160 feet in diameter and store up to 3 million gallons (11.2 ML).

Cathodic protection systems for steel tanks work in concert with coating systems to protect the steel at flaws or damaged areas in the coating. Active or impressed-current systems use a rectifier to discharge a current through a suspended anode system. The impressed current ensures that the suspended anodes and not the steel tank are sacrificed in the corrosion process. The amount of impressed current required to protect the tank will vary with the condition of the coating.

Materials Steel and concrete are permitted by most states. Other suitable materials may be approved at the discretion of the regulatory agencies. Welded steel water tanks should conform to ANSI/AWWA Standard D100. Factory-coated bolted steel tanks should conform to ANSI/AWWA D103. Wire- and strand-wound, circular, prestressed concrete tanks should conform to ANSI/AWWA Standard D110. In addition to burying concrete tanks, steel or concrete tanks may be architecturally appointed to disguise function or help the structure blend in with surroundings. Examples of a variety of concrete and steel water tanks are shown in Figures 27–5, 27–6, and 27–7.

Safety

The safety of employees should be considered in the design of the storage structure. The design must conform to laws and regulations of the area where the reservoir is constructed. Other considerations:

- Ladders, ladder guards, balcony railings, and safe location of entrance hatches should be provided where applicable.
- Appropriate safety devices should be installed to comply with confined-space entry requirements.

Fig. 27-5. Disguised concrete tanks

Fig. 27-6. Elevated concrete and steel tank

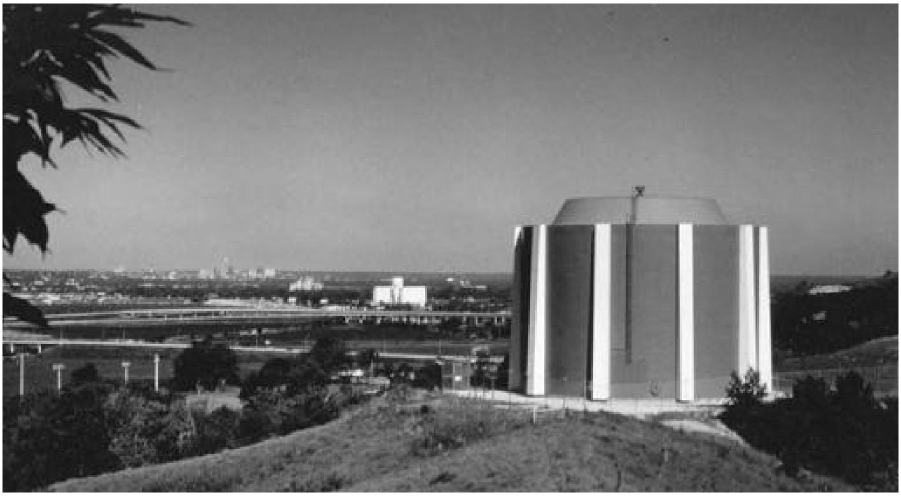


Fig. 27-7. Steel tank

- Elevated tanks with riser pipes over 8 inches (0.2 m) in diameter should have protective bars over the riser openings inside the tank.

Disinfection

New reservoirs and reservoirs that have been emptied for service or repair should be disinfected to protect against any contamination introduced by workmen or materials during the course of construction or maintenance. Detailed procedures, equivalent to those outlined in the current American Water Works Association Standard D102 for painting and repairing steel tanks, standpipes, reservoirs, and elevated tanks, should be used for tank disinfection. Also, the following provisions should be made:

- Two or more successive sets of samples, taken at 24-hour intervals, should indicate bacteriologically satisfactory water before the facility is released for use.
- Smooth end taps in riser pipes and connecting mains where disinfectant is added should be provided.

Plant Storage

Water storage may be provided at the treatment plant for in-plant uses (the principal in-plant use is filter backwashing) or to reduce the total amount of distribution system storage required for flow equalization from the treatment plant.

Clearwell Clearwell storage can, in conjunction with distribution system storage, relieve the filters from meeting all fluctuations in water use and peak demands. When finished water storage is used to provide contact time for disinfection (CT), special attention must be given to volume and baffling to promote channeled circulation through the tank.

For conventional treatment plants, the capacity of the clearwell must be sufficient to provide filter backwash water for the plant and equalizing storage for the plant's high-service pumping station. In most cases, a clearwell capacity equal to 10 percent of the maximum treatment plant design capacity is adequate. However, clearwell capacity needs should be calculated for specific local conditions. The balance of the water system's storage needs should be provided within the distribution system.

Washwater Tanks In the absence of a larger clearwell, washwater tanks should be sized, in conjunction with available pump units and finished water storage, to provide backwash water. The capacity in storage required for this in-plant use is the quantity of water required to backwash a filter for the design backwash flow rate and duration. However, consideration should be given to the possible need to wash more than one filter at a time, or several filters in succession.

Adjacent Compartments Finished water should not be stored or conveyed in a compartment adjacent to nonpotable water when the two compartments are separated by a single wall, because of the risk of leakage of nonpotable water into potable water storage through cracks or leaks in the dividing wall.

Basins and Wet Wells Receiving basins and pump wetwells for finished water should be designed as finished water storage structures.

Other Design Consideration

Pressure Variation To limit water service pressure changes to about 14 psi, the maximum desirable variation between high and low levels in storage structures that are hydraulically directly connected to or ‘‘float’’ on a distribution system is approximately 30 feet (9.1 m). Larger variations can be tolerated if other system features, such as pressure-reducing valves, are used to minimize pressure fluctuations to customers.

Level Controls Adequate controls should be provided to maintain levels in distribution system storage structures, as follows:

- Telemetering equipment should be used for storage structures when pressure-type controls are employed in control stations and any appreciable headloss occurs in the distribution system between the source and the storage structure.
- Altitude valves or equivalent controls may be desirable to prevent overflowing of a second and subsequent storage structures hydraulically connected on the system. High-water levels should be set 2–3 feet (0.60–0.91 m) below the overflow level.

Water Quality Considerations Good circulation of water through reservoirs can assist in the control of tastes and odors, maintenance of residual disinfectant, and control of slime growths. Reservoirs should be cleaned periodically to prevent accumulation of sediments and the creation of chlorine-demanding residues. Other design considerations are as follows:

- Water circulation through reservoirs should be promoted by the use of baffles, or by placing inlets and outlets on opposite sides of the reservoir with inlets near the top and outlets near the bottom.
- All pipes except the overflow should have valves.
- The minimum reservoir water depth should be about 12 feet (3.66 m).
- More than one reservoir should be provided, to give storage during outages; or, if there is a single reservoir, it should be divided into compartments so at least one section is available for use at all times.
- Reservoirs should be drained and cleaned approximately every two to three years, depending upon local conditions.

Hydropneumatic Tanks

In some cases, it may not be financially feasible to provide gravity storage in some areas of the water system. In such cases, hydropneumatic tanks often are installed as a limited means for equalizing flow and pressure and reducing the frequency of pump-cycling.[23]

When used, they should meet ASME Code requirements for unfired pressure vessels and they should comply with state and local laws and regulations. The following typical hydropneumatic tank requirements have been developed based upon operating experience:

- The tank should be located above the normal ground surface. In areas where freezing occurs, the tank should be completely housed, or earth-mounded with one end projecting into a piping/operations building.
- The tank should have bypass piping or a duplicate unit to permit operation of the system while the tank or the tank accessories are being repaired or painted.
- Each tank should have an access manhole, a drain, and control equipment consisting of pressure gauge, water sight glass, automatic or manual air blow-off, mechanical means for adding air, and pressure-operated start–stop controls for the pumps.
- The total capacity of the wells and pumps in a hydropneumatic system should be at least six to 10 times the average daily consumption rate of the community. A minimum of two wells or pumps should be provided.

For the supply of water from a pressure tank system to be adequate, the storage and water mains must be sized to meet peak demands. In small service areas, peak flow rates may be as much as 10 times greater than the average annual flow rate. Where available, meter readings should be used to determine average and peak flow rates. Pump systems should be capable of delivering 125 percent of the peak flow. Hydropneumatic tanks typically are sized to have a capacity equal to 30 times the pump capacity. Another rule of thumb is to provide 10 gallons (40 L) of storage per person served. In general, only about 10 to 20 percent of the total volume of the pressure tank is available storage.

A simple and direct method for determining the recommended pump size and pressure for conventional horizontal tank capacity was given by J. A. Salvato, Jr.[24] It is based on the following formula:

$$V = \frac{V_m}{\left(1 - \dfrac{P_1}{P_2}\right)}$$

where:

V = pressure tank volume, gallons (m^3)
V_m = 15 minutes storage at the peak hourly demand rate, gallons (m^3)
P_1 = minimum absolute operating pressure, psi (kPa)
= gauge pressure plus 14.7 (101.3 kPa)
P_2 = maximum absolute pressure, psi (kPa)
= gauge pressure plus 14.7 (101.3 kPa)

The design of bladder-type pressure tanks must also consider the number and size of tanks to provide pump protection and the precharged air pressure of the tank.

Aquifer Storage and Recovery (ASR)

General The applicability of ASR is generally evaluated using three principal criteria:

- Variability of the water supply, demand, and quality
- Sizing of the facility, since a minimum scale of development exists below which ASR may not be cost-effective
- Storage zone characteristics

Applications ASR provides seasonal, long-term, or emergency storage of drinking water in confined or unconfined aquifers.[5] This technology has other applications beyond typical storage purposes, including treatment, groundwater restoration, subsidence reduction, and wellfield enhancement and augmentation. ASR can also have application where there are large seasonal variations in water availability and there is limited siting for additional surface storage reservoirs.

Regulatory Issues As an injection practice, ASR wells are regulated by the Underground Injection Control Program, administered by the USEPA under the Safe Drinking Water Act. Given the concern over aquifer water quality, generally, water injected into an ASR aquifer should meet potable water standards. Each state has varying standards and permitting requirements that must be considered for this storage concept.

Technical Considerations The concept of ASR is to inject and withdraw water through the same well. This approach allows backflushing of the well to reduce aquifer clogging and maintains the storage bubble to minimize intermixing of the stored water with native water. Using this technique, it is not uncommon to store treated drinking water in freshwater or brackish water aquifers with varying geologic conditions. Selecting the appropriate aquifer is important to reduce loss of stored water, contamination, and intermixing.

REFERENCES

1. Brock, D. A., ''Determination of Optimum Storage in Distribution System Design,'' *JAWWA*, p. 1027, August 1963.
2. EPA, ''Manual for Evaluating Public Drinking Water Supplies,'' Superintendent of Documents, Washington, D.C., 1971.
3. Culp, Russell L., EPA Technical Guidelines for Public Water Systems, Chapter VI, ''Storage,'' NTIS, PB-255-217, U.S. Department of Commerce, Springfield, Va., June, 1976, p. 195.
4. Washington, State Department of Health, Water System Design Manual, June 1999.
5. AWWA Research Foundation, ''Aquifer Storage Recovery of Treated Drinking Water,'' 1996.
6. Wade, J. A., ''Design Guidelines for Distribution Systems as Developed and Used by an Investor-Owned Utility,'' *JAWWA*, p. 346, June 1974.
7. Jorgenson, I. B., and Close, S. R., ''Arvada 10-mil gal Treated-Water Storage Reservoir,'' *JAWWA*, p. 181, April 1976.
8. *AWWA* Committee Report, ''Multiple Use of Covered Distribution Reservoirs,'' *JAWWA*, p. 262, May 1978.
9. Larsen, L. A., ''Cold Weather Operation of Elevated Tanks,'' *JAWWA*, p. 17, January 1976.
10. AWWA Research Foundation, ''Water Quality Modeling of Distribution System Storage Facilities,'' 1999.
11. Dellaire, Gene, ''The Floating Cover: Best Way to Cover a Finished-Water Reservoir?,'' *ASCE Civil Engineering*, p. 75, June 1975.

12. Elwell, F. H., ‘‘Flexible Reservoir Covers: A Case Study,’’ *JAWWA*, p. 210, April 1979.
13. Wormald, L. W., ‘‘Water-Storage-Tank Failure Due to Freezing and Pressurization,’’ *JAWWA*, p. 173, March 1972.
14. Harper, W. B., ‘‘Inspecting, Painting, and Maintaining Steel Water Tanks,’’ *JAWWA*, p. 584, November 1982.
15. Brotsky, Bob, ‘‘Interior Maintenance of Elevated Storage Tanks,’’ *JAWWA*, p. 506, September 1977.
16. AWWA, *AWWA Standard for Coating Steel Water-Storage Tanks*, AWWA D102-97, AWWA, Denver, Colo., 1997.
17. AWWA, *AWWA Standard for Inspecting and Repairing Steel Water Tanks, Standpipes, Reservoirs, and Elevated Tanks, for Water Storage*, AWWA D101-53, (R86) AWWA, Denver, Colo., 1986.
18. AWWA, *AWWA Standard for Welded Steel Elevated Tanks, Standpipes, and Reservoirs for Water Storage*, AWWA D100-96, AWWA, Denver, Colo., 1996.
19. Crowley, F. X., ‘‘Maintenance Problems and Solutions for Prestressed Concrete Tanks,’’ *JAWWA*, p. 579, November,1976.
20. Hertzberg, L. B., and Westerback, A. E., ‘‘Maintenance Problems with Wire-Wound Prestressed Concrete Tanks,’’ *JAWWA*, p. 652, November,1976.
21. Woodside, R. D., ‘‘Prestressed Concrete Storage Tanks: Design and Construction,’’ *JAWWA*, p. 389, May 1972.
22. Harem, F. E., ‘‘Maintenance and Repair of Concrete Reservoirs,’’ *JAWWA*, p. 1160, September 1964.
23. Salvato, J. A., Jr., *Environmental Sanitation*, John Wiley & Sons, New York, 1958.
24. Salvato, J. A., Jr., ‘‘The Design of Pressure Tanks for Small Water Systems,’’ *JAWWA*, p. 532, June 1949.

CHAPTER 28

Distribution Systems

INTRODUCTION

A water distribution system conveys water from the treatment facilities to the user. The distribution system should supply water, without impairing its quality, in adequate quantities and at sufficient pressures to meet system requirements.

The facilities that make up the distribution system include finished water storage, pumping, large-scale transmission and distribution piping, supply mains, and appurtenant valves. This chapter will discuss the distribution system components not covered in other chapters, the basis for their sizing and design, considerations for construction and maintenance, and techniques to evaluate the impacts of water demands on existing systems. Storage facilities are discussed in Chapter 27, and pumping systems in Chapter 29. Water system planning is discussed in Chapter 7.

The water distribution system must be capable of supplying water needed for basic domestic purposes and commercial and industrial uses, and where possible, the flows necessary for fire protection. The safety and palatability of potable water should not be degraded as it flows through the distribution system (refer to Chapter 22).

SYSTEM CAPACITY

Distribution system design or analysis is based on peak hour demands or fire flow demand, while other system demands are something less than peak hour. Computerized analysis permits running simulated extended periods of water demand and system storage replenishment. Acquiring the basic data to calibrate the model and identify detailed variable system demands may be a straightforward procedure or it may be difficult, depending on the existence and type of records available. More likely, the engineer will be required to apply empirical ratios to simulate peak demands and demand variations.

In most water systems there are a variety of customer classes and system needs.[1] Water use and demand will vary, depending on customer class. Customer classes may be broken down into residential and commercial/industrial, or may be more finely divided into single-family residential, multifamily residential, parks, schools, heavy industrial, light industrial, commercial, and so forth. The finer the breakdown and the more accurate the data, the better the determination of demand requirements.

Once the various system demands are determined, it is then possible to analyze the water distribution system.

SYSTEM DESIGN

Nonhydraulic Design Considerations

A number of factors unrelated to system hydraulics must be considered in water distribution system design. These nonhydraulic design considerations are summarized in the following paragraphs.

The ability to isolate parts of the system is important, especially during emergency operation conditions (e.g., main breaks). All water distribution systems should be provided with sufficient isolation and drain valves to permit necessary repairs without undue interruption of service over any appreciable area.

Valves should be placed in numbers and locations that give the best possible control of the system consistent with cost limitations. Valves in smaller mains are typically more numerous than valves in larger mains. Valves should be located where they will be readily accessible in the event of a main failure. Branch mains, connecting to larger primary or secondary feeders that cross under arterial highways or streets, are usually valved close to the larger main before the crossing.

In transmission lines, valve spacing is usually determined by operating requirements, and thus is a matter of individual design. In feeder mains, valves are usually spaced so that each feeder loop can be effectively isolated. Usually this spacing will not exceed 3,000 feet (914 m) in 16-inch (0.41-m) feeders or 4,000 feet (1.2 km) in 20-inch (0.51-m) feeders.

Service main valves will be spaced so that adequate shutdown capability is provided without putting large numbers of customers out of service. In residential areas or districts, valves in 6- and 8-inch (0. 15- and 0.20-m) mains should be spaced no more than 1,000 feet (305 m) apart, and in 10- and 12-inch (0.25- and 0.31-m) mains no more than 2,000 feet (610 m) apart. Normally, each smaller main will be valved in intersections involving 10-inch and 12-inch (0.25-m and 0.31-m) mains connecting to smaller mains. Service mains crossing creeks, railroads, and expressways will be valved on each side of the crossing. Each fire hydrant branch should be equipped with a control valve.

Valve sizes are normally the same as the water main in which they are installed, except that in mains 30 inches (0.76 m) or larger, line valves are often one size smaller than the main size.

Hydraulic Design Considerations

The design of water distribution systems may rely entirely on detailed calculations of system hydraulics, or may depend in part on minimum design standards defined by the owner of the distribution facilities. For instance, some utilities size transmission lines based on limiting the headloss to 1 foot per 1,000 feet of length. Others specify using a minimum-size pipe every mile or half-mile in a grid system. Design based on minimum standards typically will meet or exceed hydraulic requirements under all the ordinary system needs. Minimum design standards generally define materials and construction methods, minimum pipeline sizes, maximum pipeline headloss or velocities, and various criteria for ensuring adequate system flows and pressures without detailed hydraulic analysis of system extensions.

An example of one such set of design standards is the Recommended Standards for Water Works published by Health Education Services in Albany, N.Y.[2] This publication is commonly referred to as the Ten States Standards. The Ten States Standards, or similar standards, can be used if detailed hydraulic engineering studies are not a viable option for the utility.

System Pressures

It is important to establish criteria for minimum and maximum system pressures occurring during the peak hour demand. It is also desirable to establish maximum pressure fluctuations within the water distribution system. Typically, minimum acceptable water system pressures are 35 to 40 psi (240 to 275 kPa), and maximum pressures are 100 to 120 psi (690 to 825 kPa). A minimum system pressure of 35 to 40 psi (240 to 275 kPa) ensures adequate flows to the individual consumers and allows for reasonable operation of home-type irrigation/sprinkler systems. A more desirable low system pressure may be in the range of 50 psi (345 kPa). This level of low system pressure provides adequate flows and working pressures for most typical residential and commercial uses. Maximum pressure limitations are desirable to minimize the additional cost of providing piping materials with adequate strength to cope with the high pressure. In addition, high pressures can injure existing residential and commercial plumbing systems. Where main pressures exceed 100 to 110 psi (690 to 760 kPa), individual pressure-reducing valves should be installed on each service.

Minimizing the pressure fluctuations in the distribution system provides reliability for the consumer. Irrigation or in-house water-based appliances operate more consistently when pressure fluctuations do not exceed certain limits. Wide system pressure variations make proper design and selection of these appliances difficult and can also create operational difficulties. Typical goals for pressure variation limitation at any single point in a distribution system is 10 to 15 psi (69 to 103 kPa).

Water system design may be based entirely on a hydraulic analysis that selects pipe sizes to ensure adequate pressures within all areas of the water distribution system under a set of system demand or loading conditions. For some water systems, a detailed hydraulic analysis may be done for every development area or main extension. In other systems, design may be based on a combination of minimum design standards and hydraulic analysis. Hydraulic analyses may be as complex as extensive modeling of the water distribution system, or as simple as a hydraulic evaluation of a single pipe utilizing standard hydraulic tables.

WATER DISTRIBUTION SYSTEM MODELING AND HYDRAULIC ANALYSIS

Modeling Overview

Water distribution system modeling and hydraulic analysis are based on fluid mechanics. The analysis and the simulations rely on basic equations used to determine friction headloss. Equations that have been applied in hydraulic analysis and simulation include the Darcy-Weisbach, Hazen-Williams, Manning, and Darcy-Weisbach/Colebrook-White formula pipe flow equations. The fundamentals of fluid mechanics are described in several basic texts.[3–5]

The design of a single pipe or main may be accomplished using any of the above equations or by the use of standard hydraulic tables or nomographs based on the above equations.[6] A multiple-pipe network branching to form complex, looped systems requires simultaneous or repetitive and converging solutions of multiple equations. The analysis and design of complex piping networks can be tedious, especially if the network consists of a large number of pipes and system appurtenances. Historically, water system simulation or hydraulic analysis required the distribution system to be reduced in complexity for it to be analyzed, yet retain enough detail to provide a meaningful result. This approach was referred to as ''skeletonization'' of the water distribution system. During the past several years, this limitation has been eliminated by the advancements in computer hardware and software programming techniques. Skeletonization is not recommended for water quality modeling applications because the flow values, paths, and velocities for all pipes are critical elements of water quality evaluation.[7] Skeletonization is still used in many situations and will be elaborated on later in this section.

The oldest, and probably the most widely utilized, method for pipe network analysis is the Hardy-Cross method. This method is based on trial and error, with corrections applied to an initial set of assumed flows or heads until the network is balanced hydraulically. This type of iterative calculation is amenable to computerization; however, the Hardy-Cross method is cumbersome for the computer because the mathematics hinder the speed at which solutions can be obtained. The Hardy-Cross method, developed before the common use of computers, used hand computations to solve pipe network problems. A number of other ''office study'' methods are available, as described in early water engineering books. The earliest ''electric'' methods for hydraulic analyses were described by Howland, Camp, and Hazen, and a commercially available electric analyzer was developed by McIlroy.[8–10]

Subsequent computer-based hydraulic analysis techniques employ pipe friction formulas with less cumbersome mathematical approaches to reduce both memory requirements and computational time. Like the Hardy-Cross method, the pipe friction equations are nonlinear and require solution by trial and error. These trial-and-error solutions may require a significant amount of computer memory and computational time; hence, various mathematical methods have been applied to accelerate solution of the equations. Jeppson provides a detailed discussion of the Newton-Raphson method, one of the most widely used, as it applies to hydraulic simulation.[11] This method requires an initial guess of the solution. If the initial guess or the initialization provided in the Newton-Raphson equation is not reasonably accurate, convergence may be difficult. However, when the initial values are sufficiently close to the solution, the convergence may be quadratic. The method obtains the solution to a system of nonlinear equations by iteratively solving a system of linear equations.

The number of computer programs currently available to perform hydraulic analysis and water system simulation is significant and expanding. These programs range in capability and complexity from those that operate on handheld calculators for solving problems dealing with up to four-loop systems, to commercially available programs written for personal computers capable of analyzing networks with an unlimited number of pipes; Table 28–1 lists examples of distribution system modeling software. These programs are capable of analyzing distribution networks containing thousands of pipes and other water system appurtenances, including pumps, pressure-reducing valves, and system storage reservoirs. Output capabilities range from flows and direction of flow in pipelines and pressures at given points within the distribution system, to programs capable of summarizing all the desired system characteristics and providing a graphical plot of the distribution system, including plotting of isopressure con-

TABLE 28-1. Examples of Distribution System Modeling Software

Name	Supplier
EPANET	U.S. E.P.A. (public domain)
Boss EMS (EPANET Modeling System)	Boss International
Cybernet/WaterCAD	Haestad Methods
H20NET	MWSoft
Pipe2000/KYPIPE	University of Kentucky
SynerGEE	Stoner Associates, Inc.

tours. Distribution system water quality data may also be obtained from the modeling software, as part of the general package or as an add-on module.

Some of the software is free and other software available must be purchased for each software license (copy) required by the user. The cost of this software varies greatly, depending on the number of software licenses required, number of pipes to analyze, and program options chosen.

Many of the currently available software programs provide steady-state, or static, simulation of the water system under given conditions of supply and demand. Under this steady-state analysis, the solution is achieved when the flow rate in each pipe is determined. The output is based on fixed system characteristics, including single-condition values for pressure-reducing valves and single-head increases for pumping and booster stations. Alternatively, some programs offer extended period simulation (EPS) solutions based on time period simulation of the water system under varying supply and demand conditions. Examples of system characteristics that can be varied are: pressure-reducing-valve set points, variation in pump curve characteristics, or a change in demand throughout the system or at a discrete node. The EPS is performed by setting the desired duration for the simulation and a time increment for calculations, generally one hour.[7] Each increment is evaluated as a steady-state condition and then the results are transferred to the subsequent steady-state condition.

It is important to plan and operate the distribution system so as to ensure an adequate level of service to all users in the distribution system under varying conditions of loading. Criteria for determining the level of service include:

- Maintaining flows and pressures (heads) at various points in the system within limits. It may be acceptable to vary these limits with time.
- The management of storage to balance the supply and the distribution.

In system design and operation, it is beneficial to know the impact of changes in demand on the level of service. This knowledge aids in the development of design and control strategies to maintain the level of service. There is also a need to evaluate the adequacy of storage or of proposed network additions with respect to an increase in total demand. Both of these needs can be met by simulating the behavior of the system over a period of 24 to 48 hours under changing demand patterns. A static solution will not be so informative in terms of changing conditions, because a static solution provides only a one-time evaluation, or a ‘‘snapshot,’’ of the system characteristics.

Another hydraulic simulation technique is the application of linear theory.[12] This method is easily applied if all external flows to the system are known. The linear theory method has several advantages over the Newton-Raphson or the Hardy-Cross methods, because it does not require an initialization and it converges in relatively few iterations. Linear theory transforms the nonlinear loop equations into linear equations by approximating the head in each pipe. A system of linear equations is developed that can be solved by linear algebra.

An excellent discussion of hydraulic analysis of pipe networks is presented by Jeppson.[11] In this reference, the reader may find detailed discussion of several methods, the mathematics and examples of which are beyond the scope of this chapter.

Skeletonizing the Water Distribution System

Water systems may be large networks consisting of hundreds or thousands of pipes and many different sources of water supply. To analyze such systems, it may be important to achieve a representation of the network consistent with computational capabilities. This representation must also be detailed enough so that the system can be accurately analyzed to meet the desired objectives of the study. Distribution systems are also skeletonized to make the analysis of the system more easily understood.

The above goals are accomplished by first skeletonizing the system, which reduces the number of pipes analyzed by the modeling software using the equivalent pipe method. An equivalent pipe is one in which the loss of head for a specified flow is the same as the loss in head of the pipes that it replaces. This skeletonizing may involve the total dismissal of an unimportant or minor pipe, the replacement of a series of pipes of varying diameter with one equivalent pipe, or replacing a system of parallel pipes with an equivalent pipe.

When skeletonizing the system, it must be recognized that the backbone of the water distribution system is the transmission mains. Large water systems may contain transmission mains that are 16 inches (41 mm) and larger. Connections to transmission mains are typically held to a minimum; so it is typical to utilize all of the existing transmission piping with no skeletonization. Connections from the transmission mains to the distribution system can then be considered the demand, or load points, on the system. Water distribution piping is interconnecting pipes, which are the principal components of the distribution system.

The decision on how various pipes are included in the model is based largely on the experience of the individual conducting the analysis. If the water system being evaluated is small, it may be desirable to include all of the system piping. If the system is very large, it may be desirable that pipes under a specific size—12 inches (30 mm), for example—not be considered, unless they are necessary to complete a loop. For medium-size systems, one must consider the effect of ignoring an existing pipe. If problems become apparent in areas of the system where pipes have been ignored, then the decision to ignore those pipes must be reevaluated.

Another factor affecting the amount of system skeletonizing is the intent of the simulation analysis. When the purpose is to examine the impacts of fire flow in large residential areas, it is necessary to include a significant portion of smaller-diameter piping. However, if the intent is to evaluate the impact of a large development on the transmission system, the majority of the small-diameter piping can be ignored.

Minor Losses

In the early analysis techniques, minor losses were included in the analysis. Water flowing in a straight pipe under pressure at a constant velocity experiences a constant friction loss, and a change in the velocity or direction of the flow creates additional losses. Such losses are classified as minor, as they are not significant compared to the pipeline friction losses. A pipeline with a length in excess of 500 diameters is usually classified as long.[13] Minor losses become important when small-diameter pipes and high velocities are considered in the total piping and appurtenances relating to a pumping station. Minor losses are produced by flow-through bends, fittings, valves, and openings such as fire hydrants, and in and out of reservoirs.

In large-scale network simulations, the minor losses may become insignificant and can be ignored. Deciding when to include minor losses is a matter of judgment; when minor losses are included, they can easily be introduced as equivalent lengths of pipe, although some programs provide for inclusion of these losses separately. Where C values (coefficient of pipe roughness) are determined from field measurements, they invariably include a component due to the various minor losses encountered.

Pipe Friction Factor

One of the input requirements for most water system simulation models is the pipe friction factor appropriate to the pipe flow equations upon which the model is based.

The interior of a pipeline changes with time, and these changes affect the pipe friction factor. Typically, over time, the friction head losses in a section of pipe will increase with the pipe's age, because of various physical and chemical characteristics of the water that change the finish or roughness of the inside of the pipe. The inside diameter of the pipe also may be reduced. Actions that affect the line capacity include sedimentation, scaling, organic growth, tuberculation, and corrosion. The effect of these actions may be partially reversed by pipeline cleaning.

When performing water system simulation, it is important to have relatively accurate information on the friction loss characteristics of the distribution piping. The hydraulic capacity of pipelines may be determined by conducting flow tests on representative sections of the pipelines. Procedures for such tests are based on a measured flow through a straight section of pipeline (as long as possible) with the pressure drop recorded by gauges installed at both ends of the section.

Model Input and Output Data

The water distribution system is defined to the model as a set of ‘‘nodes’’ and ‘‘lines.’’ Nodes are connected in pairs by network elements (lines) such as pumps, pipes, one-way valves, pressure-reducing valves, and so forth. The nodes are specified as having either a fixed head or a fixed flow. An outflow is considered to be a demand on the system; an inflow is considered to be a supply (well, treatment plant, or reservoir) to the system. Nodes may have flow values of 0—thus no inflow or outflow. These nodes are used solely for the purpose of connecting different distribution system facilities. This schematic representation is then translated onto a computer coding form for input into the computer.

Computer programs simulate the conditions within a water distribution system based on data provided by the user. For best results, it is important that the input data be accurate, and that all assumptions regarding input data be carefully made. It is also necessary that the model being used describe the behavior of the existing system. This can be checked by ‘‘calibrating’’ the model, which requires running the model under a set of known conditions. Typically, a number of recording pressure gauges will be in operation on an existing water system while demands are carefully monitored. It is then possible to compare the computer output results with those obtained in the field. Adjustments can then be made in the input assumptions to make the model simulate the real system.[14]

When applying the model, the output data provided by the model should not be construed as a true representation of the real conditions without reviewing the assumptions applied to the input data. In municipal water supply, loads at individual nodes are usually not precisely known. The flows into the system are usually metered so that the total load is known, but the problem lies in the proper distribution of individual demands to nodes. Statistical methods based on customer meter records can help.

In many water systems, there may also be considerable uncertainty about pipe roughness characteristics and actual pipe diameters. In comparison to the existing system, the program predicted pressures at specific points in the system within an accuracy of 5 to 10 percent. A predictive accuracy of system pressures in the 10 percent range should not cause problems in the interpretation of the model output results or ultimate decisions based on these results. However, it is strongly recommend that calibration of the model be performed to ensure the results are within the range listed above.

Extended Period Simulation (EPS)

Modeling using EPS methods can also be used to determine the impact of current demand conditions on an existing water system. When a problem has been detected in the field or are predicted by the model output, alternative solutions may be investigated to obtain the best and most economical solution to the problem. EPS modeling can be invaluable in system operations, as alternative operation strategies can be evaluated. As previously indicated, sophisticated models are capable of incorporating pumping stations, valves, and appurtenances along with the system piping, and these capabilities allow evaluation of operational alternatives as well as construction alternatives.

The water system EPS model may, by varying the demands on the system, determine effects at critical periods. These may include maximum hour demand, fire demands consistent with maximum day demands, and storage replenishment conditions. This information can provide valuable assistance in the cost-effective design of distribution systems. When major development is planned within an existing water system, water system EPS modeling not only can show the impact of that development on the existing system, but can assist in the sizing of the lines within the development itself.

Modeling Summary

In choosing a water distribution system model, it is necessary to evaluate the available input data, the size and complexity of the system to be analyzed, the desired uses of the model output, and the number of alternatives that are to be evaluated. One may find a simple Hardy-Cross evaluation of a small water system to be more efficient than pursuing a detailed and complex computer evaluation of the system. However, if a number of alternatives are to be evaluated, or if the simulation model will serve as a major design tool for future system expansion, the use of a highly efficient computerized model will be the best choice.

Computers and modeling software provide the ability to analyze complex systems quickly. This is an important benefit in view of the costs of operation of water systems, especially energy costs. Simulation can also reduce construction costs by allowing a determination of the most cost-effective of several possible solutions.

The use of computer models has increased extensively over the past few years, and the available programs have become more and more advanced. Some of these programs interface several peripheral programs to allow for a more exact analysis and easier database handling.[7,15]

PIPELINE LOCATION, PROTECTION, AND MATERIALS OF CONSTRUCTION

Water distribution systems can be extensive, and they are expected to be in service for long periods of time without significant cost for maintenance, repair, or replacement. For these reasons, not only are the proper sizing and design of systems important, but so are their construction and maintenance.

In general, all mains should be installed in dedicated public streets or in other public access ways with a minimum of 30 to 50 feet (10 to 15 m) of right-of-way. This will ensure adequate access for normal and routine maintenance as well as emergency repair procedures. Unless it is absolutely unavoidable, mains should never be located on private property, under structures, or under or in lakes. In the event that a pipeline must be installed across private property, agreements should be developed to ensure that no permanent structures will be constructed within the permanent easements. This will result in pipeline accessibility that will not be compromised if the property should change ownership.

In evaluating potential pipeline routes, it is desirable to select direct routes. Data on topography, soil, and geology should be considered. Rough or difficult terrain should be avoided, as should areas that may be susceptible to land- or mudslide, a 100-year flood, or other natural hazards that could cause breakage or outage. Tunneling should be considered only when there are no feasible alternate routes and when it is economically justified. All line installations should give consideration to future construction and the need for repair and maintenance.

When water mains are installed, consideration must be given to providing adequate separation from sanitary sewers. In parallel installations, water mains should be at least 10 feet (3.1 m) horizontally from any sanitary sewer, storm sewer, or sewer manhole.[2] The distance should be measured edge to edge. When conditions prevent a horizontal separation of 10 feet (3.1 m), a water main may be laid closer to a storm or sanitary sewer if the bottom of the water main is at least 18 inches above the top of the sewer. If this vertical separation cannot be obtained, the sewer should be constructed of materials and with joints that are equivalent to water main standards of construction, and should be pressure-tested to assure watertightness prior to backfilling.

In perpendicular crossings when water mains cross house sewers, storm sewers, or sanitary sewers, a separation of at least 18 inches (450 mm) between the bottom of the water main and the top of the sewer should be provided.[2] When local conditions prevent such a vertical separation, sewers passing over or under water mains should be constructed of the materials equivalent to water main standards of construction, and should be pressure-tested to ensure watertightness prior to backfilling. Water mains passing under sewers should, in addition, be protected by a vertical separation of at least 18 inches (0.46 m) between the bottom of the sewer and the top of the water main with adequate structural support for the sewers. This will help prevent excessive deflection of joints and settling of the sewer on the water mains.

A full-length water pipe should be centered at the point of crossing so that the joints will be equidistant and as far as possible from the sewer. No water pipe should pass through or come into contact with any part of a sewer or sewer manhole.

The top of all water pipes should be at least 6 inches (150 mm) below the maximum recorded depth of frost penetration in the area of installation. The minimum depth of water mains should be 5 feet (1.5 m) from the ground surface to the top of the pipe. Pipes always should have adequate cover for external design loads. In designing buried pipelines, simplified tables have been developed based on formulas established by Marston and Spangler, and Young and Smith. Buried pipeline design procedures may be found in several books, handbooks, and standards.[15–20] Pipe classes should be capable of handling the exterior loading resulting from the backfill and surface loading, and should be capable of handling interior working pressures of a minimum of 150 psi (1,000 kPa) and water hammer surge pressures of 100 to 120 psi (690 to 830 kPa). Installation of mains through hazardous areas or at depths greater than 10 feet (3 m) in roadways may also require pressure classes in excess of the above minimum.

Whenever the installation of metallic pipe is contemplated, a soil resistivity survey of the area should be performed. The survey data and calculations should be evaluated together with the history of existing pipes in the area to determine if a nonmetallic pipe should be used. These data will also indicate what level of protection should be provided for metallic fittings and appurtenances. Typically, where resistivities are less than 2,500 ohms-cm, all metallic pipe should be polyethylene-wrapped. If resistivities are less than 1,000 ohms-cm, nonmetallic pipe should be used. The corrosive effects of finished water on the interior of the pipe must also be given consideration.

All piping, joints, and fittings should conform to applicable AWWA specifications and should be rated at least the pressure rating of the straight pipe involved.

All valves should conform to AWWA specifications and, as previously indicated, are typically the same size as the main on mains smaller than 30 inches (750 mm), and may be one size smaller than the main on mains 30 inches (750 mm) and larger.

Water Main Appurtenances

New mains and repaired main sections should be adequately disinfected before being placed in or returned to service. The AWWA ‘‘Standard for Disinfecting Water Mains’’ should be followed. Before new lines and appurtenances are placed in service, the absence of pollution should be demonstrated by bacteriological sampling and examination.

Pressure-reducing valves are provided for maintaining downstream pressure at a uniform pressure less than the upstream main pressure. These valves are usually sized so that the velocity through the valve at maximum hour demand does not exceed 15 ft/sec (4.57 m/s). Cavitation should be considered in valve selection. If a wide range of flows is anticipated, more than one valve may be required. When pressure differentials across the valve are greater than 45 psi (310 kPa), or when downstream pressure will be low relative to the differential pressure, special valving materials should be considered. The *Valve Handbook* by Philip Skousen, 1998, provides extensive details on valve selection criteria and sizing.[21]

System Maintenance

After the water transmission and distribution system has been designed and installed, it is necessary to maintain the system to continue optimal performance. Proper maintenance will include repair of breaks and damaged components, and the routine operation and maintenance of system pumps, valves, and appurtenances. Maintenance of optimal transmission and distribution capacities may require a scheduled pipe-cleaning program. Maintenance of water quality in distribution system requires scheduled flushing of mains. Pipe cleaning may be accomplished by electromechanical methods, pigging, or high-pressure water jetting. Careful selection of the cleaning method and equipment is an essential step if pipes are to be cleaned efficiently. Many water departments have been successful in maintaining a Hazen-Williams friction coefficient in the 120 to 140 range by planned cleaning of pipelines. Other aspects of good distribution system care include leakage testing and control as well as routine evaluation of system performance.

REFERENCES

1. Solley, W. B., Pierce, R. R., and Perlman, H. A., ‘‘Estimated Use of Water in the United States in 1990,’’ *U. S. Geological Survey Circular 1081,* 1993.
2. Report of the Committee of the Great Lakes-Upper Mississippi River Board of State Public Health and Environmental Managers, *Recommended Standards for Water Works,* 1997 edition, Health Education Services, Albany, NY, 1997.
3. Vennard, John K., *Elementary Fluid Mechanics,* 3rd ed., John Wiley & Sons, New York, 1959.
4. Daugherty, Robert L., and Franzini, Joseph B., *Fluid Mechanics with Engineering Applications,* 6th ed., McGraw-Hill Book Co., New York, 1965.
5. Williams, Gardner S., and Hazen, Allen M., *Hydraulic Tables,* 3rd ed. rev., John Wiley & Sons, New York, 1960.
6. Fair, Gordon M., and Geyer, John C., *Elements of Water Supply and Waste-Water Disposal,* John Wiley & Sons, New York, 1958.
7. Cesario, Lee (Ed.), *Modeling, Analysis, and Design of Water Distribution Systems,* American Water Works Association, Denver, CO, 1995.
8. Howland, W. E., ‘‘Expansion of the Freeman Method for the Solution of Pipe Flow Problems,’’ *J. New England Water Works Assoc.* 48:408, 1934.
9. Camp, T. R., and Hazen, H. L., ‘‘Hydraulic Analysis of Water Distribution Systems by Means of an Electric Network Analyzer,’’ *J. New England Water Works Assoc.* 48:383, 1934.
10. McIlroy, M. S., ‘‘Direct-Reading Electric Analyzer for Pipeline Networks,’’ *JAWWA* 42:347, 1950.
11. Jeppson, Roland W., Analysis of Flow in Pipe Networks, Ann Arbor Science Publishers, Ann Arbor, Mich., 1983.
12. Wood, Don J., and Charles, Carl O. A., ‘‘Hydraulic Network Analysis Using Linear Theory,’’ *J. Hydraulics Div. ASCE* 98(HY7):1157–1170, July 1972.
13. Report of the Task Committee on Engineering Practice in the Design of Pipelines, Committee on Pipeline Planning-Pipeline Division, *Pipeline Design for Water and Wastewater,* American Society of Civil Engineers, New York, N.Y., 1975.
14. Walski, Thomas M., ‘‘Why Calibrate Water Distribution System Models?,’’ *Water Engineering & Management,* p. 27, October 1983.

15. Mays, Larry (Ed.), *Hydraulic Design Handbook*, McGraw-Hill, New York, 1999.
16. *Handbook of Valves, Piping and Pipelines,* Gulf Publishing Company, Houston, Tex., 1982.
17. *The Uni-Bell Plastic Pipe Association Handbook of PVC Pipe-Design and Construction,* Uni-Bell Plastic Pipe Association, Dallas, Tex., 1982.
18. *Handbook-Ductile Iron Pipe, Cast Iron Pipe*, 5th ed., Cast Iron Pipe Research Association, Oak Brook, Ill., 1978.
19. *American National Standard for the Thickness Design of Ductile-Iron Pipe,* ANSI/AWWA C150/A21.50-81, American Water Works Association, Denver, Colo.
20. *Concrete Pipe Handbook,* American Concrete Pipe Association, Vienna, Va., 1980.
21. Skousen, P. J., *Valve Handbook,* McGraw-Hill, New York, 1998.

SUPPLEMENTARY READING

American Water Works Standard for Underground Installation of Polyvinyl Chloride (PVC) Pressure Pipe and Fittings for Water, American Water Works Association, Denver, CO, 1995.

Sanks, R. L. (Ed.), et al., *Pumping Station Design*, 2nd ed., Butterworth-Heinemann, Boston, MA, 1998.

CHAPTER 29

Pumping Systems

INTRODUCTION

This chapter addresses a few basic applications for pumping systems commonly used in the waterworks industry. Application of pumping systems requires an understanding of both the pumping unit and how the pumping unit functions within the system. It is necessary to determine how the inlet conditions will affect the performance of the pump. Surge, or water hammer, is addressed in Chapter 26, ‘‘Plant Hydraulics.’’

Figure 29–1 shows pump classifications.[1] Each class finds application in various industries. Positive-displacement pumps are used for low-volume applications where precise delivery is required, such as chemical feed applications, and where high-viscosity fluid pumping is required. Certain classes of rotary positive-displacement pumps are invaluable in fluid power transmission. However, the centrifugal pump is the workhorse of the waterworks industry.

In a centrifugal pump, the rotation of the impeller draws liquid from the suction side into the pump and converts the rotational momentum into pressure energy by means of the volute or diffusion vanes. Figure 29–2 shows the two types of casings used in centrifugal pumps.

SPECIFIC SPEED

Impellers in centrifugal pumps are classified as radial flow, propeller or axial flow, and mixed flow, as shown in Figure 29–3. Figure 29–3 also shows the relationship between the impeller type and specific speed. Specific speed is the speed (in revolutions per minute) at which a given impeller would operate if reduced proportionately in size so as to deliver a capacity of 1 gpm (0.063 L/s) against a total head of 1 ft (0.31 m).[2]

Specific speed is defined as:

$$N_s = \frac{\mathrm{rpm}\sqrt{Q}}{H^{3/4}} \qquad \text{(U.S customary units)}$$

To calculate N_s in terms of foot-pound units when the flow and head are expressed in metric units:

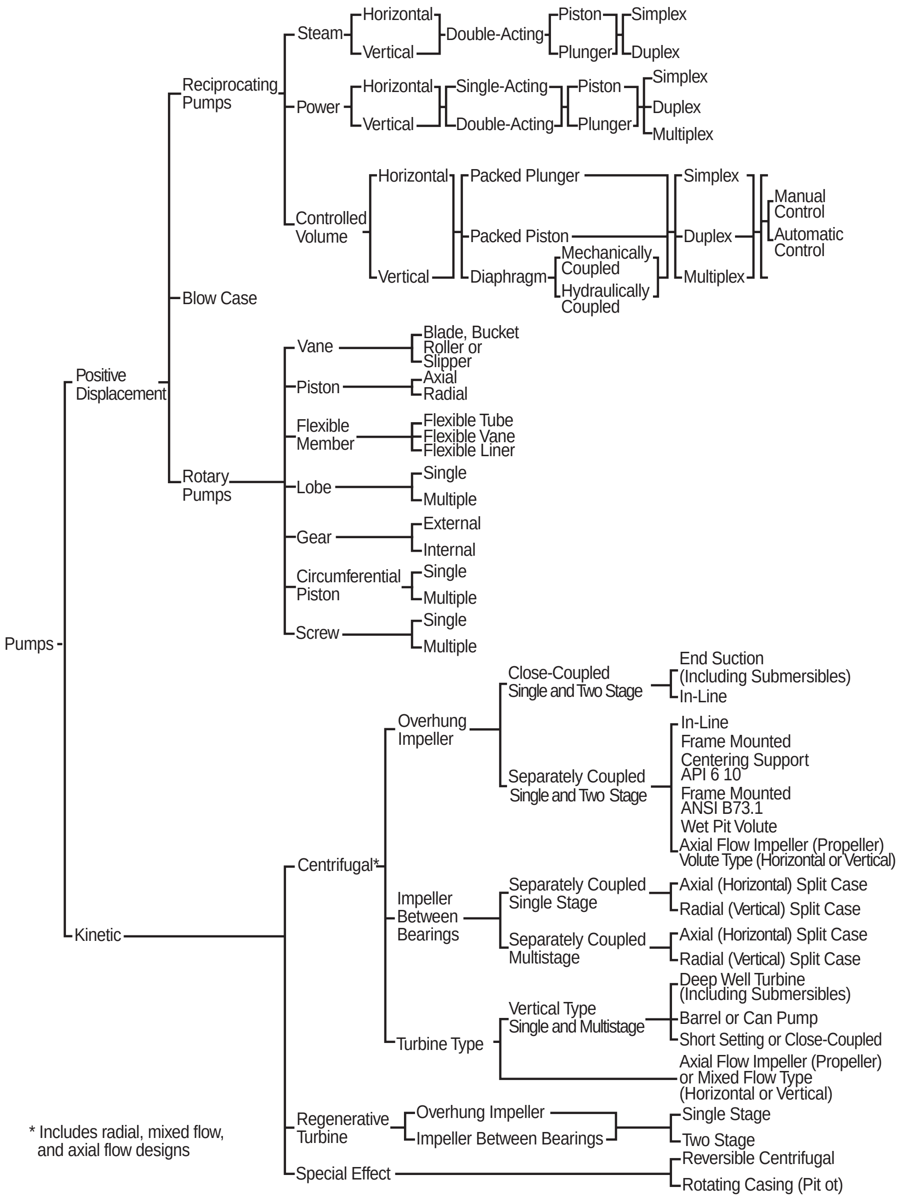
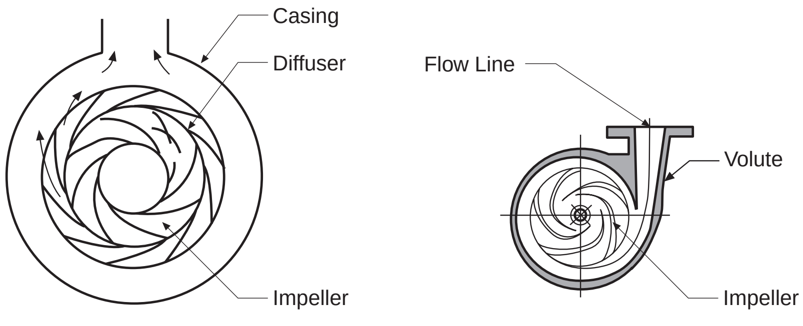


Fig. 29–1. Pump classifications (Adapted from data from the Hydraulic Institute)



Typical Diffuser-Type Pump

Section Through Impeller and Volute

Fig. 29–2. Types of casings

$$N_s = 1.63\,\frac{\text{rpm}\sqrt{Q}}{H^{3/4}} \qquad \text{(metric units)} \qquad (29\text{–}1)$$

where:

N_s = specific speed
Q = flow, gpm (L/s)
H = pumping pressure, a point of maximum efficiency, ft (m)

Specific speed is calculated at the point of peak efficiency for the maximum impeller diameter. For multistage pumps, the head per stage is used. For double suction pumps, one-half of the total flow is used to calculate specific speed. It is an index number that assists the user in assessing the maximum efficiency, as shown in Figure 29–4. Specific speed has also found use in relating pump operating conditions to the potential for cavitation. The Hydraulic Institute standards set upper limits for specific

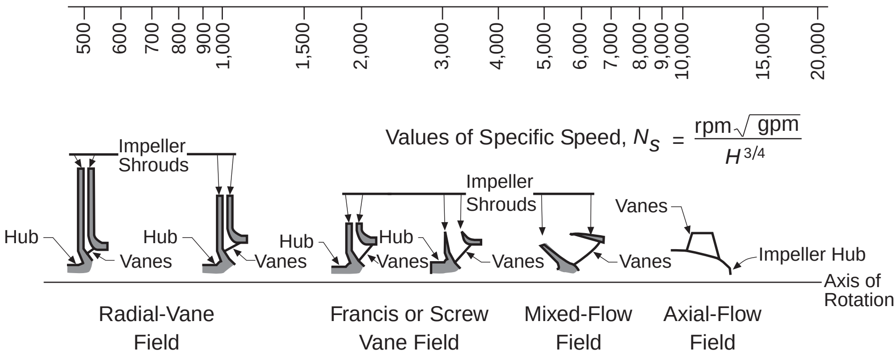


Fig. 29–3. Impellers in centrifugal pumps

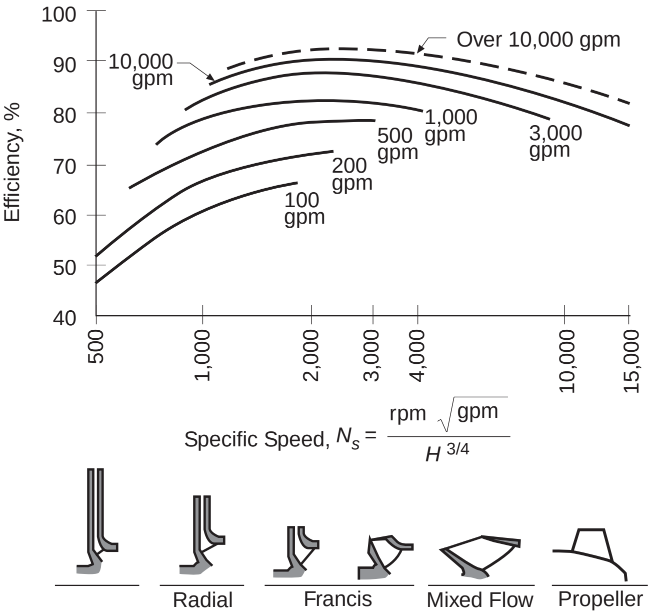


Fig. 29–4. Index number assists user in assessing maximum efficiency

speed of pumps to minimize the potential for cavitation, as shown in Figures 29–5 and 29–6 for single-suction pumps and double-suction pumps, respectively.

AFFINITY LAWS

The characteristics of centrifugal pump operation are shown in Figure 29–7. This representation shows geometrically similar pumps at a constant speed; only the pump impeller diameter varies. Most manufacturers furnish information on their pumps in a similar manner. Another common way to present information on pumps is shown in Figure 29–8, where the physical characteristics are the same but the impeller speed is varied.

In both presentations, the mathematical relationships between the family of curves are known as the affinity laws. These laws are primarily applicable to radial flow centrifugal pumps; although the affinity laws are not accurate for mixed flow or radial flow centrifugal pumps, they provide an estimate of the results of changing pump speed or impeller diameter. The affinity laws are expressed as follows:

$$\begin{array}{ll} \underline{\text{For Constant-Diameter Impeller}} & \underline{\text{For Constant Impeller Speed}} \\ Q_1/Q_2 = N_1/N_2 & Q_1/Q_2 = D_1/D_2 \\ H_1/H_2 = (N_1/N_2)^2 & H_1/H_2 = (D_1/D_2)^2 \\ \text{BHP}_1/\text{BHP}_2 = (N_1/N_2)^3 & \text{BHP}_1/\text{BHP}_2 = (D_1/D_2)^3 \\ E_1 = E_2 & E_1 = E_2 \end{array} \tag{29–2}$$

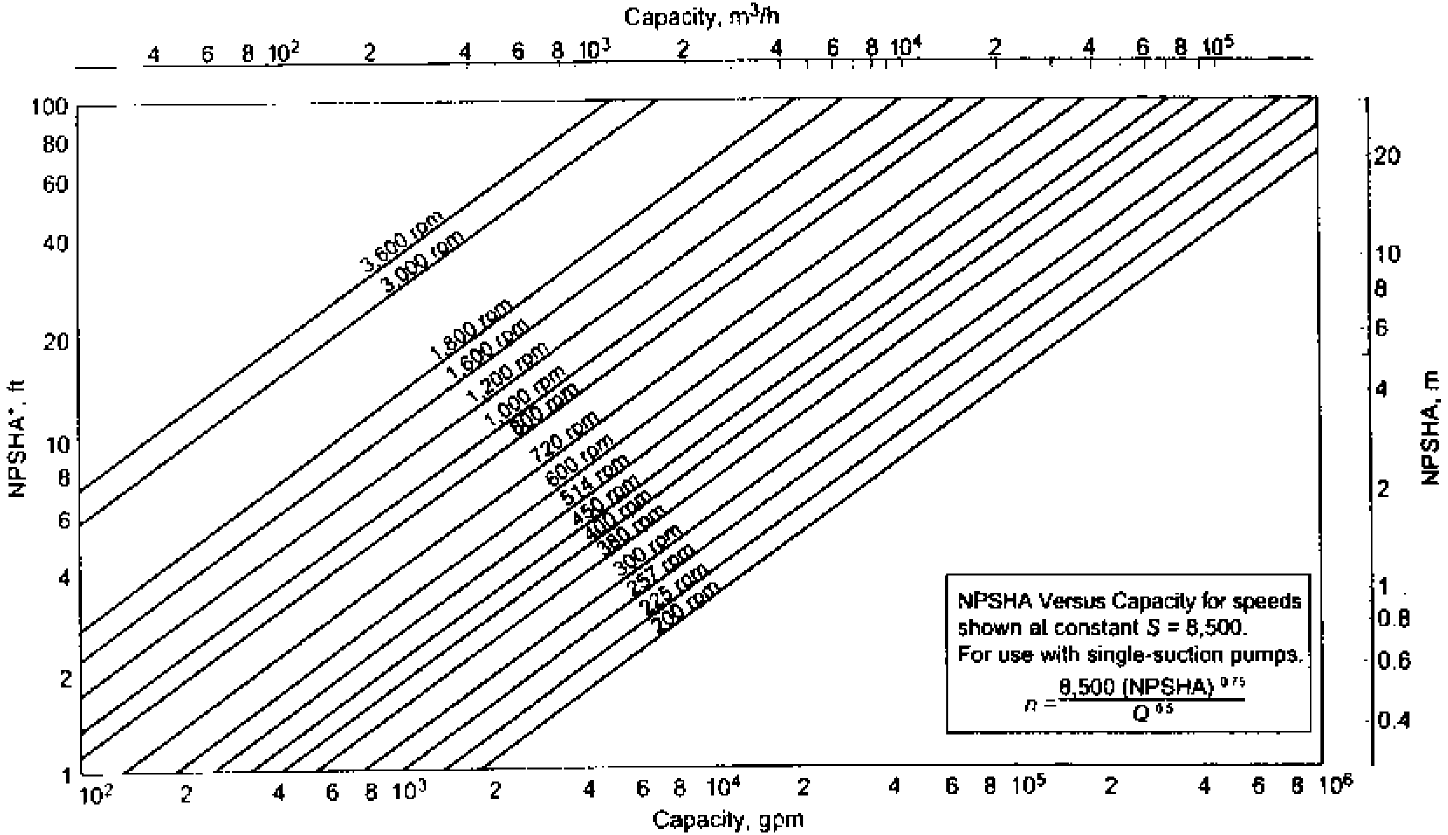


Fig. 29–5. Cavitation single-suction pump[4] (Courtesy of Hydraulic Institute)

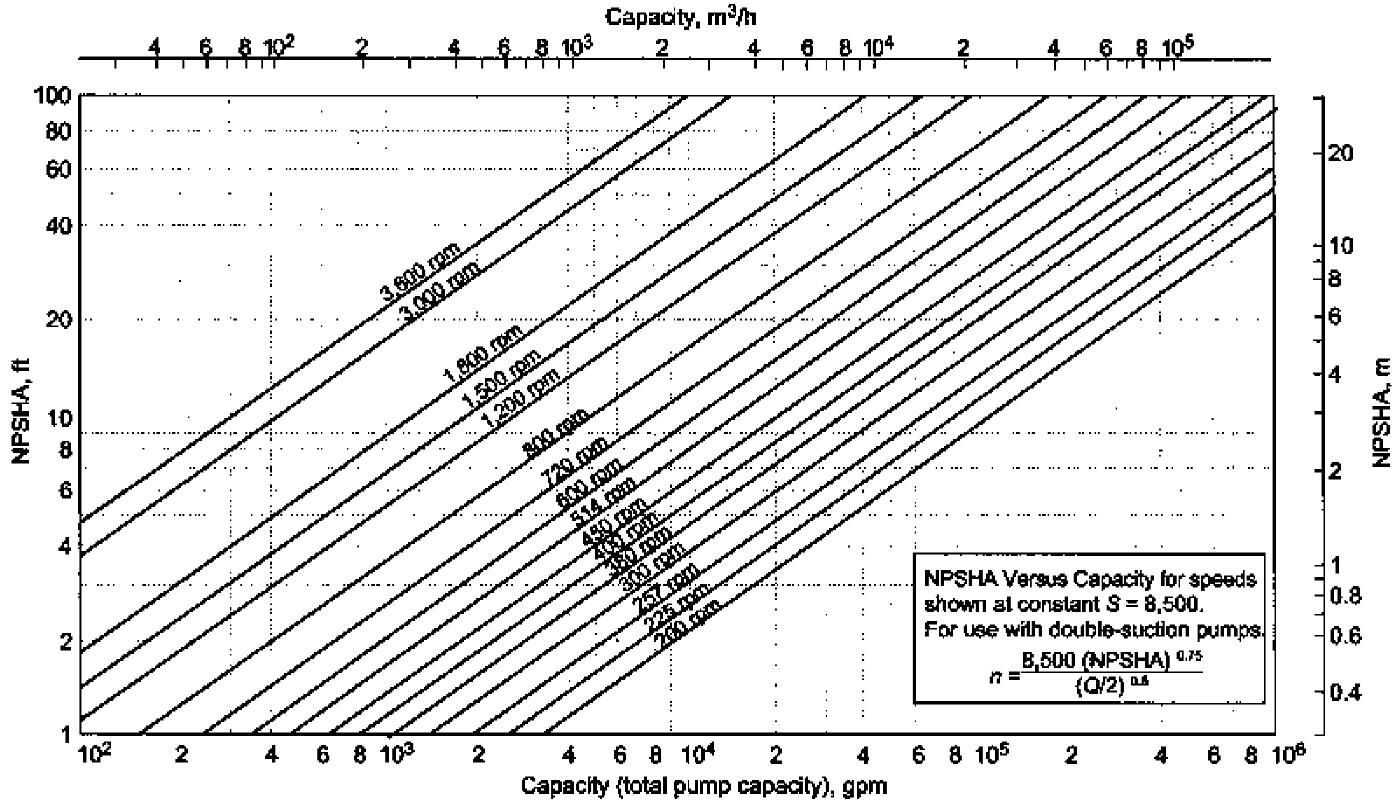


Fig. 29–6. Cavitation double-suction pump[4] (Courtesy of Hydraulic Institute)

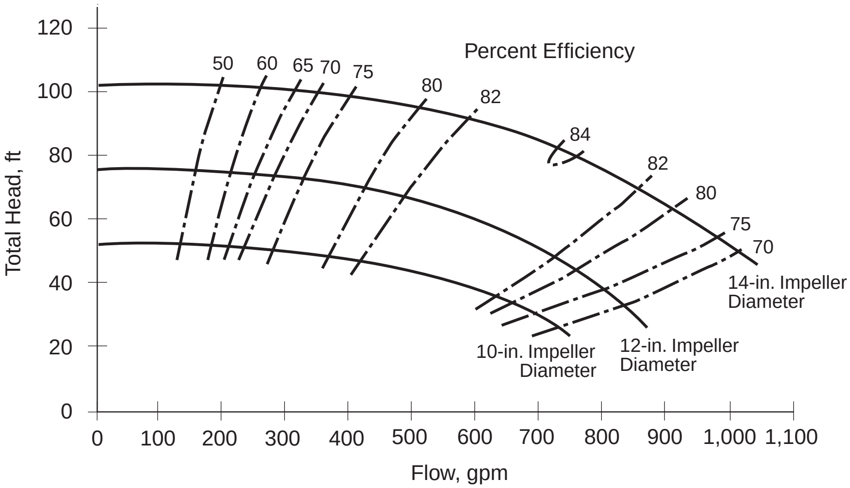


Fig. 29–7. Centrifugal pump curves for various impeller diameters

where:

Q_1, Q_2 = capacity at condition 1 and condition 2
H_1, H_2 = head at condition 1 and condition 2
D_1, D_2 = impeller diameter at condition 1 and condition 2
N_1, N_2 = impeller rpm at condition 1 and condition 2
E_1, E_2 = pump efficiency at condition 1 and condition 2
BHP_1, BHP_2 = power ("brake horsepower" in U.S. customary terms) at condition 1 and condition 2

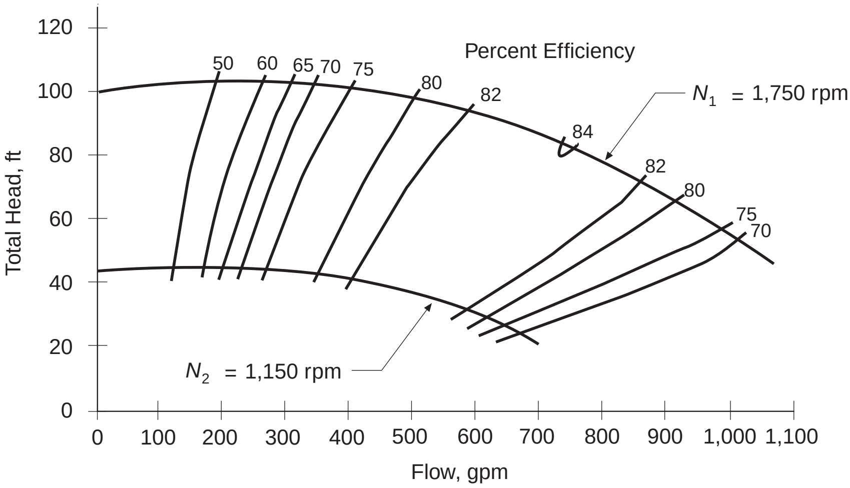


Fig. 29–8. Centrifugal pump curves for various impeller speeds

TABLE 29–1. Application of Affinity Laws in a Case of Constant Impeller Diameter and a Decrease in Impeller Rotation Rate

Q (gpm)	H_1 (ft)	BHP_1 (hp)	E_1 (%)	Q_2 (gpm)	H_2 (ft)	BHP_2 (hp)	E_2 (%)
0	100	7.5	—	0	43	2.1	—
200	102	9.9	52	130	44	2.8	52
400	100	13.5	75	260	43	3.8	75
600	92	17.0	82	400	40	4.8	82
800	75	18.3	83	530	32	5.1	83
1,000	52	18.8	70	660	22	5.3	70

(From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems.* Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

Note: In Fig. 29–8, $N_1 = 1{,}750$ rpm and $N_2 = 1{,}150$ rpm. Therefore, in this case, $N_1/N_2 = 0.66$; $(N_1/N_2)^2 = 0.43$; and $(N_1/N_2)^3 = 0.28$.

An example of the application of the affinity laws showing a reduction in rpm is given in Table 29–1 and in Figure 29–8. As the rpm is reduced, the flow and head are correspondingly reduced. If the head/flow/efficiency curve is known for one pump speed, then the affinity laws can be used to determine the curve for another pump speed.

Centrifugal pumps produce variable flows as total pumping pressure changes, as shown in Figure 29–8. Therefore, with centrifugal pumps it is incomplete to indicate a particular pump capacity unless the pumping head is also described. The pump curve shown in Figure 29–8 also illustrates that the efficiency of pumping is variable for different head or pressure conditions. This particular pump curve indicates that the power requirement increases with increasing flow capacity. Other centrifugal pumps, however, may require greater power at lower pumping capacities because of the characteristics of the pump.

NET POSITIVE SUCTION HEAD

Each liquid has distinctive vapor pressure, which varies with temperature. For example, water has a vapor pressure of 0.6 ft (0.18 m) at 60°F (16°C). In pump and piping systems, it is typical for the lowest pressure to occur at the pump impeller inlet. Should the pressure at this point be below the vapor pressure of the liquid, the water will boil and cavities will form. The cavities will collapse rapidly as they move into higher-pressure regions in the pump. The collapse of these cavities, termed *cavitation,* is accompanied by noise and vibration. Often the collapse of the cavities occurs against the impeller or pump casing and causes erosion of the pump itself.

In order to design a pumping system in which the water will flow to the pump and operate without cavitation, it is necessary to make certain that there will be a positive suction pressure at the pump inlet. Also, as mentioned previously, the specific speed of the pump must be limited to avoid cavitation. The available net positive suction head (NPSH) at the pump inlet is defined as follows:

$$\begin{aligned} \text{NPSH} &= \frac{144}{W}(P_a - P_{\text{vp}}) + h_s \qquad \text{(U.S. customary units)} \\ &= \frac{102}{W}(P_a - P_{\text{vp}}) + h_s \qquad \text{(metric units)} \end{aligned} \tag{29–3}$$

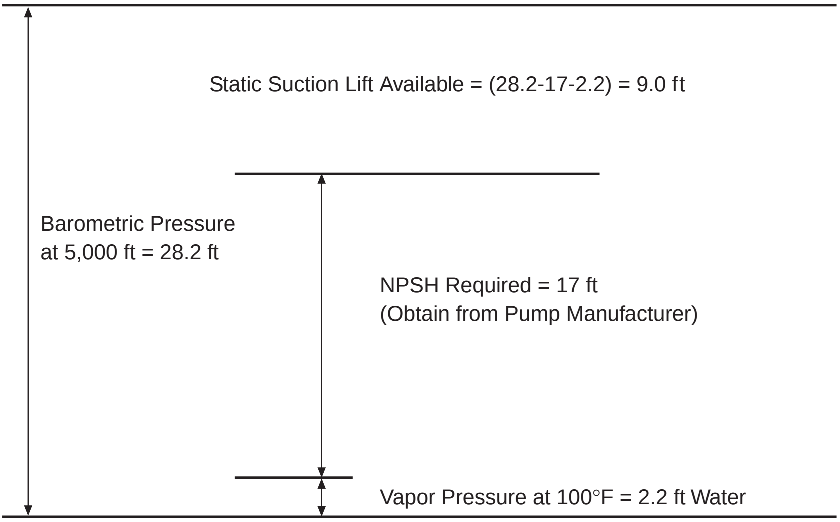


Fig. 29–9. Calculation of the NPSH

where:

NPSH = available net positive suction head, ft (m)
P_a = atmospheric pressure, psia (kPa)
P_{vp} = liquid vapor pressure, psia (kPa)
W = specific weight of liquid, lb/ft^3 (kg/m^3)
h_s = static head on pump suction, ft (m)

The "available" net positive suction head is a calculated value, dependent only on the pump installation, and will vary for each site. The "required" NPSH must be determined through a test by the pump manufacturer. The available NPSH at the installation must be equal to or greater than the required NPSH for a proper installation. Figure 29–9 illustrates calculation of the NPSH.

PUMPING SYSTEM CURVE

In order to define the conditions to which the pump will be applied, the following information is required:

- Suction reservoir water level and level variation
- Suction piping friction losses
- Discharge reservoir water level and level variation
- Discharge piping friction losses
- Desired water delivery rate

For example, consider the simple pumping system shown in Figure 29–10. The minimum difference in water levels from the suction side to the discharge side is 30

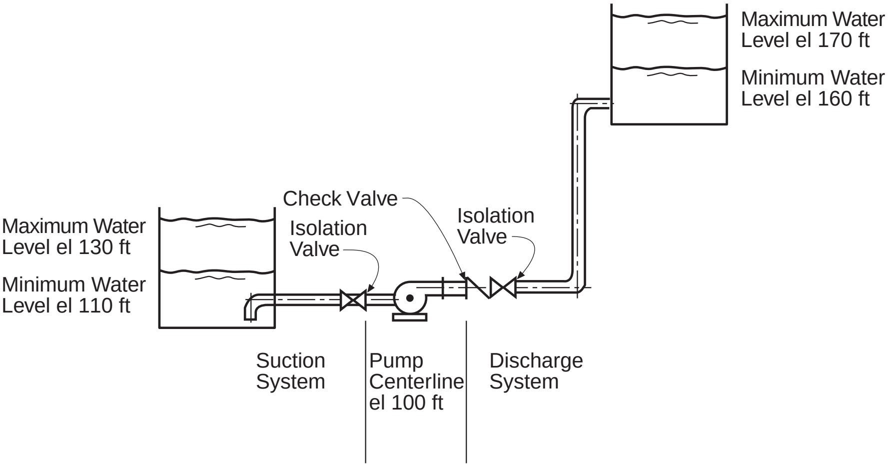


Fig. 29–10. Single pumping curve system

feet (160 – 130 = 30 ft; 9.15 m); the maximum is 60 feet (170–110 = 60 ft; 18.3 m). The friction losses in the suction and discharge piping add to the total dynamic head (TDH) that pumping must impart to the water. Table 29–2 shows how these losses can be determined in calculating TDH. These types of calculations are reduced to a diagram, called a *system curve*, that relates the variation in flow and system conditions to a discharge pressure. More detail on pumping systems can be found in *Pump Handbook* by Karassic and his coworkers.[3]

The prudent designer will make calculations based on both the initial and future anticipated friction losses within the piping system and will consider the extreme minimum and maximum water level conditions at both the suction and discharge reservoirs. Knowledge of these variations is important to ensure that the pump will be stable under all operating conditions.

The system curve (Fig. 29–11) is constructed from the calculations. The capacity requirement for a specific operating condition is determined in order to select a pumping unit. Pump curves are provided by pump manufacturers.

There are several computer programs that can be used to calculate and plot system conditions. As is the case with any software, the designer should verify that the data produced by new software are accurate prior to using the program.

Using a spreadsheet program, the designer can develop the type of pump system data shown in Table 29–2. Instead of using *K* values for fittings, some designers may choose equivalent lengths of straight pipe. For instance, Table 29–3 expresses the pump system in terms of such equivalent lengths.

Many pump manufacturers have their pump curves available on CD-ROM. This allows the designer to review several curves from several manufacturers and select the curve that is best suited to the particular application. The engineer selects the pump or alternative pumps that most efficiently meet the operating conditions and that are stable throughout the conditions that will be experienced. One such selection (i.e., one pump curve) is shown in Figure 29–11. It is apparent that the desired flow rate will be achieved at only one operating condition. The flow rate at other operating conditions should be reviewed to assure that they will be acceptable.

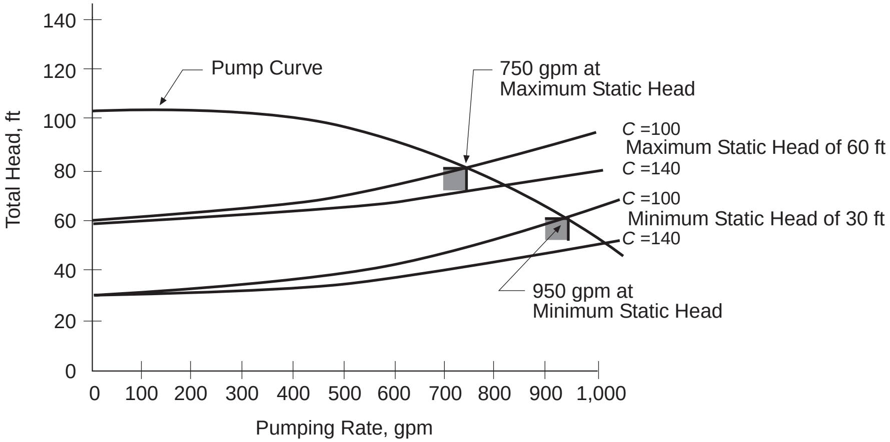


Fig. 29–11. System curves for various conditions

When pumps are placed in parallel the calculations for the system curve are similar; however, the application of the pump curves to the system curve is modified. Figure 29–12 shows an example of a system using parallel pumps. For this example, the system curve would be similar to the one presented in Figure 29–11. The pumping curve provided by the manufacturer would be modified to reflect the connecting piping between the common suction and discharge piping associated with each individual pump. These calculations, based on the same fitting loss equation in Table 29–2, are shown in Table 29–4. In order to graphically show the pumping rate of both pumps operating in parallel, the pumping flow rates for each pump at an equal head are added. Figure 29–13 shows the pumping rate for equal-size, parallel pumps. The static conditions for the parallel pump system are the same as for the single-pump system. For applications with more than two pumps, the modified pumping rate for each pump at an equal head is additive.

In some cases it is necessary to operate pumping units in series, as shown schematically in Figure 29–14. In this case, the pumping heads at equal flow rates are added. Figure 29–15 graphically shows example pump and system curves for such a system.

POWER

The power imparted to the liquid by the pump is known as pump output power (also known as water horsepower):

$$\begin{aligned}\text{pump output power} &= \frac{Q \times 8.34 \times H \times \text{sg}}{33{,}000} = \frac{Q \times H \times \text{sg}}{3{,}960} \quad \text{(U.S. customary units)}\\ &= \frac{Q \times H \times \text{sg}}{366} \quad \text{(metric units)}\end{aligned} \tag{29–4}$$

TABLE 29–2. Pumping System Calculations for the System in Fig. 29–10

$C = 100$

Flow rate (gpm)	Suction System Losses (ft)		Discharge System Losses (ft)		Total Pumping System Losses (ft)	TDH (ft)	
	Fittings	Piping	Fittings	Piping		Min.	Max.
0	0	0	0	0	0	30	60
200	0.01	0.05	0.10	1.5	1.7	31.7	61.7
400	0.06	0.18	0.44	5.6	6.3	36.3	66.3
600	0.13	0.39	1.01	11.4	12.9	42.9	72.9
800	0.25	0.66	1.78	19.5	22.2	52.2	82.2
1,000	0.39	1.00	2.80	29.6	33.8	63.8	93.8

$C = 140$

Flow rate (gpm)	Suction System Losses (ft)		Discharge System Losses (ft)		Total Pumping System Losses (ft)	TDH (ft)	
	Fittings	Piping	Fittings	Piping		Min.	Max.
0	0	0	0	0	0	30	60
200	0.01	0.03	0.10	0.8	0.9	30.9	60.9
400	0.06	0.10	0.44	2.9	3.5	33.5	63.5
600	0.13	0.21	1.01	6.1	7.5	37.5	67.5
800	0.25	0.35	1.78	10.4	12.8	42.8	72.8
1,000	0.39	0.54	2.80	15.9	19.6	49.6	79.6

Pumping System Description (see also Fig. 29–10)

One 10-in. elbow	0.3	One 8-in. check valve	2.5
One 10-in. gate valve	0.3	One 8-in. gate valve	0.3
One 10-in. entrance	0.5	Two 8-in. elbows	0.6
100 ft of 10-in. pipe		One 8-in. exit	1.0
	$\Sigma K = 1.1$	1,000 ft of 8-in. pipe	
			$\Sigma K = 4.4$

(From Culp, Gordon, and Williams, Robert, *Handbook of Public Water Systems*. Copyright © 1986 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

For fitting losses: $H = Kv2/2g$

For piping losses: $H/L = [Q/(0.43Cd2.63)]1.85$, where

H = head loss, ft (m)
K = fitting loss factor, dimensionless
v = velocity, ft/sec (m/s)
L = pipe length, ft (m)
Q = flow, ft^3/sec (m^3/s)
C = Hazen-Williams friction factor
d = pipe diameter, ft (m)
g = gravitational constant, 32.2 ft/sec^2 (9.8 m/s^2)

TABLE 29–3. Pumping System from Table 29–2 Expressed in Equivalent Lengths of Straight Pipe

Fitting	Equivalent Length of Straight Pipe (ft)
Suction system	
One 10-in. elbow	12
One 10-in. gate valve	2.8
One 10-in. entrance	24
100 ft of 10-in. pipe	100
Total (10-in. pipe)	139
Discharge system	
One 8-in. check valve	74
One 8-in. gate valve	3
Two 8-in. elbows	20
1,000 ft of 8-in. pipe	1,000
Total (8-in. pipe)	1,097

where:

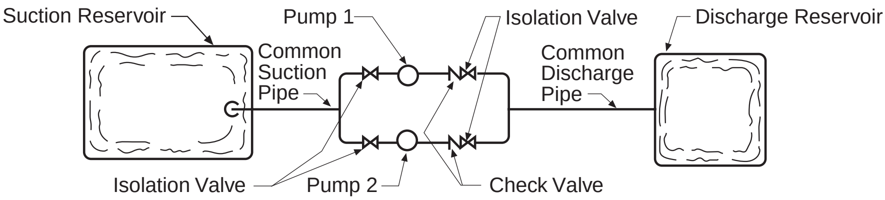
Q = pump flow rate, gpm (l/d)
H = pumping head, ft (m)
sg = specific gravity of liquid being pumped

The *brake* horsepower required to drive the pump is the pump output power divided by the pump efficiency:

$$\begin{aligned} \text{BHP} &= \frac{Q \times H \times \text{sg}}{3{,}960 \times E_p} \qquad \text{(U.S. customary units)} \\ &= \frac{Q \times H \times \text{sg}}{366 \times E_p} \qquad \text{(metric units)} \end{aligned} \tag{29–5}$$

where:

BHP = brake horsepower
E_p = pump efficiency expressed as a decimal fraction



Note: The elevations for this system are the same as shown in Figure 29–10.

Fig. 29–12. Parallel pumps

TABLE 29–4. Calculations for Modifying the Pumping Curve When Parallel Pumping System Is in Place

Pump Curve Information		Suction		Discharge		Total Additional Headloss (ft)	Revised Pump Head (ft)
Q (gpm)	Head (ft)	$v^2/2g$	Additional Headloss (ft)	$v^2/2g$	Additional Headloss (ft)		
0	100	0	0	0	0	0	100
200	102	0.01	0.01	0.03	0.09	0.1	102
400	100	0.04	0.04	0.11	0.38	0.4	100
600	92	0.09	0.10	0.24	0.85	1.0	91
800	75	0.16	0.18	0.42	1.51	1.7	73
1,000	52	0.25	0.28	0.66	2.37	2.6	49

	Fitting Loss Factor K	
Additional Pipe Fitting	Suction	Discharge
One 10-in. tee	0.5	—
One 10-in. ell	0.3	—
One 10-in. gate valve	0.3	—
One 8-in. check valve	—	2.5
One 8-in. gate valve	—	0.3
One 8-in. ell	—	0.3
One 8-in. tee	—	0.5
	$\Sigma = 1.1$	3.6

Note: Table reflects modification of pump curve to reflect individual pump piping losses.

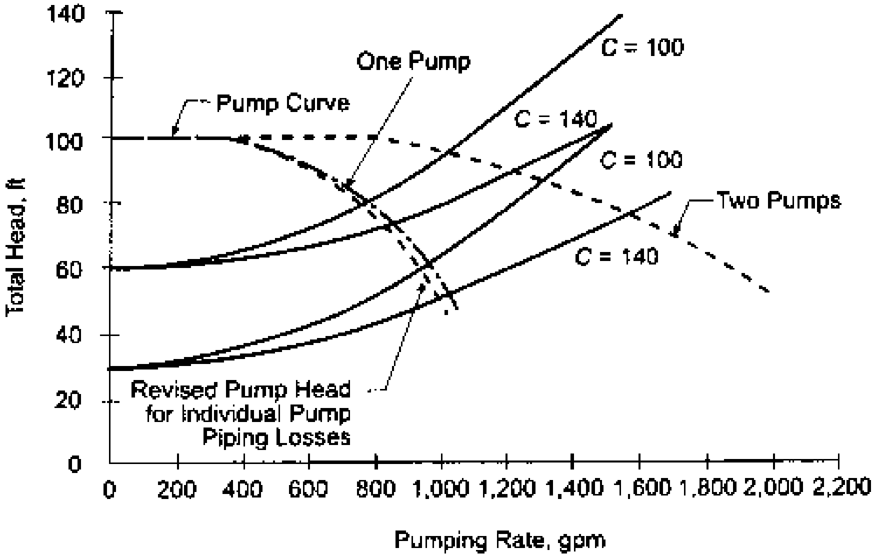


Fig. 29–13. System curves for pumps in parallel

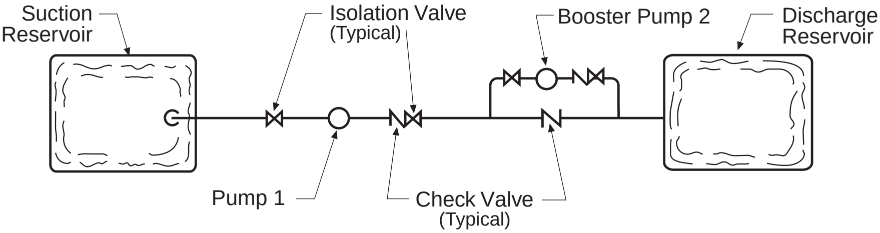


Fig. 29–14. Pumps in series

The electrical use on the electrical power supply to the water is (in U.S. customary units only):

$$\text{electrical use} = \frac{\text{BHP} \times 0.746}{E_m} \tag{29–6}$$

where:

E_m = motor efficiency expressed as a decimal fraction.

In the SI system, the unit of power is the watt (kg · m^2/s^3), so Equations 29–4, 29–5 and 29–6 would not apply.

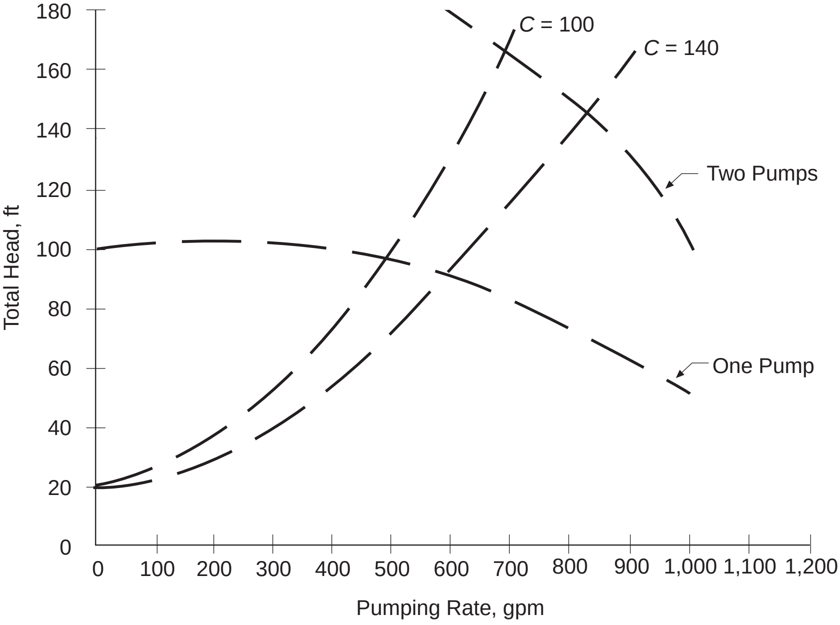


Fig. 29–15. System curve for pumps in series

SUCTION CONDITIONS

Water should be introduced to the pump so the approach velocity is uniform and the liquid is not rotating. If the velocity of approach is uneven, it will place an unbalanced load on pump impeller, and exert more load on bearings and packing than they are designed for. The Hydraulic Institute standards provide guidelines for arrangement of sumps.[4] Some problems have been experienced using these guidelines. The Hydraulic Institute subcommittee on sump design is in the process of revising many of its recommended sump layouts as a result of testing as of early 1996.

Intakes from lakes or rivers frequently use vertical turbine pumps mounted on a platform and suspended in a wet sump. The ideal flow approach to the pump/pumps needs to be uniform and evenly distributed to multiple pumps. Practically, the disturbance at the entrance, the direction changes at the pump suction, and difficulty in avoiding eddies, swirls, and vortexing in the flow make this situation difficult to evaluate. Figure 29–16 shows baffling provided for a wet sump for vertical pumps. The concept is to direct the flow to the pumps with orifices in a barrier wall and guiding baffle walls that keep the flow moving straight to the pump without swirling. The suction cone is to defeat a common vortex that forms under the pump suction.

Figure 29–17 shows the baffling for a can-pump-mounted vertical turbine pump. The concept shown intends to deflect-baffle the flow downward into the can around the pump column. A clever design is necessary to baffle-guide the flow while still permitting easy removal of the pump from the can. Where the flow moves around the pump suction bell, it passes through the grating and then turns and passes through the grating to enter the pump suction.

Manufacturers' standard layouts for vertical end suction centrifugal pumps call for a standard-radius elbow from the pump to the wetwell. During a hydraulic model test in August 1996, however, column separation of the water was observed on the inner radius of the elbow, as well as an unequal velocity distribution around the suction pipe entrance to the pump. The observed phenomenon resulted in

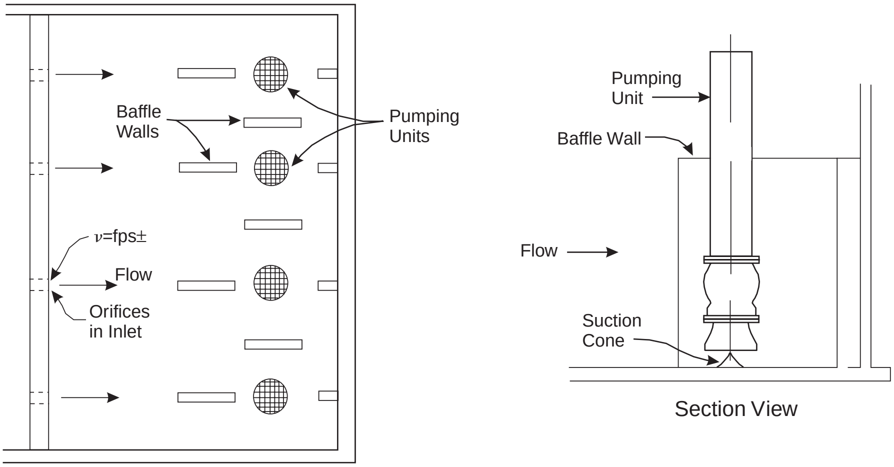
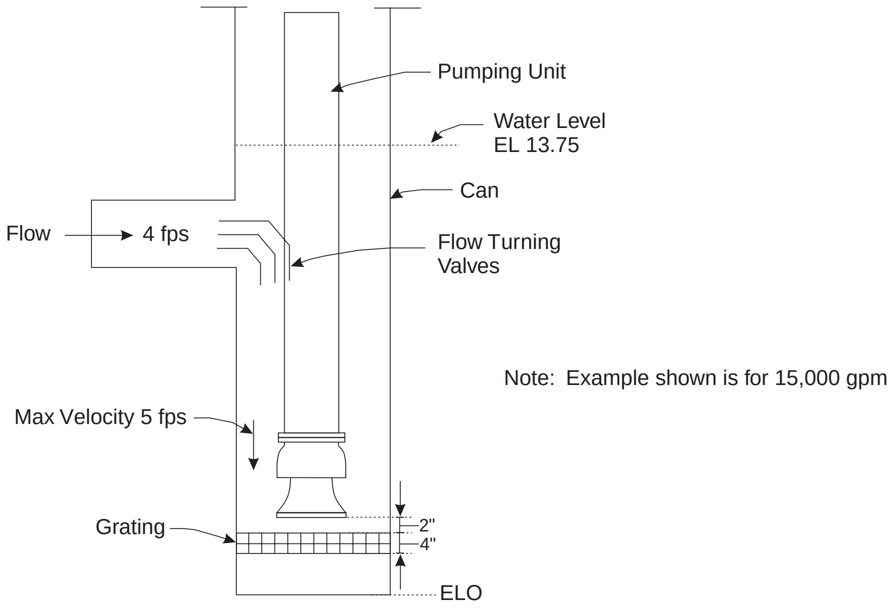


Fig. 29–16. Wet sump vertical turbine pump application



Note: Example shown is for 15,000 gpm

Fig. 29–17. Can pump vertical turbine pump application

- Unbalanced loads to the pump impeller
- Pump bearing failure
- Pump seal failure
- Reduced pump efficiency

Using a long-radius reducing elbow in place of the standard-radius elbow eliminated the column separation and reduced the velocity distribution to acceptable limits. A reducing elbow of two standard pipe diameters, 24 in. to 18 in. (600 mm to 450 mm), was required to maintain the pipeline velocity distribution within reasonable limits.

Sump and suction piping design is an art as well as science. Initial design is based on scientific principles, but dynamics within sumps require models to completely refine the design. Hydraulic model testing should be used in large pumping applications.

CENTRIFUGAL PUMPS

Figures 29–18 to 29–22 show a variety of centrifugal pumps.

The nonclog centrifugal pump is often used to pump liquids carrying low concentrations of solids, such as alum or iron salt sludge. The ‘‘nonclog’’ impeller is designed with clearances to pass solids. Heavy sludge, such as lime-softening sludge, unless mixed to keep in suspension, may concentrate too much for reliable application of centrifugal pumps.

The prudent designer arranges centrifugal pumps so the impeller is always below the suction static water level. In circumstances where a submerged impeller is not

Image Not Available

Fig. 29–18. Nonclog pump (Courtesy of Fairbanks Morse)

practical, a ‘‘self-priming’’ pump is used. The most common approach used in the water industry for ‘‘self-priming’’ a centrifugal pump is to provide an integral or separate priming chamber that keeps the impeller in water. When the pump is in operation, a vacuum is pulled in the priming chamber and induces a lower head static water level to flow into the priming chamber.

The horizontal split-case centrifugal pump is one of the most popular pumps in the water industry for clear water applications. The impeller can be removed without

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Fig. 29–19. Self-priming pump (Courtesy of Fairbanks Morse)

Image Not Available

Fig. 29–20. Self-priming pump application with separate priming chamber (Courtesy of Fairbanks Morse)

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Fig. 29–21. Horizontal split case pump (Courtesy of Fairbanks Morse)

disturbing the discharge and suction piping. The pump is easily accessible, highly reliable, and does not have the vibration issues frequently associated with vertical pumps.

Vertical turbine pumps are popular for raw-water pumping at intakes, in-plant applications, high service pumping, and distribution system booster applications. The vertical turbine pump is very versatile. It can be easily adapted to single- or multiple-stage applications. The pump impellers can be radial flow, mixed flow, or axial flow. Vertical pumps are set with the suctions in water sumps, in pipes, or in cans.

Because of the large suspended, rotating load, the balance and hydraulic suction conditions are critical to maintain an acceptable limitation on vibration. Carefully designed pump suction conditions are crucial to any centrifugal pump, but the care is even more important for vertical turbine pumps.

POSITIVE-DISPLACEMENT PUMPS

The most common types of positive displacement used in the water industry include piston-diaphragm metering pumps, progressing-cavity pumps, piston pumps, and dia-

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Fig. 29–22. Vertical turbine pump (Courtesy of Fairbanks Morse) phragm pumps. Figures 29–23 to 29–25 show examples. Positive-capacity pumps are low-capacity pumps and are applicable for high heads.

Metering pumps are primarily used to feed liquid chemicals. They come in single or multiple heads on the same drive. Typical output turndown is 20:1. Installation designs typically include a calibration column on the suction side to permit accurate calibration of the unit and a spray shield to deflect accidental or sudden high-pressure leakage of piping at the pump. Metering pumps use ball check valves on the suction and discharge. As such, a significant and positive discharge pressure is required to avoid gravity or momentum flow through the pump. For low-pressure or downhill application, a discharge pressure maintaining valve is required. One key to the successful application of positive displacement pumps is to avoid running them at high speeds. The use of lower speeds will reduce maintenance and noise. For piston-diaphragm metering pumps, it is suggested that the speed be limited to 80 strokes per minute

Progressing cavity pumps are used to pump sludge or sludge cake and polymer in most transfer, sampling, thickening, or dewatering applications. The installation of the pump should allow space and piping arrangements for easy withdrawal of the stator from the rotor. For progressing-cavity pumps, it is suggested that the speed be limited to 250 rpm.

Some utilities prefer to use piston or air-driven diaphragm pumps for sludge pumping. Similar considerations to the design of these pumps should be applied: use lower speeds and provide for removal and maintenance.

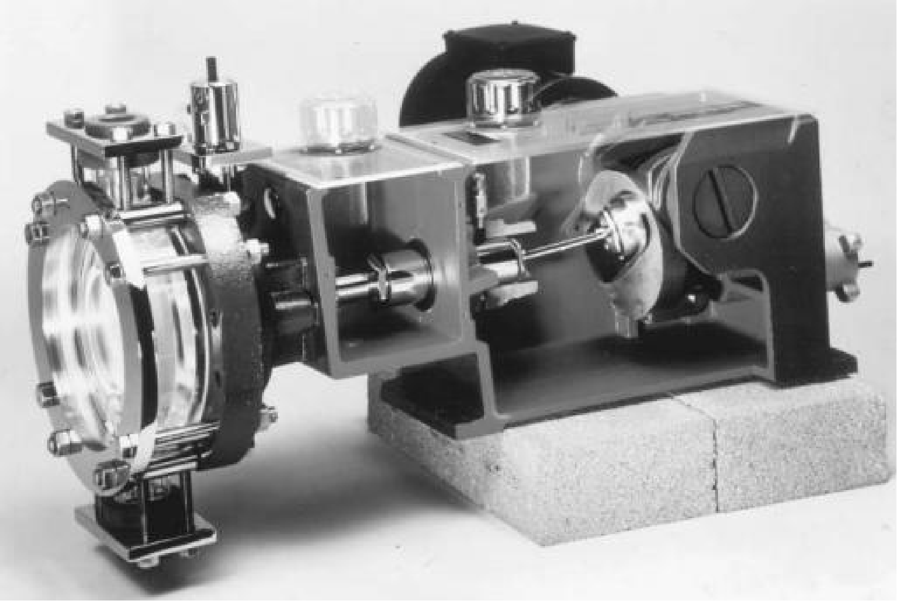

Fig. 29–23. Hydraulically balanced diaphragm metering pump (Courtesy of Pulsafeeder, Inc., a unit of IDEX Corporation)

Fig. 29–24. Progressive cavity pump (Moyno is a registered trademark of Robbins & Myers, Inc.)

Fig. 29–25. Diaphragm pump (Courtesy of Dorr-Oliver)

VARIABLE-SPEED PUMPING

Chapter 30, ‘‘Energy Management for Water Treatment Facilities,’’ describes variable-speed drives. Variable-speed drives for pumping systems can be applied either to centrifugal pumps or positive-displacement pumps, like the progressing cavity pump. The intent of variable-speed pumping is to match the flow rate to the water-demand rate. Oftentimes, the variation in flow that can be attained with multiple pumps provides the necessary flexibility. Where more flexibility is needed, variable-speed drives on pumping systems are used.

Where there are several pumping units, the question arises as to how many pumps need to be provided with variable output. Oftentimes the availability of one pumping unit fitted with a variable-output capability is enough to gain the needed flexibility. In those cases, the criticality of the variable output needs to be assessed to determine if a standby pump or variable drive is necessary.

OTHER ISSUES

Axial flow pumps and higher-specific-speed mixed flow pumps have an unstable pumping condition at higher heads. Unlike radial flow pumps that can be essentially operated throughout the pump curve to shutoff, high-specific-speed pumps have a characteristic hump in their curve whereby there are three flow rates at a specific head (see Fig. 29–26). If the pumping head reaches this zone on the pump curve, the flow will become unstable as it moves from point to point on the pumping curve. On applications for pumping units with this characteristic, it is important to select a pump that matches the system curve to avoid this zone. Also, for pumps with variable-speed drives, limit the speed reduction to avoid operating above the critical zone on the pump curve.

Radial flow pumps can be operated from shutoff throughout the pumping range. Particularly for booster pumping applications, where the outflow may fall to zero, it is necessary to provide a bleed-off on the discharge piping. If the pump is operating at shutoff, the pumping efficiency loss generates heat in the pump, and eventually the water temperature in the pump will reach boiling. The amount of bleed-off water needs to exceed the amount needed to take the heat buildup away. The bleed-off water may be reconnected to the upstream distribution system to avoid the loss of this treated water.

PUMP LAYOUT

In making pump layouts, the designer should keep in mind a number of considerations:

- Provide at least a 3-ft (0.9-m) clearance between obstacles to permit wheeled hand truck movement on the pumping station floor.
- Arrange suction piping to avoid high points where air or gas may collect. Reducers, where they are required in horizontal piping, should be eccentric.
- Discharge piping also should avoid high points. Dissolved gas will have less tendency to come out of solution on the discharge side than on the suction side;

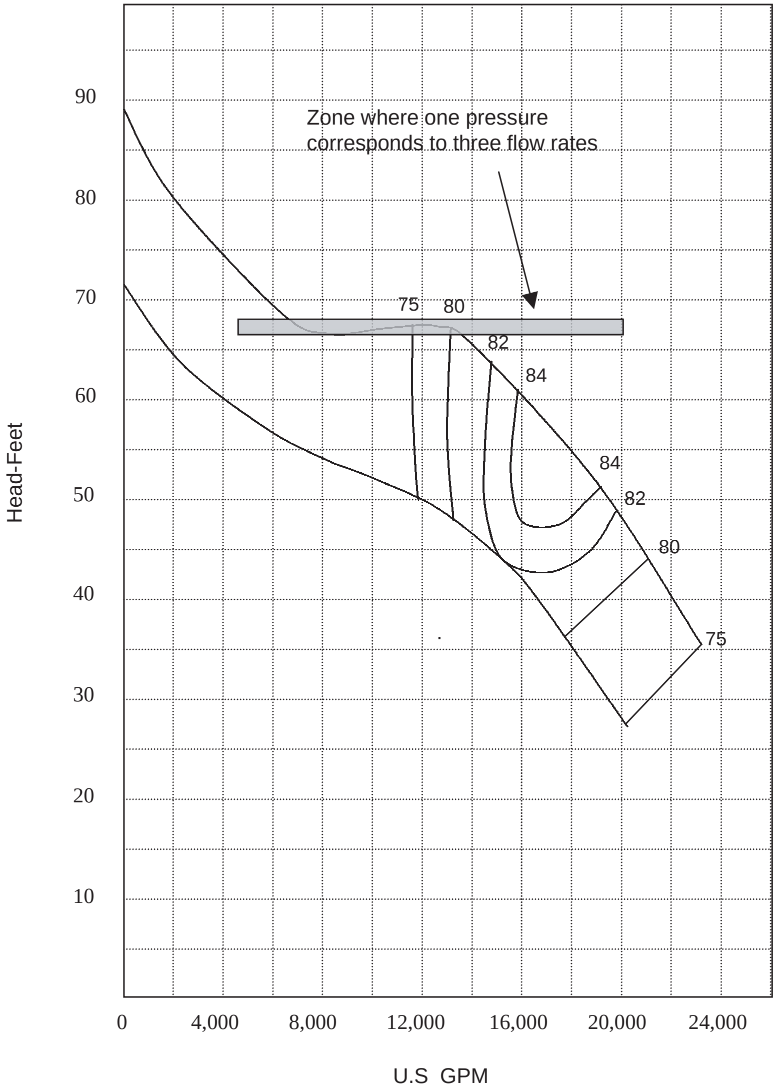

Fig. 29–26. Unstable operation zone found with some high-specific-speed pumps (Courtesy of Fairbanks Morse)

however, if air pockets are allowed to accumulate, they will restrict flow. Provide air relief valves where high points occur. Pipe the air relief to a drain.

- Provide piping supports to keep all weight off the pump. Support all discharge piping rigidly to prevent movement.
- Provide suction couplings on the pump to permit removal and replacement of the pump. The coupling is best located on the suction side, because the suction typically does not experience pressure variations that are common to the discharge.

- Locate the discharge isolation valve downstream from the check valve. The check valve, which operates frequently, usually experiences more trouble and requires more maintenance than the discharge isolation valve. The operators should be able to isolate the check valve from the rest of the system.
- When pumping sludges, locate the check valve in the horizontal. When the pump is idle, solids will settle against the closed check valve. Many times, the pump pressure will force the solids away from the check valve and allow the valve to operate; however, there may be times when solids will lodge on the downstream side of the check valve and prevent it from opening.
- Provide embedded lifting eyes or hoist rails above pumps, as well as a passageway, to allow the pumps to be removed.
- In small service areas where diversification of water demand is small and a pumped system may experience periods of zero flow, provide a small relief bypass to prevent overheating the water in the pump.
- Arrange piping to avoid situations where pipes will be less than 8 ft (2.4 m) above the floor or will obstruct passageways to valve actuators.
- Provide hatch and door openings to remove both pump and motor from the building.

  When using variable-frequency drives (VFDs), consider (1) air-conditioning of the VFD area and (2) noise abatement of pump motors

REFERENCES

1. Hydraulic Institute, 9 Sylvan Way, Parsippany, NJ, 973-267-9700, www.pumps.org.
2. Colt Industries, Fairbanks Morse & Co., Pump Division, *Hydraulic Handbook,* Kansas City, Kan., 1979.
3. Karassic, Igor J., Krutzsch, William C., Fraser, Warren H., and Messina, Joseph P. (eds.), *Pump Handbook,* McGraw-Hill, New York, 1976.
4. Hydraulic Institute, *Hydraulic Institute Standards,* ANSI/HI 1.1–1.5–1994, Hydraulic Institute, Cleveland, Ohio, 1994.

CHAPTER 30

Energy Management for Water Treatment Facilities

INTRODUCTION

In recent years, dramatic increases in energy prices have made the public acutely aware of the significance of energy costs. *Energy,* once a term used by relatively few scientists and engineers, is not only well known to waterworks managers but also a household topic of discussion. In the past, energy was rarely considered in the design and operation of waterworks facilities. Now energy plays an important role in decisions that pertain to the design and operation of water treatment plants, pumping stations, and distribution systems. This chapter examines energy fundamentals, energy optimization, and how these factors relate to capital and operating costs for waterworks facilities. A generalized approach to an energy optimization evaluation is then presented, followed by a discussion of energy conservation equipment and techniques.

ELECTRICAL ENERGY FUNDAMENTALS

Electrical systems are of either the direct current (DC) or alternating current (AC) type. In direct current systems, the voltage remains constant, and current always flows in the same direction. In alternating current systems, voltage and current follow sine wave patterns, reversing direction regularly as shown in Figure 30–1, which represents instantaneous power with coinciding voltage and current waveforms.

Apparent power delivered to an alternating current circuit is calculated by the vector dot product of resistive power and reactive power. This relationship is illustrated in the power triangle shown in Figure 30–2. Apparent power is expressed in units of kilovolt-amperes (kVA); this is the demand placed on the electrical utility’s system by a customer. The resistive load actually performs work and is known as *active power,* which is the power actually delivered to the customer; it is the value measured by a customer’s power meter as kilowatts (kW). Reactive load does not perform work but is necessary to provide energy for changes in magnetic flux. The reactive demand on a circuit is the algebraic sum of capacitive and inductive demands. When these demands are equal, the sum is zero, and the reactive demand is zero. Under this condition, the current and voltage function coincide, and a wave pattern (shown in Fig. 30–1) results, with all power delivered to a circuit available as active power. When circuit inductance is greater than capacitance, current will lag voltage, as shown in Figure

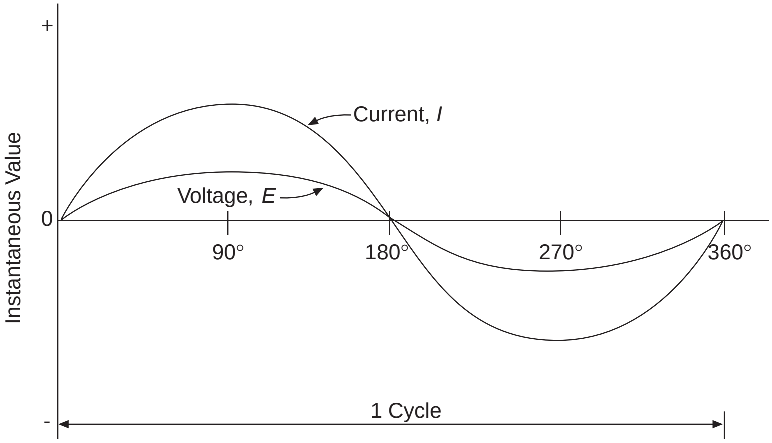

Fig. 30–1. Instantaneous power in an AC circuit with coinciding voltage and current waveforms

30–3. When capacitance is greater than inductance, current will lead voltage. The amount by which the current lags or leads the voltage is expressed as an angle, with one full cycle being 360°. Power factor is the cosine of this angle and is the ratio of the active power to the apparent power. Partially loaded induction motors create the largest reactive demand on most systems, and current tends to lag voltage on most.[1] Useful electrical relationships are shown in Table 30–1.

ENERGY CONSUMPTION IN WATER TREATMENT PLANTS

Energy is a major cost in the waterworks industry. The distribution of energy by process in water treatment plants of various sizes is presented in Table 30–2. Design

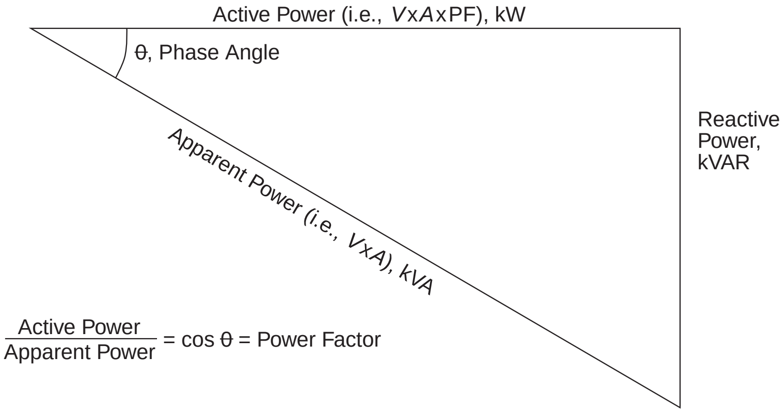

Fig. 30–2. Power triangle

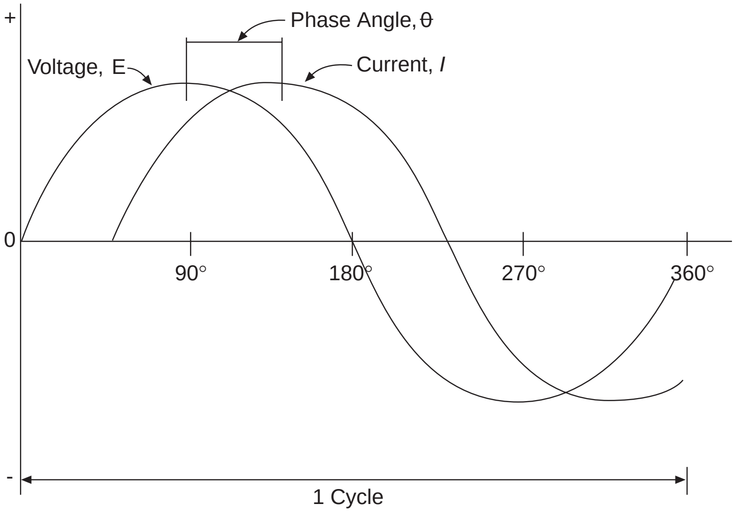

Fig. 30–3. Instantaneous power in an AC circuit with current lagging voltage

criteria for the processes listed in Table 30–2 are listed in Table 30–3. As shown in Table 30–2, finished water pumping is by far the largest energy-consuming process. Data from Table 30–2 are graphically presented in Figure 30–4, which depicts the relative distribution of energy at a 10-mgd (38-ML/d) facility.

ENERGY EFFICIENCY CONSIDERATIONS FOR WATER SYSTEMS: AN OVERVIEW

Energy efficiency must focus on both existing facilities and new construction. The following issues issues related to energy efficiency are discussed in detail in this chapter:

- Energy optimization studies
- Electric motors
- Variable-speed drives
- Electric utility billing schedules
- Energy management systems
- Supervisory control and data acquisition (SCADA) systems
- Optimum conveyance systems
- Pumping considerations
- Lighting
- Water conservation

TABLE 30–1. Useful Electrical Conversion Formulas

		Alternating Current	
To Find	Direct Current	Single Phase	Three Phase
Amperes when horsepower known	$\frac{\text{hp} \times 746}{\text{V} \times e}$	$\frac{\text{hp} \times 746}{\text{V} \times e \times \text{PF}}$	$\frac{\text{hp} \times 746}{\text{V} \times \sqrt{3} \times e \times \text{PF}}$
Amperes when kilowatts known	$\frac{\text{kW} \times 1{,}000}{\text{V}}$	$\frac{\text{kW} \times 1{,}000}{\text{V} \times \text{PF}}$	$\frac{\text{kW} \times 1{,}000}{\text{V} \times \sqrt{3} \times \text{PF}}$
Amperes when kilovolt-amperes known		$\frac{\text{kVA} \times 1{,}000}{\text{V}}$	$\frac{\text{kVA} \times 1{,}000}{\text{V} \times \sqrt{3}}$
Kilowatts	$\frac{\text{A} \times \text{V}}{1{,}000}$	$\frac{\text{A} \times \text{V} \times \text{PF}}{1{,}000}$	$\frac{\text{A} \times \text{V} \times \sqrt{3} \times \text{PF}}{1{,}000}$
Kilovolt-amperes		$\frac{\text{A} \times \text{V}}{1{,}000}$	$\frac{\text{A} \times \text{V} \times \sqrt{3}}{1{,}000}$
Power factor		$\frac{\text{kW} \times 1{,}000}{\text{A} \times \text{V}}$ or $\frac{\text{kW}}{\text{kVA}}$	$\frac{\text{kW} \times 1{,}000}{\text{A} \times \text{V} \times \sqrt{3}}$ or $\frac{\text{kW}}{\text{kVA}}$
Horsepower (output)	$\frac{\text{A} \times \text{V} \times e}{746}$	$\frac{\text{A} \times \text{V} \times e \times \text{PF}}{746}$	$\frac{\text{A} \times \text{V} \times \sqrt{3} \times e \times \text{PF}}{746}$

A = amperes; *e* = efficiency; hp = horsepower; kVA = kilovolt-amperes; kW = kilowatts; PF = power factor; V = volts.

Source: See Reference 2.

Note: When used in this table's formulas, power factor and efficiency should be expressed as decimals.

ENERGY OPTIMIZATION STUDIES

One of the most common approaches to improving energy efficiency at existing facilities is to start with an energy optimization evaluation (also called an energy audit).* Clearly, optimization of energy for water treatment facilities will focus on reducing energy and demand charges associated with raw and finished water pumping. The next several subsections describe steps that a water utility should take as part of conducting this type of study.

Conduct Kickoff Meeting

The agenda for the study's kickoff meeting can include the following:

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*This section draws from Reference 4. This section draws from information found in publications by the Illuminating Engineering Society of North America (IESNA) (see References 5 and 6) and EPRI (see Reference 7).

TABLE 30–2. Distribution of Energy by Process in Water Treatment Plants (5 to 200 mgd)

	Energy at Listed Plant Capacity (kWh/year)				
Process	5 mgd	10 mgd	50 mgd	100 mgd	200 mgd
Raw-water pumping	356,250	712,500	3,562,500	7,125,000	13,500,000
Alum feed	10,250	13,250	33,375	49,875	62,750
Polymer feed	23,250	23,375	23,875	24,375	25,375
Rapid mix	149,875	299,750	21498,125	2,995,000	5,990,000
Flocculation	44,500	89,000	444,625	889,000	1,775,000
Clarification	16,500	21,375	93,125	186,250	372,000
Gravity filtration	103,750	192,500	825,375	1,487,000	2,795,000
Filtration media	0	0	0	0	0
Hydraulic surface wash	9,500	17,750	91,125	181,000	373,625
Backwash pumping	15,750	31,500	151,625	315,250	630,250
In-plant pumping	283,875	551,000	2,677,250	5,322,000	10,104,125
Cl_2 feed	16,250	25,750	94,625	119,750	204,875
Clearwell storage	0	0	0	0	0
Finished water pumping	2,137,500	4,275,000	21,375,000	42,750,000	81,000,000
Admininistration, laboratory, maintenance building	83,000	131,625	394,875	509,250	642,250
Total	3,250,250	6,385,375	31,265,500	61,953,750	117,475,250
Unit consumption (kWh/MG)	1,875	1,841	1,803	1,787	1,693

Source: See Reference 3.

Note: Values apply to hypothetical plants operating at 95 percent capacity; 1 mgd = 3.79 Ml/d. See 30–3 for design criteria.

TABLE 30–3. Treatment Plant Design Criteria for Use with Table 30–2

Process	Remark
Raw–water pumping	TDH = 50 ft
Alum feed	Liquid stock
Rapid mix	*G* = 900 sec–1
Flocculation	Horizontal paddle, *G* = 80
Clarifier	Rectangular FeCl and alum sludge
Gravity filtration structure	HVAC
Filtration media	Mixed media, no energy
Hydraulic surface wash	Rotary arms with pumped source
In-plant pumping	Pump to clear storage
Cl_2 feed	Gas, 1-ton cylinders
Clearwell storage	No energy
Finished water pumping	TDH = 300 ft

Source: See Reference 3.

HVAC = heating, ventilation, and air conditioning; TDH = total dynamic head

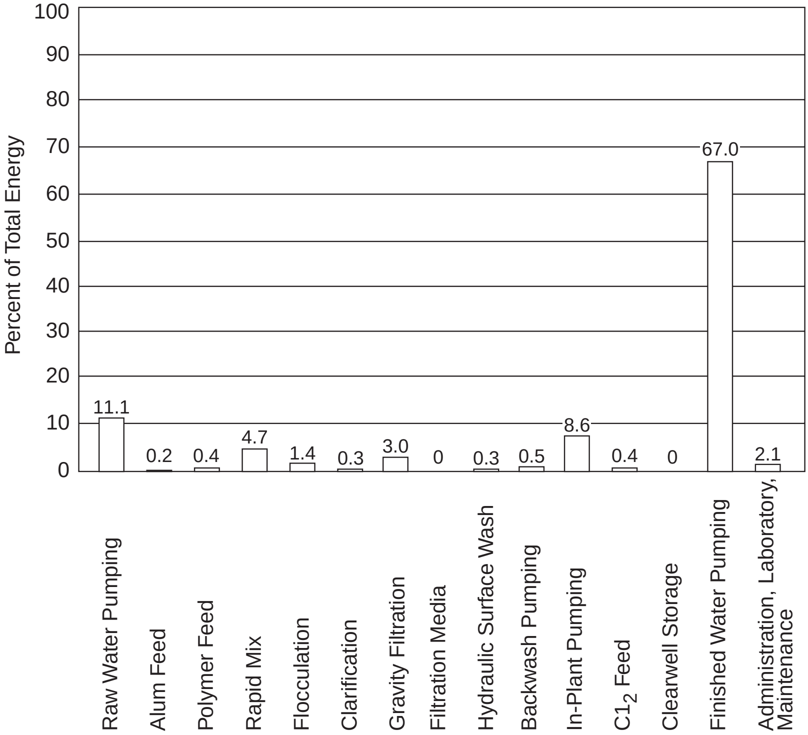

Fig. 30–4. Example relative distribution of energy at a 10 mgd (38 ML/d) facility

- Identification of goals and objectives for the study
- Discussion of expectations, needs, and success factors by representatives of the electric utility, water agency, funding agency, and other parties
- Identification of lines of communication and contact people
- Development of the schedule for conducting the study
- Detailed discussion of project goals and deliverables
- Identification and review of plant data collected to date and requests for additional data if needed
- Discussion of processes or areas to be included in or excluded from the evaluation
- Discussion of teamwork
- Creation of action items and schedule for completion
- Determination of the date for the next meeting

Create the Team

Energy optimization studies are often initiated by the engineering staff of the water facility or by the electric utility. The intent of the engineering staff in performing the study is to reduce energy costs. The electric utility's interest may be to reduce demand during peak hours. The interest of the operating personnel may be to minimize both the disruption of processes and the potential impacts on water quality. The project manager must integrate these interests into a successful project. The key is to create a team that pursues the project goals. Operating personnel represent the biggest challenge, because implementation of energy conservation measures (ECMs) may affect them the most. These staff members have honed the operation of the facility over many years and may be sensitive to changes. They can be especially sensitive to ECMs that could embarrass them or affect finished water quality.

These issues should be addressed in the kickoff meeting, where the operating staff must be put at ease. Asking for their help in identifying ECMs is one way to involve them in the optimization process. The project manager should mention that all ECMs will be discussed with staff before being published in a report that may be reviewed by their supervisors. This approach will give the staff members an opportunity to modify or eliminate ideas that could adversely affect plant operations or water quality. Teamwork should be emphasized at the kickoff meeting as well as throughout the entire project.

Collect System Data

Obtaining plant data is essential to initiating the study. The data collection effort should be appropriate for the level of effort for the overall study. Useful data can include

- Plant flows (average and yearly total for 2 years)
- Two years of electric and natural gas bills
- Electric load profile
- Pumping records and pump performance curves
- Documentation of all pressure-reducing/regulating valves
- Number of hours per day the plant is attended and hours per day the plant is operated

- Design summary and drawings, as well as specifications
- Normal operating time for intermittently operated processes, such as filter backwashing and residuals handling
- Utility bill schedule and possible alternative schedules
- Water treatment plant water quality standards
- Information from previous pump, lighting, and HVAC (heating, ventilation, and air conditioning) audits or testing
- Description of finished water storage facilities

Based on plant data obtained, the ‘‘big picture’’ can be determined for water treatment facilities in terms of unit energy consumption (energy consumed per million gallons of water treated). Typical values for various types of water treatment plants are shown in Figure 30–5. Plants with unit energy consumption in the lower range for their category would be considered energy efficient and may not need a detailed energy evaluation. Those in upper ranges for their category could potentially benefit from a detailed evaluation.

Evaluate Utility Bills and Schedules

Unfortunately, most people at water utilities do not understand energy costs or how those costs are billed. In fact, supervisors and managers at many plants never see

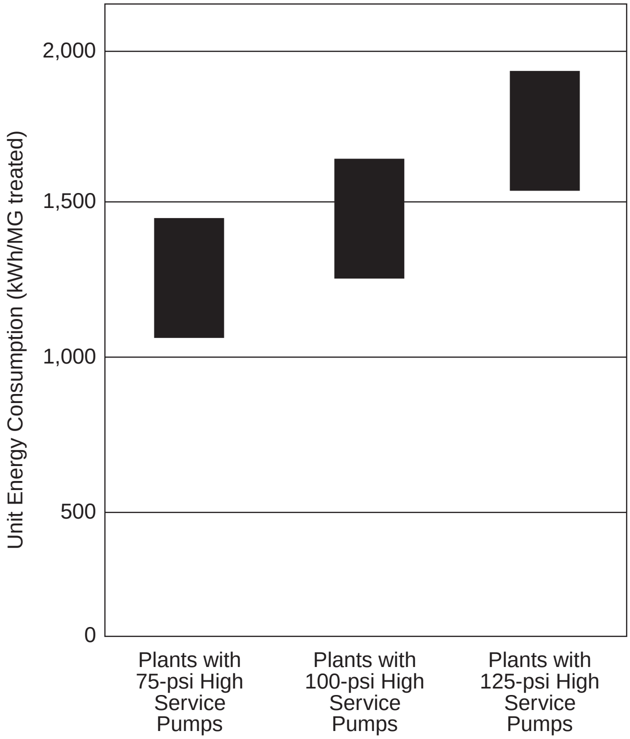

Fig. 30–5. Ranges of unit power consumption for typical water treatment plants

energy bills. Their only knowledge of energy cost may be that it is a line item for their yearly budget. An explanation of time-of-use rates, demand charges, power factor charges, and other elements of a bill will serve as a foundation for plant staff to reduce future energy and demand. Involvement of the electric utility account manager is essential. Electric utility billing schedules are discussed in more detail later in this chapter.

Conduct Field Investigation

The heart of an energy evaluation is a detailed field investigation, which is used to gain a thorough knowledge of the plant or facility operations. A brief investigation can be accomplished in a day or less. Two or three days may be required for a detailed process investigation. The time required is a function of plant size and complexity, project budget, schedule, level of detail desired, and expertise of the evaluators. Typical activities of a detailed field investigation include

- Obtaining information for the equipment inventory (described later)
- Detailed discussions with plant management and operations and maintenance (O&M) personnel on how each process or system is operated
- Discussions with plant staff of the operational effects of energy conservation ideas
- Soliciting energy conservation ideas from plant staff
- Obtaining field information needed to ask the ‘‘what if’’ questions described below
- Ascertaining time-of-day usage for equipment

Examine “What If” Scenarios

One way to analyze each process is to evaluate ‘‘what if’’ scenarios. To develop an ECM, the following questions should be asked about each process or piece of equipment:

- Does the process or equipment need to run at all?
- Can the process or equipment achieve the same results at a lower flow or capacity to enable reduced energy?
- Can it be run for fewer hours (e.g., part-time versus full-time use)?
- Can the operation be shifted from peak hours to off-peak hours?
- Will a variable-speed drive help to reduce demand or energy if the equipment is subjected to variable flows or loading conditions?
- Is the process or equipment efficient at the existing loading conditions, or does equipment need to be modified or replaced?

Pump efficiency testing will determine whether equipment is efficient at the existing conditions. Often, modifications of equipment such as pumps are needed because operation is at an inefficient or unstable point. To improve pump efficiency, for example, one or more of the following steps may be necessary:

- Replacing bearing rings or impellers
- Replacing existing impellers with impellers sized for the actual operating point

- Using a variable-frequency drive to operate at a more efficient point
- Replacing the pump with a unit that will operate more efficiently

Create Equipment Inventory and Distribution of Demand and Energy

A typical equipment inventory accounts for the following information:

- Name of equipment and process area
- Nameplate horsepower or measured kilowatts
- Load factor, that is, estimate of actual motor load divided by nameplate horsepower (used if measured power is unavailable)
- Hours of operation per year
- Kilowatt-hours per year

A worksheet for field use is presented in Figure 30–6. The information from the inventory can be arranged by process and plotted to show the distribution of energy or demand. A sample distribution of energy at a 10-mgd (38-ML/d) water treatment plant is presented in Figure 30–4. Raw water pumping, rapid mixing, filtration, in-plant pumping, and finished water pumping are the most energy-intensive; hence, conservation efforts should concentrate on these processes.

Develop ECMs and Implementation Strategies

The preceding tasks provide the background for identifying ECMs. These measures can be generated by any member of the project team, and plant staff should be encouraged to participate. The project team should discuss ECM feasibility, because not all ECMs will be viable. The following factors should be considered in determining feasibility:

- Capital cost and operating costs
- Energy and electrical demand savings
- Effect on plant processes or finished water quality
- Payback, cost/benefit ratio, present worth, or other means of quantifying the ECM
- Effect on plant labor or chemical costs
- Complexity
- Safety
- Availability of financing, funding, and rebates
- Implementation schedule: short, medium, or long.
- Risk
- Payback: no, low, or high

Most ECMs at water facilities affect operation. ECMs should be discussed with plant personnel as they are identified. ECMs that are difficult or impossible to implement should be deemphasized or eliminated.

Many electric utilities have rebate programs for lighting, heating, energy-efficient motors, variable-speed drives, and other equipment upgrades. Rebate programs should

Equipment Inventory

Process Area or Equipment	Nameplate Horsepower or Measured Kilowatts	Load Factor*	Hours Per Year Operated	Kilowatt-Hours per Year

* Estimated operating horsepower as a percentage of nameplate horsepower if power cannot be field measured.

Fig. 30–6. Electrical equipment inventory sheet be clearly described, and the effect of the rebate program on paybacks should be identified. Funding and budgeting are key to implementation of cost-intensive ECMs. Grants, loans, and technical support are available from some federal, state, and local sources. State energy offices often have detailed information on the available funding sources.

Follow-up, Monitoring, and Project Implementation

An often overlooked aspect of energy optimization efforts is the follow-up process after the analysis is completed. Often these reports are put on bookshelves and simply forgotten because of funding problems. The follow-up process should take as long as 2 years to check the progress of implementing conservation measures.

Parameters to be monitored include but are not limited to the following:

- Plant flow
- Plant demand, in kilowatts
- Plant energy, in kilowatt-hours
- Monthly energy costs
- Demand and energy for individual processes or pieces of equipment
- Unit energy consumption for the plant, in kilowatt-hours per million gallons (or million liters) treated
- Pump efficiency as measured by (1) wire-to-water efficiency and (2) unit energy consumption

These parameters should be monitored both before and after ECMs are implemented. The suggested frequency is at least once every 6 months for 2 years. After that, yearly monitoring may suffice. The savings achieved should be compared to the savings identified in the optimization study. If the savings achieved are less than the identified savings, then further investigation may be needed.

Many ECMs cannot be implemented until the water utility’s next budget cycle. Therefore, many measures could take more than a year to implement.

ECMs with high capital costs may require special funding or grants. Identifying electric utility rebate programs, if any, as well as potential financing or other grants, is important. The follow-up process can enable the development of a recognition program that singles out departments and/or individuals for awards as the unit energy usage at the facility drops. These awards can include certificates, trophies, or wall plaques—even cash awards if appropriate. The recognition program could be further enhanced by special presentations by the electric utility at city council or board-of-directors meetings. These presentations could single out individuals or groups for commendation and award. This approach provides a very high profile for energy conservation at the facility.

ELECTRIC MOTORS

The vast majority of energy in the waterworks industry is consumed by electric motors. Therefore, sizing, applying, and operating electric motors to ensure maximum efficiency are essential. Two approaches are used to improve efficiency in existing and proposed systems:

- Use of premium efficiency motors
- Operation of motors at or near the nameplate rating

Premium Efficiency Motors

Premium efficiency motors have gained widespread use in recent years. Relative to standard efficiency motors, they offer motor efficiency gains ranging from 1.0 percent at 200 hp (150 kW) to 9.0 percent at 1 hp (0.75 kW) at 1,800 rpm. Table 30–4 summarizes efficiencies for premium efficiency (at 100 percent load) motors.[8] Figure 30–7 illustrates full-load efficiency of standard and premium efficiency motors.[9]

Premium efficiency motors cost more than comparable standard efficiency motors. For new installations, the extra cost of premium efficiency motors can usually be recovered within a few years or less if the motors are operated more than 50 percent of the time. In fact, where power costs are above $0.05/kWh and motors are operated more than 50 percent of the time, premium efficiency motors offer exceptional value for new installations. Economics must be evaluated carefully where standard motors are being retrofitted with premium efficiency motors. A methodology for determining simple payback for conversion from standard motors to premium efficiency motors is presented in Figure 30–8.[7]

Benefits of premium efficiency motors include the following:[4]

- *Lower energy costs.* Premium efficiency motors use energy more efficiently, and their superior design may allow them to provide a higher power factor.
- *Durability.* Because of the high-quality design and materials, premium efficiency motors generally last longer, require less maintenance, and are more reliable—resulting in less equipment downtime. Consequently, premium efficiency motors often carry a warranty two to three times longer than that for a standard motor.
- *Improved tolerance to overvoltage.* Many premium efficiency models are more resistant to overvoltage conditions.
- *Interchangeability.* Design improvements are internal to the motor and do not affect the frame size or mounting dimensions.
- *Lower noise.* Premium efficiency motors run more quietly than standard motors.

To increase efficiency, emphasis is given to reducing the five major types of motor losses:

- Magnetic losses
- Windage and friction losses
- Stator losses
- Rotor losses
- Stray load losses

To accomplish this goal, premium efficiency motors may include some or all of the following design features:

- High-grade electrical steel to reduce magnetic losses
- Longer core to lower flux density
- Increased cooling capacity to reduce magnetic and load losses

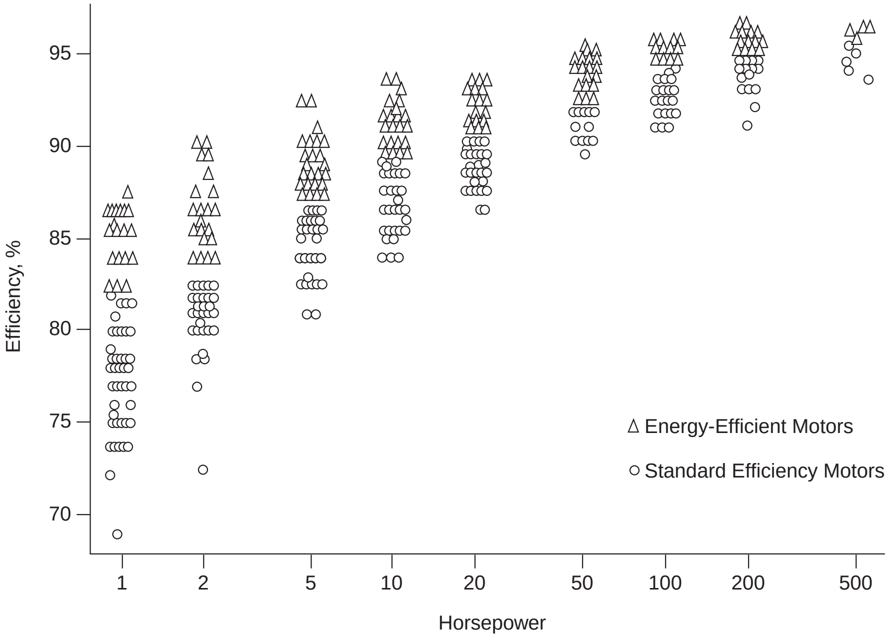

Fig. 30–7. Efficiencies of standard and energy-efficient motors (*Source:* Reference 9)

- Thinner laminations to reduce eddy currents and thus reduce magnetic losses
- Improved fan design to reduce windage losses
- More copper in windings to improve cooling to reduce stator losses
- Increased conductor cross section to lower stator load losses
- Larger rotor bars and rings to reduce resistance and thereby lower rotor load losses

Motor Sizing

Frequently, motors are oversized for their actual load. As indicated in Figure 30–9, the efficiency of induction motors remains fairly constant over a broad range of mechanical loadings but drops rapidly below about 50 percent of rated load.[9] Smaller motors are particularly susceptible to efficiency deterioration at lower loading. There are many good reasons for oversizing motors. Longer service life under adverse operating conditions is the most important. However, these considerations should be weighed carefully against capital cost and efficiency, the latter of which may be lower if motors are grossly oversized.

VARIABLE-SPEED DRIVES

Traditional approaches to pumping system design, which evolved when low-cost energy was abundant, are based on component analysis and selection; thus, they do not provide adequate consideration of overall system operation. Proper pump selection

1. Calculate reduction in electrical load:

$$(\mathrm{kW})_R = (0.746) \times (\mathrm{hp}) \times \frac{1}{\mathrm{Eff}_s} - \frac{1}{\mathrm{Eff}_p}$$

where:
$(\mathrm{kW})_R$ = reduction in electrical load, in kilowatts
0.746 = factor for conversion between horsepower and kilowatts
hp = operating horsepower of motor
Eff_s = efficiency of standard motor
Eff_p = efficiency of premium efficiency motor

2. Calculate annual savings:

$$\text{annual savings} = [(\mathrm{kW})_R \times \text{runtime} \times (\mathrm{kWh})_C] + [(\mathrm{kW})_R \times (\mathrm{kW})_C \times 12 \text{ months}]$$

where:
runtime = hours of operation per year
$(\mathrm{kWh})_C$ = electrical rate, in \$/kWh
$(\mathrm{kW})_C$ = monthly demand charge, in \$/kW

3. Calculate motor cost:

   a. If existing motor is due for major repair or replacement: $C_I = C_p - C_r$
   b. If replacing a working motor: $C_I = C_p$

where:
C_I = incremental motor cost
C_p = cost of premium efficiency motor
C_r = repair/rewind cost of existing standard efficiency motor

4. Calculate simple payback:

$$\text{years} = \frac{C_I}{\text{annual savings}}$$

Fig. 30–8. Methodology for determining simple payback for conversion from standard motors to premium efficiency motor (*Source:* Reference 7) (Courtesy of EPRI)

represents an important method of energy conservation in municipal water systems. Since the 1950s, variable-speed pumping has become popular to match pumping to variable system conditions. In the past, little attention was focused on the efficiency of variable-speed controllers. Increases in energy rates and a corresponding reduction in the cost and complexity of "energy-efficient" drive systems have led to a dramatic increase in the use of these systems.

The constant-speed drive with a simple on–off control is the most efficient overall drive unit, provided it operates at or near the most efficient point on the pump curve. Unfortunately, this system is not suitable for all pumping applications, and some method must be used to control the output from a pump, fan, or other equipment.

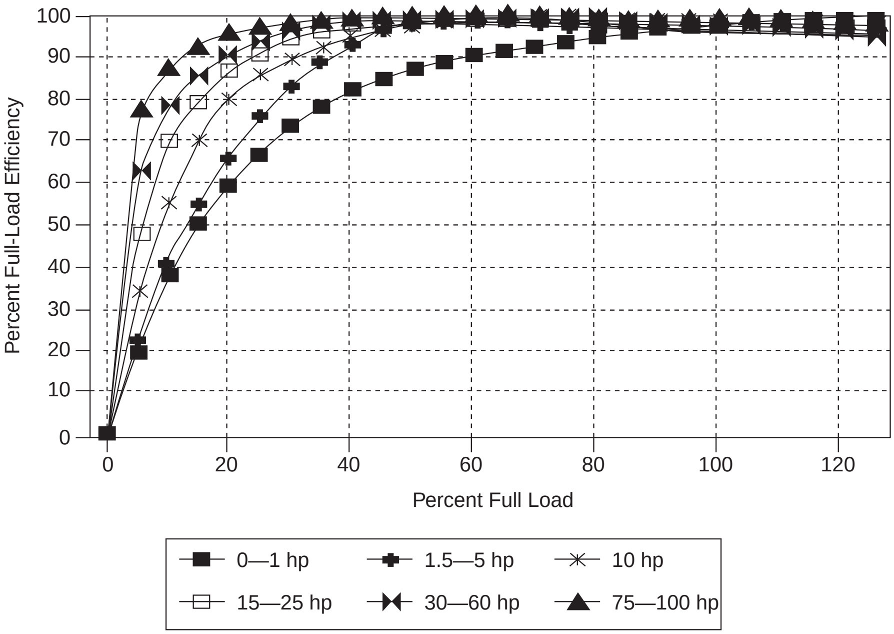

Fig. 30–9. Typical percent full load efficiency versus load curves for three-phase motors (*Source:* Reference 9)

The throttling valve is perhaps the simplest and lowest-cost flow control device. It relies on ‘‘burning up’’ head to achieve the desired system operating conditions. Flow control of raw water pumps is often achieved by using a throttling valve. The relatively low efficiency of this type of control relegates its use to systems where simplicity rather than efficiency is preferable.

Speed variation is another efficient method to control equipment output. The following is a listing of drive types currently available for variable-speed control:

- Variable voltage
- Hydraulic clutch or coupling
- Eddy current
- Wound rotor motor with resistance or reactance secondary control
- Variable-frequency drives
- Wound rotor motor with secondary power recovery (slip recovery)
- Silicon controlled rectifier (SCR) DC

The line-to-shaft efficiencies of several variable-speed devices are shown in Figure 30–10.[10] As the figure indicates, the efficiencies of the eddy current, wound rotor, and hydraulic coupling types are poor. Essentially, these devices use approximately the motor nameplate horsepower at any speed, whereas variable-frequency drives (VFDs) use much less.

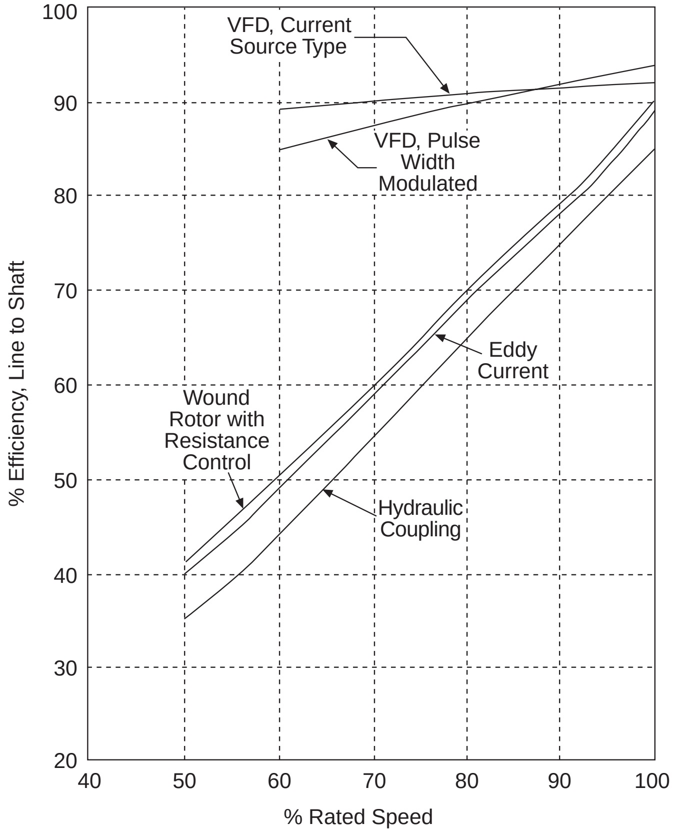

Fig. 30–10. Comparative efficiencies of variable-speed drives (*Source:* Reference 10) (Adapted from *Journal AWWA,* Vol. 72, No. 6 (June 1980), by permission. Copyright © 1980, American Water Works Association.)

Variable-frequency drives are now the most common electronic devices used to control motor and equipment speed. VFDs eliminate the hardware used by mechanical or hydraulic adjustable-speed drives to control pump shaft speed. They work with motors of almost any size in any location, from heat pumps to fans to pumps in water treatment plants and distribution systems.

Benefits of VFDs

VFDs have numerous benefits.[7] They are more easily operated and feature better power factors and lower audible noise than mechanical adjustable-speed drives. In proper applications, VFDs provide the following:

- *Reduced energy costs.* VFDs enable pumps, fans, or other equipment to track flow demands by decreasing motor speed, as opposed to burning up energy in flow control valves.

- *Extended motor life.* Abrupt starts and stops put a motor under stress from high torque and electrical current surges. The electrical current drawn at motor start without a VFD is usually six to ten times the full-load current. The VFD's soft starting capability brings motors to full load gradually, resulting in less wear or stress on motor bearings, shafts, windings, and insulation. This ability can also reduce voltage sags that could affect other equipment on the electrical system.
- *Increased capacity of standby generators.* VFDs require lower starting current, enabling standby generators to handle more load.
- *Improved control and equipment reliability.* VFDs can be used to vary chemical feed rates and process equipment motor speed, allowing for more efficient process control. Wear and maintenance needs are reduced by eliminating unnecessary starts and stops.

Motor Selection for VFD Operation

VFDs work on both induction and wound-rotor motors.* VFDs are commonly installed on existing motors when the motor is still in good condition. The suitability of the existing motor should be checked for operation with a VFD. For example, if harmonics are present, increased heating may damage the winding insulation on motors not rated for such conditions.

Motor cooling is also a critical factor. Power requirements of positive displacement pumps or blowers are directly proportional to speed. In these applications, as well as others, motors can overheat at speeds less than 50 percent of full nominal speed. Cooling is less of a problem for centrifugal applications, where power is proportional to the cube of speed.

Motors are now available with inverter duty ratings. These motors are essentially premium efficiency models with higher insulation ratings, underrated to protect against harmonic current heating. Models with inverter duty ratings, as well as energy-efficient models, perform better with VFDs than standard motors.

Potential Problems with VFDs

A VFD can cause certain side effects, all of which can be controlled with proper planning and design. The following paragraphs discuss some commonly encountered problems.

Harmonic Distortion VFDs can contribute to high harmonic currents in the power supply. These currents result in the need for electrical equipment, such as conventional power transformers, to be derated (oversized). Otherwise, the transformer or circuit breaker that appears to have adequate capacity to handle the 50-Hz load could overload from high harmonic currents. Therefore, transformers must be evaluated before VFDs are added.

Selecting a method to control harmonics is part art and part science. Although small VFDs may not produce significant harmonic currents, larger units can create substantial problems if steps are not taken to mitigate harmonics. For design purposes, these

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*This section and the following section draw from EPRI (1997); see Reference 7.

problems must be addressed. The type and magnitude of the problem can be characterized through a harmonic analysis.

There are several ways to solve harmonic problems. VFD isolation transformers effectively reduce harmonic currents but are usually the most expensive method. Input reactors cost less than isolation transformers and can be just as effective Harmonic filters reduce harmonic distortion levels by providing a low-impedance path for the harmonic currents. Many VFDs are now available with built-in harmonic filters. Harmonic-free 18-pulse pulse-width-modulated (PWM) VFDs also control harmonics. They are fairly new to the market and require a careful comparison of cost and capabilities.

Unfortunately, there is no clear way to determine how to control harmonics. Larger installations require case-by-case evaluation by an experienced designer familiar with VFDs. For smaller applications, plant staff can experiment through trial and error.

Audible Noise Specifying the acceptable limits of audible noise (in decibels) from the VFD-controlled motor is important. Pulse-width-modulated drives supply a relatively clean voltage to the inverters. The use of PWM drives will also reduce the harmonic heating in the motor, but it can create an irritating high-pitched noise due to the higher modulation frequency (on the order of several kilohertz). Generally, an output choke will reduce the higher-frequency harmonics in the motor that produce the audible noise.

Capacitors Facilities that have capacitors installed for power factor correction may experience problems when VFDs are added. Capacitors for power factor correction should not be applied to VFD-driven motors and are normally not needed. The PWM drives have high power factors, thus eliminating the need for correction.

Equipment Conflicts A VFD installed near a Doppler-type flowmeter will cause problems with the flowmeter. Similarly, VFDs can cause problems with other electronic equipment. VFD manufacturers can provide further information on compatibility problems with their units and existing or proposed equipment.

Water System Applications for VFDs

Constant-speed pumps, especially those with throttling valve flow control, have inherent inefficiences. With constant-speed pumping, pump efficiency changes when discharge characteristics (i.e., total dynamic head [TDH] or flow) change. If a throttling valve is used to control flow, energy is lost in the form of heat through the valve.

In contrast, VFDs enable operators to match motor speed to the immediate pumping need and avoid using excessive energy. VFDs can be applied to equipment in the following circumstances:

- Where raw-water supply pumps are being throttled with rate-of-flow control valves to meet variable demands within the treatment plant
- Where fluctuating groundwater levels, which create changes in the suction head and resulting TDH for well pumps, are present
- Where fluctuating demands in the distribution system, which create variable flow and pressure conditions for high service pumps, are present

- Where fluctuating reservoir levels create significant changes in the TDH for high service pumps
- Where distribution systems are being controlled by pressure-reducing or -regulating valves. VFDs can be used in lieu of these valves to maintain system pressure without burning up head unnecessarily across the valve

Figure 30–11 illustrates a scenario where a VFD has been used to lower pump motor speed from 1,800 to 1,200 rpm. The resulting difference in horsepower for the lower flow is 56 hp (dropping from 90 to 34 hp). If a throttling valve had been used to decrease the flow by inducing a higher TDH and backing up along the 1,800-rpm pump curve, the resulting difference in horsepower would be only 20 hp (from 90 hp to 70 hp).

ELECTRIC UTILITY BILLING SCHEDULES

Electric rates vary among utilities. Rate schedules applicable to water systems are composed of some combination of demand, reactive demand (power factor), standby, energy usage, and time-of-day charges.

Demand charges are based on the maximum average demand in kilowatts for a 15-, 30-, or 60-minute billing period. These charges can be significant, amounting to 25 to 30 percent or more of the total electrical costs. The maximum demand and charge can carry over for 1-, 3-, 6-, or 12-month (or longer) periods. The basis of the

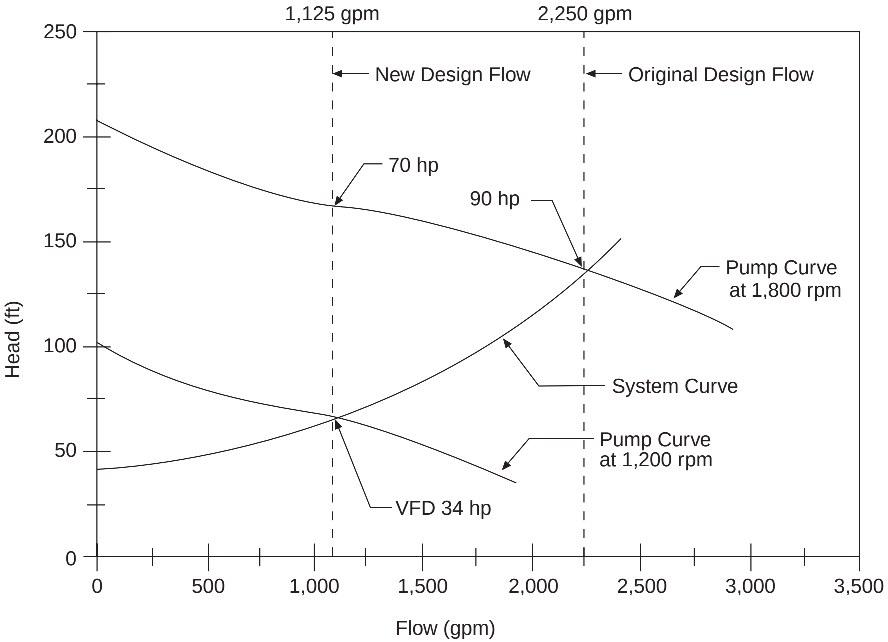


Fig. 30–11. Comparison of flows and loads for throttling valve versus VFDs

charge is the electric utility's responsibility for providing power generation and transmission capacity to meet peak demands when needed. Demand charges can be reduced by making diligent efforts to ensure that high-energy devices are not operated simultaneously. For example, additional system storage can reduce peak pumping demands by allowing high water production and storage during off-peak hours. The water utility can also encourage regulations that mandate that swimming pool filling and lawn watering be rescheduled to low-demand periods. This approach has been taken in many communities along the west coast of the United States. Additional demand savings are possible by reducing the system head curves of the existing distribution system; this goal is accomplished by cleaning and lining or by paralleling and looping to allow fewer pumps at lower heads to meet water demand.

Power factor charges are used by some electric utilities to compensate for the increased cost of supplying energy to customers with certain electrical load characteristics. For example, induction-type motors commonly used to drive pumps can exhibit unfavorable power factors, especially when the motors are lightly loaded. Two common methods of charging for power factors are a direct charge for reactive demand if the power factor drops to a predetermined value, typically 85 to 95 percent, or a charge for each percentage point by which the power factor drops below a stated value. In some cases, customers are also offered a credit if the power factor is above the stated value.

Capacitors are added to a system to increase power factor. Their installation requires detailed analysis to determine cost-effectiveness and the best location. In most cases, power factor correction frequently is not cost-effective because power factor charges are generally small. Capacitors can be installed on the primary or secondary side of the transformer, at the motor control center, or at the motors. Installation at the motors is the preferred method from an energy standpoint, because it minimizes voltage drop from additional current flow. Power factor correction has the advantage of freeing up transformer and feeder capacity, which are limited by the high currents when the power factor is poor.

Many large electrical customers (usually 500-kW demand and over) are placed on time-of-day rate schedules. Facilities can take advantage of this system by scheduling equipment to run during off-peak hours or to operate at lower capacity on peak and at higher capacity off peak. Operations such as filter backwashing and residuals dewatering can be performed during this period; if storage is available, pumping can be increased at these times as well. Energy management systems, described in the next section, are very effective in automatically controlling equipment to operate during off-peak hours.

ENERGY MANAGEMENT SYSTEMS

A majority of energy conservation savings can be realized by using relatively simple control devices. Many of these measures can be planned and implemented by operating personnel. Alternatively, microprocessor-based energy management systems, offer convenience, sophisticated controls, and an extensive database. Typically, these latter systems can provide

- HVAC energy management
- Lighting energy management

- Power monitoring and process control to minimize peak use
- Security
- Fire protection
- Maintenance scheduling
- Report generation

These systems should be applied with realistic expectations. They are versatile, but there are limits to their capabilities. They can be very cost-effective if integrated with a plant control system. Systems to aid in shifting to off-peak use range from simple ones that schedule for the operation of pumps, such as backwash, to relatively sophisticated computer programs that monitor and predict system needs to optimize storage to defer pumping.

The following software features should be considered for any energy management system:

- Prioritized selection of the most energy-efficient (in terms of kilowatt-hours per million gallons or million liters) booster pumps, plant pumps, and wells
- Real-time monitoring, load reduction, or demand load shedding to take better advantage of time-of-use rate structures
- Monitoring of real-time electrical energy prices from a variety of suppliers and selection of the lowest-cost power available. This capability will be advantageous as electricity markets are deregulated.
- Sounding an alarm when a demand target is exceeded or when a pump or other equipment's efficiency changes from the target condition
- Automatic operation of pumping systems to fill storage tanks during off-peak or partial-peak utility rate periods (for facilities where there is sufficient storage)
- Using programmable logic to control VFDs for maintaining distribution system pressures and reservoir/wetwell levels
- Locking out certain equipment or pump stations during peak periods

Energy management systems that monitor and control electrical demand can be inexpensive and highly effective. They operate by regulating real-time demand information (taken directly from the electric utility meter) and comparing the demand to a preset target value. If the target value is approached or exceeded, an alarm will sound and a manual or computerized load-shedding program will initiate. Energy from nonessential or even essential equipment will be shut off or reduced until the demand is reduced below the target or alarm value.

The impact of installing a simple energy management system is shown in Figures 30–12 and 30–13. In 1994, HDR conducted an energy optimization study of a 1.2-mgd (4.5-ML/d) water treatment plant and recommended the implementation of a simple EMS to control demand. The system was installed in late 1995. As shown in Figure 30–12, the demand was systematically reduced from the 150–175-kW range to approximately 110 kW. More important, costs were reduced by over $1,000 per month, a 17 percent decrease (Fig. 30–13).

Most electrical utilities are anxious to assist with planning and implementation of energy conservation measures. Utilities should be contacted early in the project planning stage in order to gain maximum benefit from their advice.

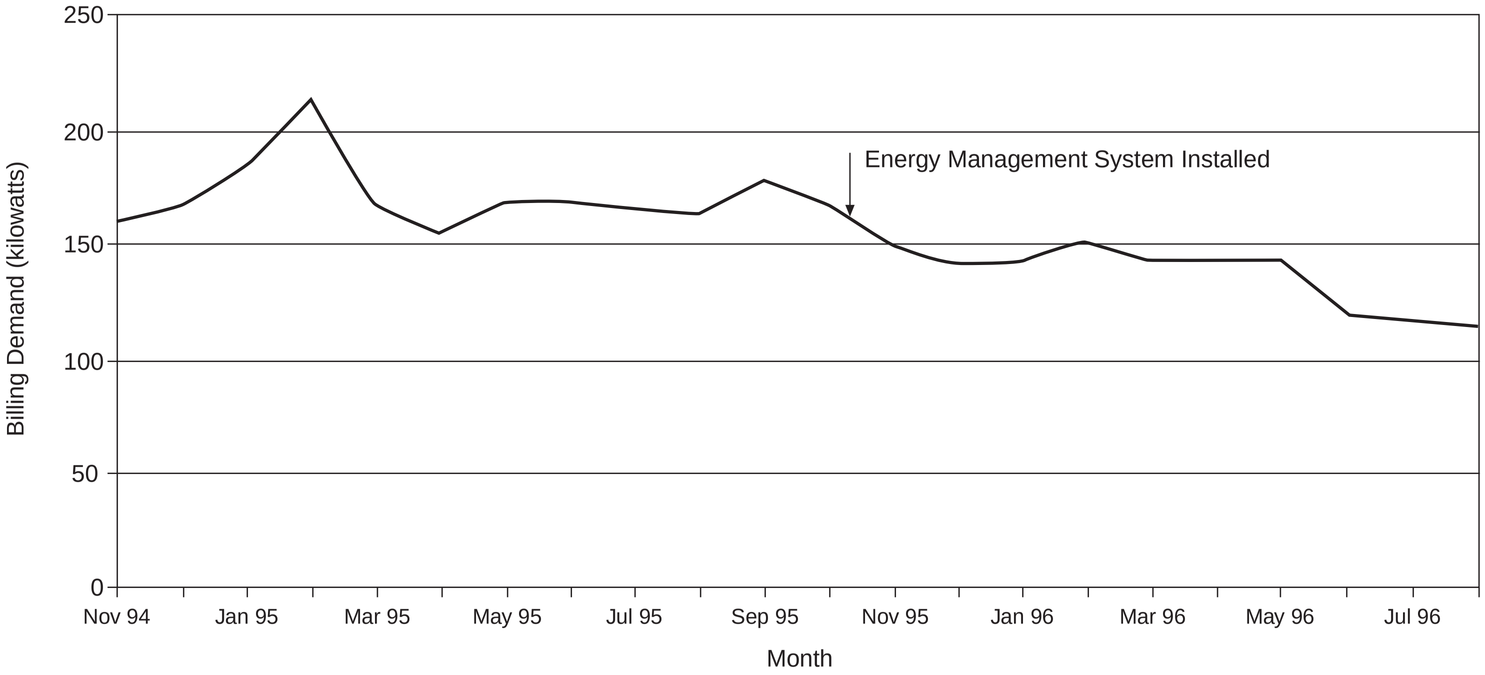


Fig. 30–12. Impact of installation of an energy management system on billing demand for a 1.2 mgd WTP

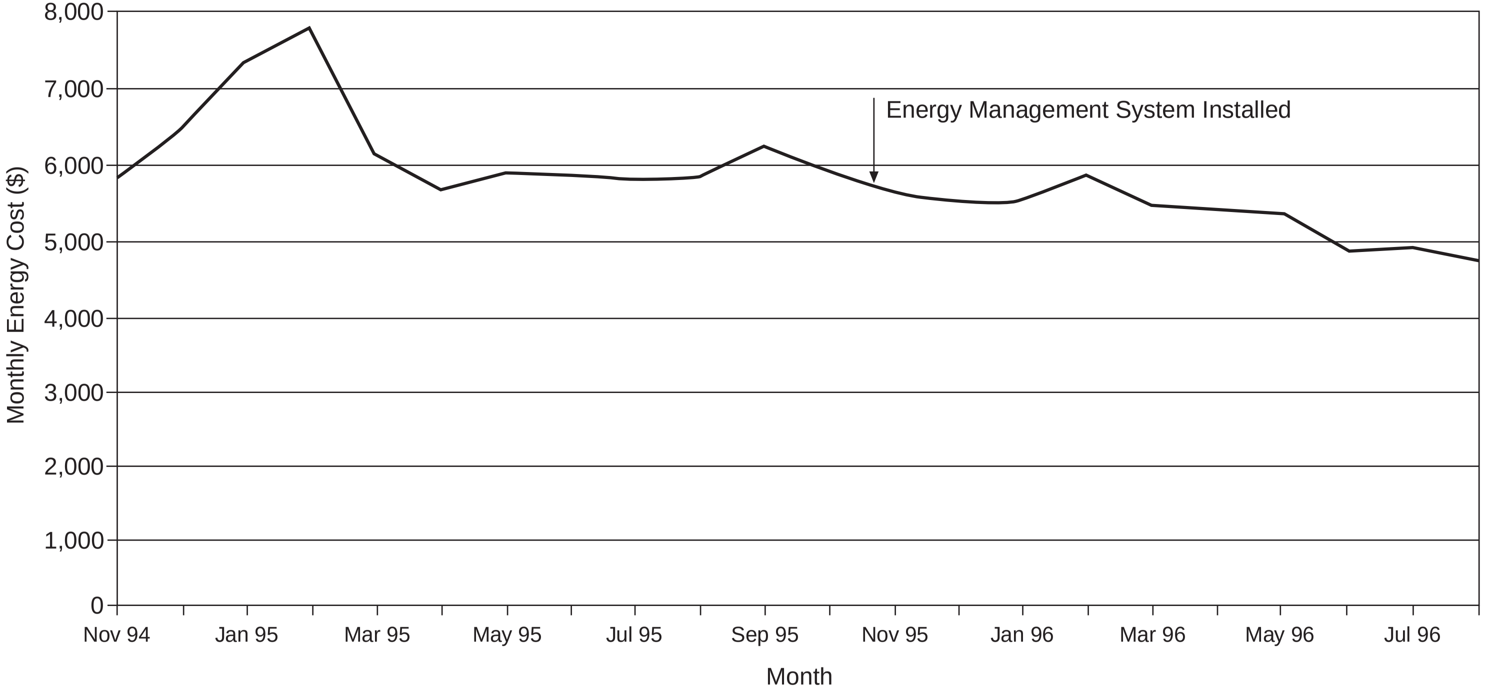


Fig. 30–13. Impact of installation of an energy management system on monthly energy cost for a 1.2 mgd WTP

SUPERVISORY CONTROL AND DATA ACQUISITION SYSTEMS

Advances in computer hardware, communication networks, and software continue to enhance the control and management of water supply, treatment, and distribution systems. In addition to improved control within the treatment plant, operators can make informed decisions related to energy savings through modern SCADA systems tied to remote reservoirs, distribution system monitoring points, and pumping sites.

Many older plants still operate under their original control systems and could benefit from an upgrade. When assessing the feasibility of upgrading controls and data acquisition capabilities, facilities should evaluate the cost and potential energy savings, as well as the other benefits.[7]

SCADA Components and Operation

A typical SCADA system consists of a supervisory control station or master station, remote terminal units (RTUs), communications devices, interconnecting conductors required for the input-output (I/O) to the RTUs, primary elements, transmitters used to monitor and control process conditions, and software to allow automatic monitoring and control of the system. RTUs can include programmable logic controllers (PLCs).

The master station usually consists of one or more computers with operator interface capabilities, including a keyboard, color monitor, report printer, event and alarm printer, hard disk drive, network interface, and uninterruptible power supply (UPS). A tape or optical disk is often included for archiving data. Projection video and large-screen monitors are included in some systems. The RTUs are typically connected to a master station by radio, fiber optics, or a leased telephone line. Most RTUs have their own microprocessor and control the process by monitoring field inputs and performing various control functions by transmitting electronic signals (to start pumps, open valves, etc.) as programmed.

Typical RTU I/O points consist of analog (proportional) and discrete (on–off or start–stop) signals. Analog signals are usually 4–20 mA or 24 VDC. Digital signals can be of various voltage levels from 12 VDC to 120 Vac. The I/O point information is transmitted to the master station and converted to graphical displays so that the operator can view the process conditions (tank levels, flow water pressures, pump conditions, etc.). This information can be archived on tape or disk for future reference. Processes such as starting or stopping pumps and opening or closing valves may also be manually controlled from the master station. Various graphical displays can be created by the user at the master station.

Figure 30–14 is a simplified SCADA system schematic.[7] RTUs are shown in typical applications used to control and monitor critical production treatment and distribution system components. The master station, which allows for operator interface to the control system, is shown at the treatment plant. In some instances, this may be part of an integrated system located at some other centralized location that is used to monitor and control additional municipal facilities.

Optimized pumping combinations can be determined by computer-generated pumping schedules. This goal is accomplished by integrating data from remote monitoring instruments with known pump characteristics and billing rates. Tank elevations can be monitored by the SCADA system from remote sites. This allows the operator to find the most efficient pumping schedule that will put levels at the desired elevations at the end of a pumping period. Various pump combinations capable of meeting the demands

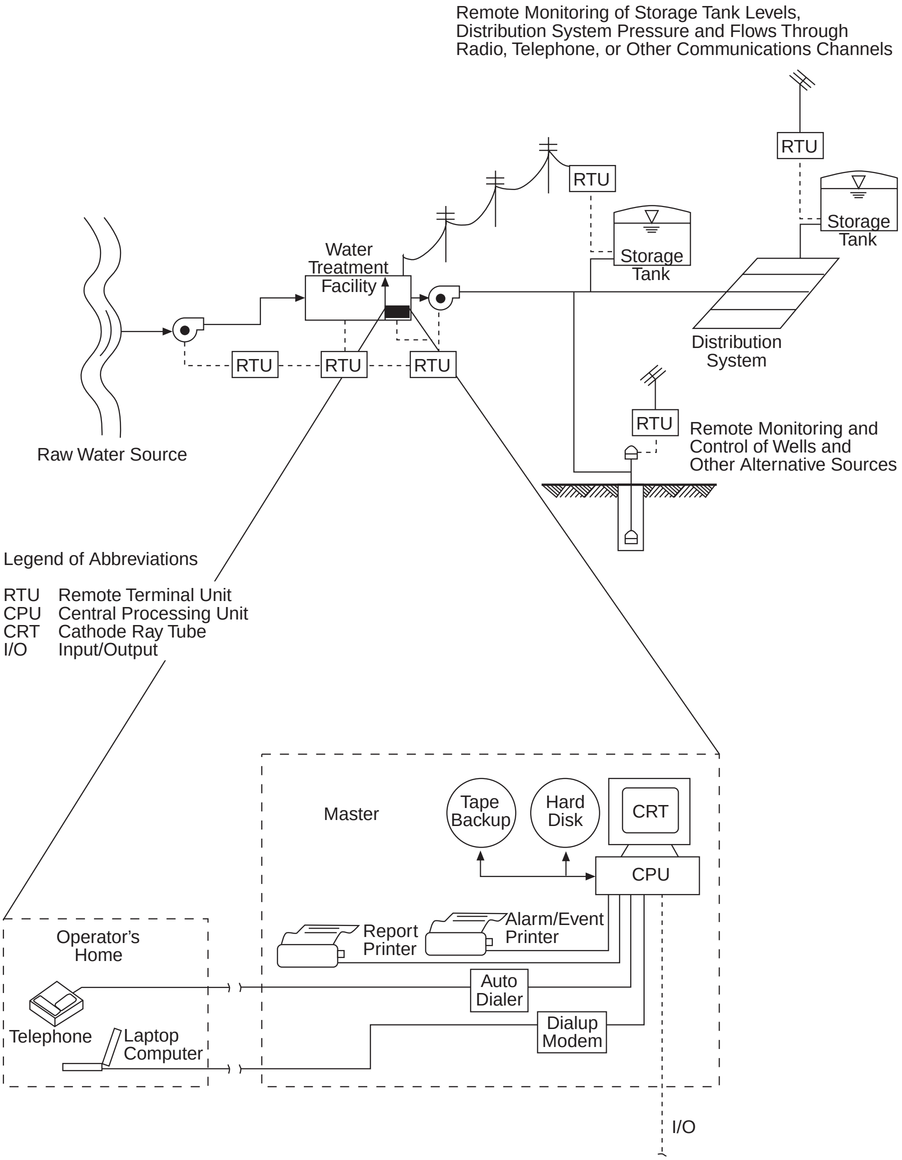


Fig. 30–14. Simplified SCADA system schematic (*Source:* Reference 7) (Courtesy of EPRI)

can be evaluated and ranked according to cost efficiency in order to determine the most economical combination.

A network analysis system can be incorporated into the SCADA computer so that information on pump characteristics can be obtained over a wide range of tank level conditions and system demands without actually placing the pumps on-line. Energy management can also be integrated into the SCADA system.

Benefits of SCADA Systems

SCADA systems automatically monitor and control water treatment and distribution systems. They offer a variety of benefits:[7]

- *Energy cost savings.* SCADA systems provide a central location for monitoring and controlling remote energy-consuming devices. Decisions regarding when to run specific equipment items are made with the aid of computer logic. The ability to schedule operations and automatically start and stop devices—coupled with software-driven decision making—results in the most efficient operation of pumps, valves, chemical feed, and equipment. SCADA systems effectively achieve cost savings by facilitating pumping during off-peak hours, when energy and demand rates are lower. SCADA systems can incorporate energy management system hardware and software.
- *Reduced operating and maintenance costs.* SCADA systems help optimize labor and other resources by providing continuous and precise control of processes and service levels. Operators need not be at the site during all hours of operation, but they can provide input or retrieve data through remote monitoring and control. The central computer will generate maintenance reports, send an alarm to operators, or produce work orders when equipment needs scheduled or emergency maintenance.
- *More timely information.* Results are readily available because information is monitored on a continuous basis. Automatic data recording and report generation eliminate the need to manually record and report process parameters. Remote reservoir levels and system pressures can be monitored on a real-time basis for optimizing the control of pumps and valves. SCADA systems assess trends of critical process parameters quickly and easily to help with data analysis.
- *More accurate process control.* Centralizing control and including programmable control logic enhances field equipment control. For example, optimizing equipment run times reduces energy costs. More accurate control of chemical feed processes, such as coagulation and chlorination, can lower chemical usage and reduce the frequency of filter backwashes. The ability to meet drinking water standards is also improved through better control at the water treatment plant.

OPTIMUM CONVEYANCE SYSTEMS

The design of piping systems has long been guided by intuition and rule of thumb. Obviously, larger pipe sizes result in lower pumping energy; however, excessively large pipes are not economical. Therefore, several mathematical models have been developed for optimizing pipeline sizes. Patton and Horsley have developed a simplified approach to pipeline sizing based on the following equation:[11]

$$V_{\text{opt}} = 0.140\ C^{0.65} D^{0.0595} \left(\frac{PE}{KT}\right)^{0.35}$$

where:

V_{opt} = optimum pipeline velocity, ft/s
C = Hazen-Williams coefficient, dimensionless
D = pipeline diameter, ft
P = unit cost of pipe, \$/linear ft per inch diameter
E = average overall pumping unit efficiency (expressed as a decimal)
K = average price of electricity, \$/kWh
T = design life, years

From this equation it can be shown that

$$D_{\text{opt}} = 2.92\ Q^{0.486}\ C^{-0.316} \left(\frac{KT}{PE}\right)^{0.170}$$

where:

D_{opt} = optimum diameter, ft
Q = average flow, ft^3/s

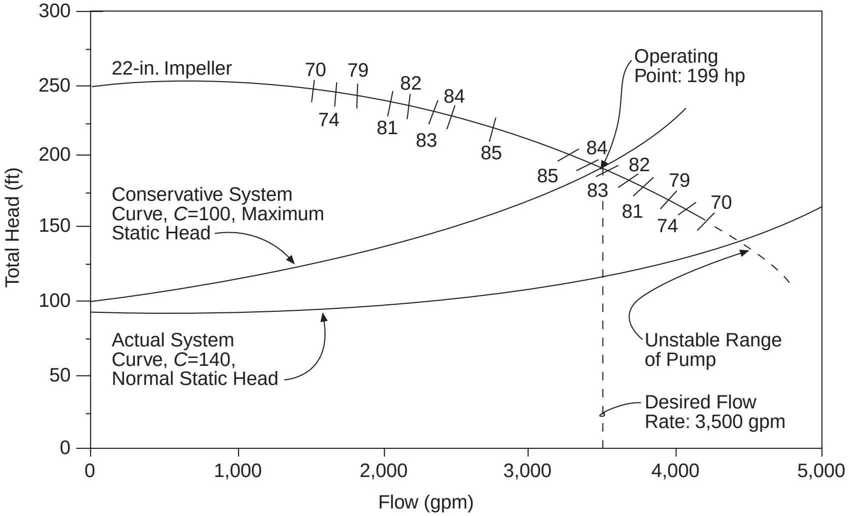
For example, for a flow of 5 mgd (7.74 ft^3/s or 18.9 ML/d), a Hazen-Williams coefficient of 100, a pipeline cost of \$2/linear ft per inch diameter, an overall pumping system efficiency of 75 percent, electricity at \$0.045/kWh, and a design life of 20 years, the optimum pipe size is 20 in. for a velocity of 3.5 ft/s (1.1 m/s).

PUMPING CONSIDERATIONS

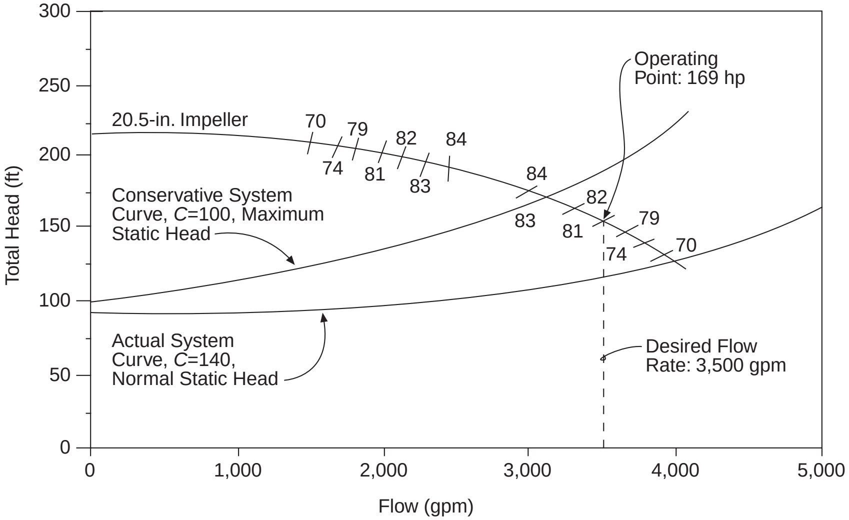
Pump Selection and Design

Oversized pumps represent a major source of inefficiencies in waterworks systems. Pumping stations are often designed for maximum flow at ultimate plant capacity. However, the best pump selection usually maximizes efficiency at average operating conditions rather than at maximum conditions. Consider an example case involving a constant-speed pump.

A constant flow rate of 3,500 gpm (220 L/s) is desired, and the designer selects a design point and pump based on “worst-case” conditions: a Hazen-Williams C value of 100 (as opposed to C = 120 at project midlife or C = 140 at start-up), maximum possible static head (as opposed to normal operating conditions), and fitting and valve headloss based on equivalent length (as opposed to the velocity head method). The worst-case system curve and an actual system curve are shown in Figure 30–15. As the figure shows, use of the worst-case design and the conservative system curve will yield a pump with a best efficiency point at or near the design flow of 3,500 gpm (220 L/s).



Pump Selection Based on Worst-Case Conditions (22-in. Impeller)



Pump Selection Based on Actual Conditions Expected for First 10 Years of Service (20.5-in. Impeller)

Fig. 30–15. Comparison of pump selection for ‘‘worst case’’ conditions versus actual conditions for first 10 years of service

However, when the pump is placed in operation, the actual system curve forces the pump to operate far to the right on the performance curve. The efficiency is poor, and the pump is operating in an unstable range that may subject it to damage due to cavitation and vibration. To achieve the desired flow rate of 3,500 gpm (220 L/s), a discharge valve must be throttled to reduce flow. Excess head is unnecessarily ‘‘burned up,’’ and an inefficient pumping system results (as measured by volume per unit of energy). The required input to the pump at 3,500 gpm is 199 hp (149 kW).

The designer would have made a better choice by selecting a pump based on the most probable conditions within the first 5 to 10 years after start-up of the facility or installing a VFD. Figure 30–15 also shows how a smaller 20.5-in. (0.52-m) impeller can provide excellent efficiency at a lower operating head for initial operation. If this impeller is used, the required power input is only 169 hp (127 kW) at 3,500 gpm (220 L/s). An inexpensive change to a 22-in. (0.56-m) impeller at a future date will allow relatively efficient pumping even after the distribution system ages and head loss increases. In this case, a motor changeout is avoided by sizing the motor for future conditions.

Design engineers are obligated to ensure that pumping stations will deliver desired flow in worst-case situations. However, all too frequently actual operating system curves are different. Thoughtful pump design calculations show best-case, worst-case, and most probable system curves plotted together with the performance curve of the prospective pump. When this step is taken, an efficient and stable pump can easily be selected.

Pump Testing

Routine testing is an essential requirement for efficient pump operation. Pump test data can be compared to manufacturers’ performance sheets to identify methods of improving efficiency.

Maintenance: Wear Rings, Impellers, and Packing

Proper maintenance is required to ensure efficient pump operation. Wear on impellers, casings, and wear rings can result in considerable internal recirculation of flow, which lowers efficiency. Installation or replacement of wear rings or adjustment of the impeller may enable a pump to regain its original efficiency. Improper adjustment to worn or damaged packing can result in binding of the pump shaft and loss of efficiency. Rather than continually tightening packing on a problem pump to stop leakage, operators can consider replacing the packing on a routine basis or changing to a mechanical seal.

LIGHTING

Lighting can account for 30 to 50 percent of a building’s electrical energy consumption.* Therefore, lighting energy conservation can be important even though lighting

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*This section draws from information found in publications by the Illuminating Engineering Society of North America (IESNA) (see References 5 and 6) and EPRI (see Reference 7).

represents only a small fraction of energy at a waterworks facility. Important considerations in lighting energy conservation include

- Lighting levels
- Controls
- Lamp efficacy and characteristics
- Fixture efficiency
- Ballast efficiency
- Maintenance

Lighting Levels

The practice in many water treatment plants has been to have lights burning continuously regardless of the needs of the lighted areas. Nonwork areas and yards are often highly illuminated. Electricity required for lighting could be reduced by 20 to 30 percent by reducing or eliminating lighting in nonessential areas and by reducing overillumination in work areas.

Controls

Control modifications can yield substantial cost and energy savings. Manual controls are effective if properly used. Manual control is provided by wall switches; when applied to control small groups of luminaires, these switches can permit excellent selectivity, resulting in energy and cost savings. Manually controlled dimmers are available for most types of lighting. Indoors, dimmers applied to a uniform system can generate many of the benefits of nonuniform task-oriented lighting.

Automatic controls are becoming increasingly popular. Used predominantly for outdoor lighting, they can allow week-at-a-time programming and can automatically compensate for changing hours of light and darkness. Batteries or spring-wound mechanisms maintain accuracy during power interruptions. Time clocks are also used indoors with local switch overrides.

Photocell controls are used extensively for outdoor purposes. They activate and deactivate lighting based on the amount of ambient light detected. When a photocell–time clock control is used, the time clock keeps lighting off for a certain period; during other periods, the lighting is photocell controlled. Photocells are also used indoors, particularly for lighting near windows and skylights. Some systems use one photocell to control all lighting in an area; others use one photocell per fixture. Photocell-dimmer controls increase or decrease lighting levels based on the amount of ambient lighting available to maintain a constant illumination level.

Personnel detection controls activate and deactivate lighting based on the presence or absence of people in a space. Ultrasonic controls perform this function through motion detection; passive infrared controls operate by sensing body heat; active infrared controls activate lighting when beams they emit do not return; and acoustic controls are available to detect noise and human activity. Most personnel detection controls can be integrated with dimmers, and some can be integrated with in-space heating/cooling units or motorized dampers of central multizone systems.

Several types of centralized lighting control systems are available. Many are computer controlled and allow user programming. They are ideally suited for control of all lighting (and other loads) in and around large buildings or groups of buildings.

Other types of centralized systems use existing wiring to conduct signals to actuators mounted inside fluorescent luminaires. Another type sends control signals through wireless radio transmission to receivers mounted in ballast modules.

Almost all the controls just discussed are applicable to both new and existing buildings.

Lamp Efficacy and Characteristics

In most applications for a water treatment facility, lamps can be divided into several categories: incandescent, fluorescent, and high-intensity discharge (mercury vapor, metal halide, high-pressure sodium, and low-pressure sodium). The basic characteristics of such lamps are shown in Table 30–5. In this discussion of lighting, the term *efficacy* is used to describe lamp light output divided by power input (lumens/watt). The term *efficiency* is used in reference to luminaires and is the ratio of the light emitted by the luminaire divided by the light emitted by the lamp.

Incandescent Lamps Incandescent lamps have the poorest efficacy but are popular because the fixture and lamp are inexpensive. In addition, no ballast is required to modify the characteristics of the power supply.

Fluorescent Lamps The fluorescent lamp is the most common light source in water plants. Unlike the incandescent lamp, the fluorescent lamp requires a ballast to strike the electric arc in the tube initially and to maintain the proper voltage and current to the lamp to maintain an arc. Proper ballast selection is important to optimum light output, lamp life, and overall efficiency.

Typical lamp sizes range from 40 to 125 W. The efficacy of a lamp increases with lamp length [from 4 to 8 ft (1.2 to 2.4 m)]. The reduced-wattage fluorescent lamps introduced since the mid-1990s use 10 to 20 percent less wattage than conventional fluorescent lamps.

The cool white and warm white lamps provide very acceptable color and energy efficacy ratings in most locations. New types of fluorescent lamps can produce color that is similar to the incandescent lamps or daylight but at lower efficacy. Several dimming technologies are available for fluorescent lamps. Some provide full-range dimming; others permit limited dimming only but require no modification for existing fixtures and ballasts.

Fluorescent lamp life is rated according to the number of operating hours per start—for example, 20,000 hours at 3 hours of operation per start. The greater the number of hours operated per start, the greater the lamp life. Because fluorescent lamp life ratings have increased, the number of starts is less important. If a space is to be unoccupied for more than a few minutes, lamps should be turned off.

High-Intensity Discharge Lamps *High-intensity discharge* (HID) is the expression commonly used to describe mercury vapor, metal halide, high-pressure sodium, and low-pressure sodium lamps. Each requires a few minutes (2 to 15) to reach full output. In addition, if lamp power is lost or turned off, the arc tube must cool to a given temperature before the arc can be restruck and light produced. Up to 15 minutes or more may be required for metal halide lamps.

A mercury vapor lamp produces light when the electrical current passes through a small amount of mercury vapor. The lamp consists of two glass envelopes: an inner envelope where the arc is struck and an outer or protective envelope. The lamp requires

TABLE 30–5. Comparison of Lamp Characteristics

Lamp Type	Wattage	System Efficacy (lm/W)	Lamp Life (hours)	Color Rendering Index	Start to Full Brightness	Restrike Time
Standard incandescent	3–1,500	6–24	750–2,000	95+	Instant	Instant
Tungsten halogen	10–1,500	18–33	2,000–4,000	95+	Instant	Instant
Fluorescent	40–125	50–100	7,500–24,000	62–92	0–5 seconds	Instant
Compact fluorescent	5–40	50–80	10,000–20,000	82–86	0–1 minute	Instant
Mercury vapor	40–1,250	25–50	24,000+	22–52	2–5 minutes	3–10 minutes
Metal halide	32–2,000	40–100	6,000–20,000	65–85	4–6 minutes	10–20 minutes
High-pressure sodium	35–1,000	40–140	16,000–40,000	21–80+	2–5 minutes	1 minute
Low-pressure sodium	18–180	120–175	12,000–18,000	0–18	10–15 minutes	Instant

a ballast designed for its specific use. Electronic dimming is also available. Mercury vapor lamps are most commonly used in industrial applications and outdoor lighting because of their low cost and long life (approximately 24,000 hours). The color-rendering qualities of a mercury vapor lamp are not as good as those of incandescent and fluorescent lamps. Because the color rendition and lamp efficacy of phosphor-coated mercury vapor lamps is better than that of their clear (no phosphor coating) counterparts, the development of phosphor-coated mercury vapor lamps has enabled the application of such lamps indoors. Mercury vapor lamp sizes range from 40 to 1,250 W.

A metal halide lamp is similar in construction to a mercury vapor lamp, the major difference being that the metal halide lamp contains various metal halide additives in addition to mercury vapor. The efficacy of metal halide lamps is much higher than that of mercury vapor lamps. Some of the newer metal halide lamps provide color similar to that of incandescent lamps; others emulate daylight. Metal halide lamp sizes range from 32 to 2,000 W. Ballasts designed specifically for metal halide lamps must be used.

A high-pressure sodium (HPS) lamp has the highest lamp efficacy of all lamps commonly used indoors. It produces light when electricity passes through a sodium vapor. The lamp has two envelopes. The inner envelope is made of polycrystalline alumina, in which the light-producing arc is struck. The outer envelope is protective and is clear or coated. The sodium in the lamp is pressurized; hence, the light produced is not the characteristic bright yellow associated with sodium, but rather a ‘‘golden white’’ light. Although the HPS lamp first found its principal use in outdoor lighting, it now is a readily accepted light source in industrial plants. HPS lamp sizes range from 35 to 1,000 W. Ballasts designed specifically for high-pressure sodium lamps are required.

A low-pressure sodium (LPS) lamp has the highest efficacy, providing up to 175 lm/W. It is used where color is not important, because it has a monochromatic light output—that is, reds, blues, and other colors illuminated by an LPS light source all appear as tones of gray or yellow. Low-pressure sodium lamps range in size from 18 to 180 W. Ballasts designed specifically for LPS lamps are required. These units are best used for outdoor applications; indoor applications are practical only where color is not important.

Lamp Substitution One type of lamp substitution involves a lamp retrofit, where one lamp is removed from a fixture and another is installed to increase efficacy. Another type of substitution involves replacement of fixtures themselves. Lamp efficacy is not the only criterion to be used for selection. Other important factors include color rendition, useful life, light distribution, restrike time, lumen depreciation rate, and disposability.

Fixture Efficiency

Manufacturers of lighting fixtures provide product information regarding fixture efficiency in terms of the coefficient of utilization (CU). This value is the percentage of light emitted by the lamps that is delivered to the work plane. The luminaire direct depreciation (LDD) is the amount of light lost by dirt buildup on the fixtures or lens. This reduction of light with time requires the designer to install more fixtures than would be required if light output did not deteriorate. When luminaires become outdated or damaged, the best approach is to replace the fixture with a modern, efficient unit with good cleaning capabilities, as well as excellent lumen maintenance characteristics. Simply changing the lens of a fixture can substantially increase luminous efficiency.

Ballast Efficiency

A ballast transforms line voltage and controls lamp current to match the operating characteristics of the lamp. The only lamp type that does not require a ballast is the incandescent lamp. Manufacturers have introduced high-efficiency ballasts that reduce energy consumption of fluorescent fixtures by as much as 9 percent. These ballasts have a longer service life than the older design and can provide an overall savings of as much as 27 percent when used with high-efficiency lamps. This energy savings is produced with only a small loss of light. Other ballasts will reduce energy consumption by 20 percent when used with standard lamps, but with a corresponding decrease in light output. They should be used only where reduced lighting is acceptable.

Maintenance

Lighting systems are poorly maintained at most facilities, with consequent waste of both money and energy. Lamp output for most lighting systems decreases with use. Relamping may be cost-effective before lamps burn out. Consideration should be given to group relamping to reduce maintenance, storage, and energy requirements.

WATER CONSERVATION

The primary focus of water conservation has often been on reducing the impact on limited raw water supplies, treatment plant capacity, and distribution system capacity. However, when energy savings are considered, water conservation can take on a new meaning. With the exception of plants that have gravity-fed distribution systems, water-demand reduction can correspond to significant savings in energy required for pumping.

Many water utilities have mounted effective water conservation programs to reduce water consumption at the end use. The greatest source of water waste in most communities is in landscape irrigation. Proper selection of plant materials and irrigation equipment, as well as good irrigation scheduling, can significantly reduce water use. Some of the more innovative programs include training for landscape architects and publishing of evapotranspiration data so irrigation rates can be adjusted. Large users should be encouraged to conduct a landscape irrigation audit to assess current water uses and associated costs.

Significant water and energy savings can be achieved by optimization of industrial processes, consumer education, water recycling, and installation of low-flow pumping fixtures. Many of these measures reduce sewer flow, which saves pumping energy at the local wastewater utility.

A water utility could also consider installing meters, if not already in place, for monitoring domestic accounts. Although a decision to implement metering in a previously unmetered area may generate disapproval from water customers, the increased savings may prove to be a financially viable alternative in terms of annual cost savings.

In addition to implementing conservation at the tap, water purveyors should pursue efforts to identify losses in the distribution system on an ongoing basis. A survey of California water utilities showed that unaccounted-for water ranged from 2 to 30 percent of all water use.[12] Unaccounted-for water often includes losses from malfunctioning or inaccurate water meters, leaks from the distribution system, overflows from storage tanks, releases from pressure relief valves, and various unmetered uses.

Experience indicates that utilities should try to restrict their unaccounted-for water to no more than 5 percent, although many utilities set a target of 10 percent. In service areas where the infrastructure is relatively new and water is becoming more precious, prudent management is needed to maintain the integrity of water distribution systems and strive for low system water losses.

A leak-detection program can identify distribution system losses. A water utility may need to invest in additional meters to monitor where the major demands or losses are occurring in the system. Leaky pipes and fittings can cause water losses of 10 to 50 percent within a distribution system.

A utility should track water consumption regularly and flag any suspected higher-than-normal demands in order to identify potential problems. Spillage from tanks or a float control valve stuck in the open position on a reservoir can go unnoticed and generate a considerable demand. Localized leaks from buried pipes often show up as wet or swampy areas in unexpected locations. A faulty check valve on a well can result in a backflow from the system into the ground.

Example

Consider an example involving a 10-mgd (38-ML/d) plant where water conservation yields a 10 percent (1-mgd, or 3.8-ML/d) reduction in demand. Assume the discharge pressure is 60 psi, the wire-to-water efficiency is 70 percent, and energy costs are \$0.07/kWh. The horsepower required for this 1-mgd demand can be calculated as follows:

$$\text{horsepower} = \frac{QH}{3{,}960 \times \text{efficiency}}$$

$$= \frac{1\text{ mgd} \times \dfrac{694\text{ gpm}}{\text{mgd}} \times 60\text{ psi} \times \dfrac{2.31\text{ ft head}}{\text{psi}}}{3{,}960 \times 0.70} = 35\text{ hp}$$

where:

Q = flow, gpm
H = pump differential pressure, ft

By converting horsepower to kilowatt-hours, the resulting annual savings can be determined:

$$\text{annual savings} = 35\text{ hp} \times \frac{0.746\text{ kW}}{\text{hp}} \times \frac{24\text{ hours}}{\text{day}} \times \frac{365\text{ days}}{\text{year}} \times \frac{\$0.07}{\text{kWh}} = \$16{,}000/\text{year}$$

REFERENCES

1. Fay, J. W. ‘‘Applying Capacitors to Improve Operations,’’ *Electrical Consultant*, July/August 1982.
2. WPCF (Water Pollution Control Federation), *Energy Conservation in the Design and Operation of Wastewater Treatment Facilities,* Manual of Practice No. FD-2, Lancaster, Pa., Lancaster Press, 1982.
3. Gumerman, R. C., Culp, R. L., Hansen, S. P., and Lineck, T. S., *Estimating Water Treatment Costs,* Vols. 1–4. EPA-600/2-79-162a, 162b, 162c, and 162d, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1979.
4. EPRI (Electric Power Research Institute), *Energy Audit Manual for Water/Wastewater Facilities,* EPRI’s Community Environmental Center, St. Louis, Mo., 1994.
5. IESNA (Illuminating Engineering Society of North America), *Industrial Lighting,* Recommended Practice, RP-7, IESNA Publications Department, New York, 1991.
6. IESNA (Illuminating Engineering Society of North America), *Office Lighting,* Recommended Practice, RP-1, IESNA Publications Department, New York, 1993.
7. EPRI (Electric Power Research Institute), *Quality Energy Efficiency Retrofits for Water Systems,* Report CR 107838, Collaborative effort with California Energy Commission, CEC Report P400-97—003, EPRI, St. Louis, Mo., 1997.
8. NEMA (National Equipment Manufacturers Association), *Motors and Generators,* Publication MG-1-1998.
9. USDOE (United States Department of Energy), Buying an Energy-Efficient Electric Motor. Motor Challenge Fact Sheet, USDOE, Washington, D.C., n.d.
10. Doffer, R. A., and Price, J. M., ‘‘Pump Systems: What Price Inefficiency?’’ *JAWWA* 72:338, 1980.
11. Patton, J. L., and Horsley, M. B., ‘‘Curbing the Distribution Energy Appetite,’’ *JAWWA* 72(6), 1980.
12. CDWR (California Department of Water Resources), *Water Audit and Leak Detection Handbook,* CDWR, Sacramento, Calif., 1986.

CHAPTER 31

Electrical Design Considerations

INTRODUCTION

Electrical service is an integral part of all water facilities and must be planned as the water system is developed. This chapter provides general information on electrical design considerations that will be useful when planning or designing water facilities.

ELECTRICAL PLANNING

Planning for effective and efficient electrical systems involves a team effort among planning, design, operation, and maintenance staff, and the serving electric utility. It is important for water system planning and design staff to understand basic electrical considerations, including needs and impacts. Electrical impacts must be recognized early and incorporated into the planning stage.

Some of the important impacts are:

- Loads and resultant voltage level
- Serving electric utility and available electrical service
- Auxiliary power systems
- Support systems, which include telephone, fire alarm, security, etc.
- Site layout
- Major electrical equipment and space
- Hazardous and corrosive areas

LOADS AND VOLTAGE LEVEL

The first major step in electrical planning is to estimate the loads requiring electricity and then determine optimum voltage level(s). Loads usually consist of the following major components for most facilities:

- Motors for pumps and other process-related equipment
- Lighting

- HVAC, including any electric heating and/or air conditioning
- Future needs

As a general rule, the magnitude of load will determine the voltage level. Most small to medium facilities are served at 480 volts, three phase. Very small facilities with little or no motor load and little requirement for future expansion might be served at 120/208 volts, three phase, or even at 120/240, single phase. Large facilities with larger motors or facilities scattered over an extensive site may be served at a medium voltage ranging from 4,160 up to 13,800 volts, three phase. Once estimated loads are determined for a facility, the serving electric utility should be consulted to determine what voltage levels are available.

Unless specific information is available, initially it will be necessary to estimate loads. It is desirable to estimate both a connected load (the total load) and a demand load (the estimated maximum load on line at any one time). Loads should be calculated in kilovolt-amperes (kVa) and each motor over 10 HP should be identified.

Process loads, including pump motor sizes and other miscellaneous motor loads, should be estimated in consultation with the process staff. A rough estimate initially is better than no estimate, but be realistic and conservative.

Motor horsepower can be converted to kVa as follows:

$$\text{kVa} = \frac{(\text{motor full load or running current})(\text{supply voltage})(1.732)}{1{,}000}$$

An initial estimate can be based on kVa = motor HP.

The data in Table 31–1 will be useful for preliminary estimates unless better data are available.

Future loads and contingencies should be added to estimates. Estimation of future loads must be done based on conditions specific to the situation.

TABLE 31–1. Electrical Load Estimation

1. Total motor load (pumping) HP $= \dfrac{(\text{Head, ft } H_2O)(\text{Total Flow, gpm})}{3{,}960\ (\text{pump efficiency})}$		
2. Special loads must be identified and estimated or quantified.		
3. Outlets		
	Office areas	2.0–3.0 watt/ft^2
	Utility areas	1.0 watts/ft^2
4. Exterior lighting		
	Parking	0.1–0.2 watt/ft^2 of parking area
	Roadway	3.0 watts per lineal foot of roadway
5. Interior lighting		
	Office, lab, etc.	2.5 watt/ft^2
	Utility areas	1.5 watt/ft^2
6. HVAC		
	Office, ventilated and air conditioned	5.5 watt/ft^2
	Office, no air conditioning or electrical heat	0.5 watt/ft^2
	Utility area, ventilation and electric heat	0.5 watt/ft^2

A contingency of at least 15 to 30 percent should be added to all preliminary estimates, depending on the situation and the confidence in the load data used. Loads usually increase as facility planning and design progresses.

A typical facility might be served at a voltage shown in Table 31–2.

SERVING UTILITY AND AVAILABLE ELECTRICAL SERVICES

Electricity may be purchased from the serving utility at either a primary rate or a secondary rate. A primary rate would meter the power taken ahead of any transformer. The secondary rate would meter the power after a transformer. Primary meter rates are lower than secondary meter rates; however, there are details involved in primary meter rates that are beyond the scope of this chapter. Electrical rate schedules on large facilities requiring large amounts of electrical energy are negotiable. Determining the cost of electricity will be a large factor in the feasibility of a project.

Generally, the serving utility will want to know the connected load, demand load, largest motor sizes, and desired point or points of service. Most utilities will work with customers in planning the location for the service facilities. For small to medium plants with loads up to 2,000 to 2,600 kVa, the typical service would consist of a takeoff point for utility medium voltage (usually a utility power pole), an underground medium voltage line to a pad mount transformer, and an underground 277/480 volt, three-phase service. The customer usually furnishes and installs the conduit for the medium voltage conductors, the pad or pad-vault and grounding for the pad mount transformer, and the conduit for the secondary service—all to utility standards.

The utility usually furnishes and installs the medium-voltage conductors, the pad mount transformer, and the secondary conductors and makes all connections up to the billing meter, but this can vary depending on the utility. The customer service equipment must meet utility standards for metering, and the utility will install the meter.

The typical pad mount transformer will require a ground space from about 5 feet by 5 feet up to about 10 feet by 10 feet.The transformer must be accessible for service, must have 8 feet clearance in front of the doors, usually must be 3 to 5 feet from structures, and will require protection from vehicles using bollards.

In some cases the serving electric utility will provide service at prime on voltage (4,160 to 34,000 volts) for large facilities. These are special cases that require close coordination with the serving utility.

AUXILIARY POWER SYSTEMS

Auxiliary and standby power is an essential element of most facilities. If continuity of service is needed on loss of normal power, some type of standby power will be

TABLE 31–2. Service Voltages

Demand Load, Including Future, kVa	Largest Motor, HP	Typical Service Voltage
2,600	500	277/480 volt, 3 phase
Over 2,600	3,000	2,400/4160 volt, 3 phase*

*With loads over 2600 kVa and where load centers such as buildings and pumping stations are spread over a larger site such as a large water treatment facility, it may be necessary to distribute power at medium voltage, to several load centers located at the major site loads.

required. In most areas of the country, a second power feed from the serving electric utility can serve as a standby power source. However, unless the facility is extremely large, or a convenient second primary feed is available, the capital cost for this will be prohibitive. The other general form of standby power is on-site power generation.

There are code definitions of standby power that describe auxiliary power systems: emergency power, legally required standby power, and optional standby power. Emergency power requires the power to be available and on-line within 10 seconds. Legally required standby power is required to be available and on-line within 60 seconds. Optional standby power does not have any time constraints. Chlorine scrubber facilities usually require emergency power. All other facilities would normally fall under legally required or optional standby power.

Standby power generation usually takes the form of an engine-driven generator using either diesel, natural gas, or liquid petroleum gas (LPG). The most common and least expensive is a diesel engine-driven generator. Units are available for indoor or outdoor installation. Usually the generator is connected to the facility electrical system using an automatic transfer switch that will automatically start the generator and transfer load to the generator upon loss of utility power, return to utility power when it is restored, and stop the generator.

Fuel storage for diesel-powered generators can be provided by underground or aboveground tanks or by tanks built into the base of the generator.

If underground fuel storage is required, the cost of the diesel engine–driven generation system will be increased substantially and an alternate fuel, such as natural gas or LPG, if available, may be more cost-effective. Above-grade diesel fuel storage is cost-effective and can be accomplished with standard equipment. Tanks built into the base of the generator are common and provide up to about 24 hours of operation in storage. If the generator is to be installed indoors, the in-base fuel storage capacity may be limited by code and an outdoor tank may be required. The local fire authorities should be consulted when planning indoor fuel storage. The selection of generator fuel (diesel, natural gas or LPG) should consider the following:

- Cost of unit
- Availability of the fuel under emergency conditions
- Fuel storage requirements and costs
- Air quality requirements and codes

An alternative power system is the use of engine-driven equipment, usually a pump, using either natural gas or diesel as a fuel source. The engine-driven pump may be direct-driven, or operated through a right-angle drive, with an electric motor used to normally power the pump.

SUPPORT SYSTEMS

There are a number of support systems to be considered in design of water facilities. Many are code requirements and will vary from location to location. This section will serve as a checklist of items to consider. Local code authorities should be consulted during design for particulars.

1. *Telephone system.* Most facilities will require provisions for telephone utility service and for telephone wiring within the facility. Usually the local telephone company will provide telephone service to a facility main telephone terminal board in a customer-furnished conduit.

The customer must determine the type of telephone system needed. This is best accomplished by determining the present and planned future uses and needs for telephone and data services and then discussing these uses and needs with suppliers of the systems.

The customer is responsible for telephone and data equipment and wiring within the facility.

2. *Fire detection and alarm system.* These are specialized systems that are usually regulated by local fire authorities. In most cases, smoke/heat detectors will be required. In a ducted HVAC system, a smoke detector(s) may be required in the duct system. Where a sprinkler system is installed, special monitoring equipment for the sprinkler system riser will be necessary. In many cases, manual breakglass stations will be required. Some authorities may require facilities to turn off ventilation, electrical service, and standby generation from the outside of the building. Each system is different, and it is wise to obtain a clear determination of the requirements during design to avoid later delays in occupying the facility.

Usually, the fire detection apparatus is wired to a fire alarm control panel that automatically reports to an alarm service company via telephone.

3. *SCADA and computer systems.* Where Supervisory Control and Data Acquisition Systems (SCADA) are to be installed, provisions may be required for communications between various components in the system, such as from Programmable Logic Controllers (PLCs) to the SCADA central terminal. This communication will usually take the form of hard wire, such as twisted and shielded wires or coaxial cable. Fiber-optic cable is becoming common because it provides a communication system immune to electrical noise. Usually a conduit system will be required for the communications. In some cases, a radio may be used and antennas will be required as the communications link.

If computer use is anticipated, computer outlets and network wiring should be planned as part of the construction. In many cases, this would be coordinated with telephone wiring and joint use outlets would be provided. Special power outlets may be needed for computers that are served by uninterruptible power supplies (UPS).

4. *Security.* Security systems may be required, including motion detection, TV surveillance, code pads for entrance and exit, and similar facilities. These facilities will require conduits for the specialized wiring required.

SITE LAYOUT

Site layout for electrical facilities will vary, depending on the extent of the facilities to be served.

For a single facility, space allowances will normally be required for a utility pad–mounted transformer with a primary conduit to the utility primary system and secondary service conduit(s) into the facility. Usually a telephone service conduit will be required from the telephone utility to the telephone terminal point in the facility. Exterior conduits or duct banks may be required for site and parking lighting, security gates or checkpoints, and for surveillance equipment.

An outdoor standby generator or fuel storage facility will require appropriate space and truck access.

Distributed facilities on a site may require an extensive system of underground duct banks and pullboxes or manholes. Space must be allocated for such systems as site utility systems are planned.

MAJOR ELECTRICAL EQUIPMENT AND SPACE

Major electrical equipment can be quite large and requires suitable space and environment. It is wise to plan dedicated space for electrical switchgear, motor control centers, and panels—preferably in a separate room from the process where possible. Obtaining realistic dimensions for electrical equipment will aid greatly in the planning process. There are National Electrical Code requirements for working space clearance and entrances into the work space, and it is suggested to review these requirements with an eye toward the size of electrical equipment expected.

Switchgear and motor control centers can be mounted outdoors in NEMA 3R raintight enclosures on concrete pads where indoor space is limited. Maintenance can be more difficult in cold, rainy or snow climates. In addition, electronic components such as VFDs may require heating and/or air conditioning. It is always better to install such equipment in buildings where possible.

HAZARDOUS AND CORROSIVE AREAS

Steps should be taken to avoid installing all but essential electrical and electronic equipment in hazardous (NEC-classified areas) and/or corrosive areas. If available, equipment should be installed in adjacent nonclassified and/or cleaner areas. Electrical equipment and its maintenance and service will be more costly and difficult when installed in hazardous and/or corrosive areas.

Hazardous areas require explosion-proof-rated equipment, which usually consists of bolted, heavy enclosures that make maintenance and inspection difficult and tend to discourage these essential activities.

On water treatment facilities, generally there are not any areas that are required to be rated hazardous with respect to fire or explosion potential. Fuel-dispensing or -storage areas would be a possible exception. Another area of concern is the storage of powdered activated carbon (PAC). Local code officials have had differing views on whether the carbon dust poses an explosion hazard. It is best in this instance to get a ruling from the local code enforcement agencies prior to beginning design using this chemical.

Any areas that store chlorine, or chlorine-based derivatives, should be classified as corrosive.

INSTRUMENTATION

Almost every water system or facility can benefit from the use of a SCADA system. SCADA systems are applicable for small water facilities with less than 100 input/output points to large facilities with a 1,000 or more.

A typical basic plant SCADA system will include:

- Central SCADA system consisting of operator terminal(s), data archiving, SCADA server(s), communications interfaces, and printer(s). A simple system may consist of one PC-based terminal that combines the functions of operator terminal, SCADA server, and data archiving. Larger systems may consist of a number of PC-based terminals with dedicated terminals for SCADA server and data archiving functions and multiple operator terminals, all interconnected on a local network such as Ethernet.
- Communications between central SCADA and the field system, which can be radio, wireline, telephone, or fiber-optic.
- Field system, which will typically consist of intelligent devices such as Programmable Logic Controllers (PLCs) or specialized Remote Terminal Units (RTUs). Both will communicate with the central SCADA and receive data (input–output) from field devices. Most will have capabilities to be programmed to perform field analog control strategies and digital (on–off) control.
- Field wiring is needed between field devices and the input–output of the PLC or RTU.
- Field devices and instruments consist of a variety of signals and devices, such as run contacts in motor control center starters, motor ready contacts, motor in auto or manual contacts, pressure switch contacts, float switch contacts, analog pressure transmitters, analog level transmitters, and similar devices that provide contact closures (digital) or analog signals to a PLC or RTU.

Some field devices now have enhanced communications capabilities and intelligence that allows them to communicate directly with a SCADA central or directly with a PLC or RTU through a communications port rather than through an input–output module.

A small, isolated facility such as a pumping station may have only a local PLC for data collection and control, with a local panel-mounted operator interface such as an LCD indicator and touch pad for communication with the PLC. The interface to the PLC can also be a laptop computer plugged into a communications port on the PLC. Many isolated facilities may have no direct communication to a central SCADA terminal.

A more extensive water facility—such as a water treatment plant, even a small plant—can benefit from a plantwide SCADA system. Typically, this system will consist of one or more PLCs or RTUs out in the plant for local process control and monitoring that will communicate with a central SCADA terminal. Typical communications within a plant site will be wireline or fiber-optic and sometimes radio. Facilities off the plant site, such as pumping stations and reservoirs, can also be a part of the same SCADA system by communicating over telephone or radio.

A SCADA system can be tailored to the needs of the facility or facilities. Typically, it will have a number of in-plant process control centers within a larger facility consisting of one or more PLCs or RTUs. The central control area can consist of multiple SCADA operator terminals at various locations on a local network such as Ethernet. Terminals can be in the main control room, laboratory, programming room, and other areas as desired. It is also possible to place complete operator terminals out in plant process control centers as needed. The possibilities are almost unlimited.

For a vast majority of water facilities, the central system will consist of personal computer hardware running on a Windows-based SCADA software package such as Intellution FIX, Rockwell RS View, Wonderware, or Factory Link. These software packages contain drivers for most PLCs and RTUs; high-resolution user-configurable graphics that can display real-time data and perform data trending; report generators, alarm processing, and other functions. In addition, the SCADA can communicate with other Windows-based software, such as word processing and spreadsheets, using dynamic data exchange (DDE).

Large facilities may be served by much more complex SCADA systems or by sophisticated Distributed Control Systems (DCS), which are more integrated (and usually proprietary) computer-based plant control systems.

The capabilities of a modern SCADA system eliminate the need for plant graphic panels and most plant indicators, recorders, controllers, and annunciators.

CHAPTER 32

Economic and Financial Issues

ECONOMIC EVALUATION OF ALTERNATIVES

A common issue facing utility managers is the choice among competing alternatives, whether it is capital improvements, program changes, or even policy considerations. There are several economic and planning evaluation techniques that allow for a direct ‘‘apples-to-apples’’ comparison among alternative approaches. Some evaluation techniques are narrowed to a comparison of costs, while other methods can incorporate consideration of benefits, revenue or other income, and nonquantitative factors such as environmental impact.

In choosing a suitable method, the analyst must first determine the extent of issues involved in the evaluation. For instance, if the various alternatives to be considered provide a comparable level of service performance and similar environmental impact, then a more straightforward comparison on costs alone may be relevant. However, if the options have more distinct features that would allow for greater or lesser sales of service or produce a varying array of impacts, a more comprehensive pro forma, benefit–cost, or multi-objective evaluation approach may be more fitting.

Further, with each evaluation approach, there are differing degrees of analytical complexity that can be undertaken. The following sections outline less complex evaluation methods that are generally suitable for early planning assessments.

Basic Planning and Financial Concepts

In order to better understand and interpret the results of the various economic evaluation methods, a review of some basic approaches and planning and financial concepts is warranted.

Analytical Tools A powerful tool for conducting an economic evaluation of alternatives is using an electronic spreadsheet software package that can easily do the math; has built-in features for automatically calculating financial statistics such as amortization, present and future value amounts; and can produce tables and graphs that provide informative backup and interpretation. Another important reason for using an electronic spreadsheet in economic evaluations of alternatives is the ability to conduct ‘‘what-if’’ sensitivity analyses of varying key parameters (such as inflation rates, interest rates, and power costs) and quickly assessing the effect of that change on an alternative’s economic performance. With additional time and effort, these analyses can also be performed using financial or standard calculators or by hand.

Delineation of Planning Scenarios An initial planning consideration is determining which and how many alternatives are to be evaluated. However, in many cases, a particular alternative or course of action may require several associated sub-actions over time. For example, Option A may entail the first improvement occurring in year 1, a second expansion phase in year 3, and rehabilitating still other equipment in year 5. Alternatively, Option B may accomplish all of the improvements in year 1 and defer the rehabilitation until year 7, but require more operational staffing and maintenance on a year-to-year basis. It is these groups of related actions, or *planning scenarios,* that serve as the more appropriate and comprehensive basis for comparison among alternative courses of action.

One scenario that is always valuable to consider is the ‘‘no action’’ future, or what are the implications of no new significant action. The ‘‘no action’’ scenario serves not only as a benchmark for comparison to other action-oriented scenarios, but also as a tool for the utility manager to communicate the consequences of not taking any action to higher-level decision makers or the public.

So how many other action-oriented planning scenarios should be considered? A variety of technological, cost, political, environmental, or regulatory factors may determine that answer, but the analyst should attempt to keep the range of alternative planning scenarios to a reasonable few options that bound the major choices or issues facing the decision maker(s). Some alternatives may exhibit obvious significant problems and should be considered early on for elimination from further consideration.

Planning Period Most capital or program decisions faced by a utility manager have both near- and longer-term consequences; therefore, a good assessment must consider the implications of a particular action over a period of time. Thus, a second consideration in the economic evaluation is the appropriate length of the planning period. The period planning should not be so brief as to mask the effects of any significant longer-term issues, nor so expansive that the longer-term forecasts have little consequence on the answer or undermine the credibility of the analysis.

For major capital investments, a good rule of thumb is to examine a sufficient future time frame that, at a minimum, represents the ‘‘life of the bonds’’ used to finance the improvements (usually 10 to 30 years). A broader time frame may be appropriate if there are significant useful service life, rehabilitation, depreciation, or salvage value issues to be considered beyond the period of financing. For less capital intensive program improvements, the time frame for evaluation is typically more brief and may extend from a 3- to 10-year planning horizon.

Annualized Costs In order to assess alternative scenarios on a comparable basis, a next step is to describe the cost implications of an option on an annualized basis over time. Two concepts are key to this aspect of the analysis: the debt financing of capital improvements, and ongoing operations and maintenance (O&M) expenses.

More significant utility actions will usually involve the debt financing of major capital improvements, typically with the sale of revenue bonds whereby the revenue of the utility system is pledged to the repayment of the debt. Knowing the correct details of the debt financing is useful early in the economic evaluation stage. However, the particular financing approach and terms may not be fully known until much later, when the borrowing package is nearing sale, where a variety of short- and longer-term financing combinations—interest-only, front- or back-loading of principal, and so on—may eventually be utilized.

In most earlier economic evaluations, an amortization calculation will usually suffice to convert the capital debt amount of an alternative into a fixed annualized payment of principal and compound interest. A first step in this process is to determine the amount to be financed. Typically, the amount to be financed will include the estimated costs of final engineering design and permitting, construction, and inspection expenses, less any cash contribution or grant funding, and then increased by the cost of bond issuance (generally an additional 1 percent to 5 percent more). The resulting total amount to be financed is then amortized into a series of equal payments made at equal periods of time based on a constant interest rate that ultimately discharges all principal and interest liabilities of the debt.

The simple amortization formula that produces annualized debt service payments is:

$$\text{PMT} = A \times \frac{r}{1 - (1 + r)^n}$$

where:

PMT = the annualized constant debt payment comprised of principle and interest components,
A = the amount of original debt,
n = the number of annual payment periods, and
r = the annual interest rate as a fraction.

A table of amortization factors is provided in Appendix A. By selecting the appropriate interest rate and term of the loan, the resulting amortization factor can be applied to the original amount of the debt to yield a constant annual debt payment that would extend for the life of the loan.

A second consideration in the economic evaluation is projecting annualized operations and maintenance (O&M) expenses related to the alternative. In many cases, the majority of these types of expenses can be related to staffing, electric power, chemicals, or replacement parts with the annualized values potentially varying over time primarily due to changes in level of use and/or inflationary pressures. In today's dollars, real variable costs (such as electric power and chemicals) can be predicted based on engineering efficiencies, levels of use, and unit costs. Real fixed costs, such as levels of staffing, may not change or change only periodically with significant changes in the level of operations.

Aside from real underlying changes in the level of use or staffing, both fixed and variable expenses may also be affected by inflation over time, increasing the nominal value of future operating expenses above the value of the base or current year. Inflation is usually defined as a general rise in price levels over time, affecting a whole variety of goods and services and thus all alternatives relatively the same. If this is the case, inflationary effects will not significantly affect an economic comparison of alternatives. However, a more careful treatment of inflation may be needed if certain expenses or key features of the alternatives being considered are subject to differing rates of inflation. For instance, one wastewater conveyance alternative may rely on a more electric-intensive pumping approach that could be subject to a higher rate of inflation over time than another option that relies more on gravity flow.

A simple formula that projects the annualized, compounded effect of inflation is:

$$\mathrm{FV}_n = \mathrm{PV}_1 \times (1 + i)^n$$

where:

FV_n = the future value or inflated amount of expense,
PV_1 = the present value or base year amount of expense,
n = the number of the period after the initial base year, and
i = the annual inflation rate relevant to that particular expense as a fraction.

Appendix A provides look-up values for these inflationary factors.

After calculating annualized capital debt and O&M expenses, the two values can then be added to produce a projection of overall annual cost for each of the individual years in the planning period for each planning scenario.

Present Value Each planning scenario can produce a unique series of annual costs over time, with some options having higher costs in the near term and others presenting higher costs later on. To place each option on a comparable basis, the present value of each year's annualized costs can be calculated using the following formula:

$$\mathrm{PV}_n = \frac{\mathrm{FV}_n}{(1 + r)^n}$$

where:

FV_n = the future value or inflated amount of year n expense,
PV_n = the present value of year n expense,
n = the number of the period after the initial base year, and
r = the annual discount rate (i.e., the time value or preference for money).

Appendix A provides look-up values for the discounting factor. Once the present value of each year's annualized cost is determined, these can then be summed to determine the *net present value* (NPV), or cost, of that planning scenario.

Applications

Life Cycle Costing Analysis Life cycle costing (LCC) is the comparison of alternative investments using the entire cost of owning and operating the equipment. The objective of LCC analysis is to choose the most cost-effective approach from a series of alternatives so the *least long-term cost* of ownership is achieved rather than only choosing the lowest first cost.

Using the above simple methods, one can project the expected costs and present value of those costs over the life cycle of a capital improvement. One should also consider any salvage value of the equipment as a negative cost (i.e., revenue) in the analysis at the end of the planning period. If costs are basically the only consideration and the various options are comparable in service levels, the net present value of each option can be directly compared to other alternatives to ascertain the one with the lowest net present value or cost. More sophisticated LCC techniques can involve simulation of effects of maintenance levels on equipment reliability, useful life, and overall maintenance costs over time.

If the alternative projects have differing service capacities, the analyst can divide the annualized cost by the equipment capacity to yield a unit cost of service (cost per gallon, cost per MGD, etc.) for any given year. Then the annual unit costs can be converted to present value terms and summed across the years to a net present value total that can be compared with similar statistics for the other alternatives.

Pro Forma Analysis A financial pro forma analysis is appropriate if the alternative projects being contemplated can be considered investment options where a return (earnings, revenue, income, etc.) is garnered over time in return for the investment of up-front capital and/or ongoing O&M costs. A pro forma analysis may encompass a more brief period of time than the life cycle cost method, but essentially extends the annualized cost concepts related above in the LCC method with the additional consideration of projected revenue or income stream and the calculation of a net income after expenses.

The analyst should first determine the appropriate planning period for the investment. Once determined, the expense aspects of the investment should be delineated as previously described for calculating annualized capital and operating costs. Then the potential revenue associated with the investment should be forecast on an annual basis over the planning period, in most cases using an assumed price level (utility rate, fee level, etc.) and the same assumed service levels utilized in forecasting annual O&M expenses. In some cases, the positive cash flow may include more than pure service-related revenues, such as property tax revenues (in special water districts), fee receipts, grant proceeds, or interest earnings.

Once annual service revenues or other income are forecast on an annual basis, these should be summed to arrive at total annual revenue. Then annualized total expenses are subtracted from annual total revenues to determine the net annual income. The net annual income can then be converted to present value terms and summed to arrive at the net present value (NPV) for that alternative or the value of that investment in today’s dollars.

However, it is unlikely that any two alternatives will involve the same level of investment and the NPV statistic may not be useful for comparing among alternatives. In this case, another related statistic, the *internal rate of return* (IRR), provides a more consistent basis for comparison. The IRR is the discount rate at which the NPV is equal to zero. The calculations to determine the IRR are very tedious and involve an iterative process of testing various discount rates for their effect on producing an NPV equal to zero. Use of an electronic spreadsheet or financial calculator, with such built-in functions, is highly recommended. Once the IRR is determined for an alternative, this percentage value can be contrasted to various other alternatives, including the no action future. For example, a manager has the option of not funding a new project and instead continuing to earn 6 percent interest on invested funds. If the IRR of the new project is less than the current rate of return—say, 3 percent instead of 6 percent currently being realized—then the new investment is not financially attractive. If the IRR is equal to the desired rate of return, then the investor is indifferent from a financial perspective. If the IRR of the new project is greater, the investment is more financially attractive than continuing the no action option. The internal rates of return for all alternative projects that exceed the return for the no action future scenario can then be compared to determine which option provides the most attractive rate of return.

The analyst is cautioned that sole reliance upon a rate of return calculation may not be an adequate basis for evaluation of an option or comparison among alternatives. For instance, one alternative may have substantially more risk than another in producing the anticipated revenue stream. If varying risk is an issue, the analyst can apply the statistical concept of *expected value* to the revenue or cost in question. This essentially involves an estimation of the probability of occurrence applied to the value that is suspect. The analyst may feel that there is a 75 percent chance that service sales will materialize at the anticipated level of 100 units with one option, but a 90 percent chance that sales will reach target with another alternative. The expected value of the sales in the first case is 75 units (75 percent × 100 units), whereas in the second case the expected value of sales is 90 units. The expected values of the various cash flows are then included as adjusted annualized revenue or expense values and used to then calculate the net annual income, NPV, or IRR of that option.

Benefit–Cost Analyses In addition to an internal financial assessment, economic effects on outside parties or other, more indirect, factors may also be relevant considerations and warrant an even broader analysis of its major effects on society. In some cases where these other major issues can be monetarily quantified, the benefit–cost method may be the analytical method of choice. For example, a more narrow analysis of a reclaimed water system may indicate that is barely financially viable from purely financial point of view. However, a broader benefit–cost approach may indicate greater societal benefits from the option through avoided costs (i.e., savings) of developing a more expensive replacement water supply or avoided costs of additional wastewater treatment capacity or quality. Some projects may also have identifiable cost impacts to other third parties.

Similar to the pro forma assessment, quantifiable direct and indirect benefits for each year of the planning period are summed to a total, as are the array of quantifiable direct and indirect costs. Using the same discount rate, the annual values of total benefits are converted to present value amounts and summed to a net present value of benefits ($\mathrm{NPV}_{\mathrm{B}}$). Similarly, the net present value of costs ($\mathrm{NPV}_{\mathrm{C}}$) is also calculated.

Two useful evaluation statistics can be developed from these values. By dividing the $\mathrm{NPV}_{\mathrm{B}}$ by the $\mathrm{NPV}_{\mathrm{C}}$, the *benefit/cost* (B/C) ratio or value is determined. If the value is greater than one, society incurs greater benefit than cost from taking the proposed action. One can also compare the B/C ratios of various alternatives to ascertain which is the most cost-effective for society, although this comparison can be misleading when making a decision on how to proceed.

In most cases, affordable alternatives should be selected on the basis of their ‘‘excess benefits’’ rather than the benefit/cost ratio. For example, an alternative may have a high benefit/cost ratio of 3:1, but may involve only a small present value investment of $100,000 and realize present value benefits of only $300,000. In this case, a net gain of $200,000 is realized by society. However, another alternative may have a lower benefit/cost ratio of 2:1 in costing $1 million and producing $2 million in benefits. However, the second option produces $1 million in ‘‘excess benefits,’’ or five times the net benefit to society than the first alternative.

Multidisciplinary Analysis In some cases, there may be a variety of relevant issues—some of which are monetarily quantifiable, some quantifiable with different measures, and other factors that may only be qualitatively or more subjectively described. A typical approach is to develop a multidisciplinary evaluation matrix, where a table is constructed with the alternatives listed by row and the columns describe the evaluation criteria (or vice versa). A monetarily quantifiable measure, such as the IRR, can be listed on a row for each measure to identify its internal investment potential. Other monetarily quantifiable but broader measures, such as the benefit/cost ratio or excess benefits amount, can be listed on a separate rows for each option. Then, on the next several rows, other quantifiable impact measures (which are less prone to be monetized) can be listed, such as the number of acres of wetlands impacted or the number of residential structures within a 1,000-foot radius of the project site. Finally, other issues that may be relevant, such as visual aesthetics, can be very subjective. For each option, these more subjective issues might be described through a scale index (example: 1 [unsightly] to 10 [pleasing]), through some broader qualitative description (such as low negative impact or moderate benefit), or by a general narrative.

Once this is done, it may also be possible to assign ‘‘weighting factors’’ to the evaluation criteria to reflect the analyst’s or decision maker’s placement of relative importance on each of the criteria. The multiplication of the weighting factor to each quantified measure produces a weighted score for that criterion for that alternative. Then the weighted scores for each criterion is summed across the alternative to arrive at an overall weighted score for that alternative. The weighted scores for each alternative should then be compared.

With the multidisciplinary evaluation approach, the ultimate decision maker(s) are presented with an array of categorized, and generally comparable, data for each alternative, which allows a more informed, and perhaps better, decision to be made.

FUNDING SOURCES AND MECHANISMS

Providing high-quality water and wastewater infrastructure for existing customers requires a huge outlay of funds. However, many utilities face the *additional challenges* of financing water and wastewater capital improvements for growth-related demands, and the additional capital costs for existing and new customers to provide for compliance with Safe Drinking Water Act and other federal and state regulations.

Decades ago, many municipally owned utilities used tax revenues to fund the issuance of General Obligation debt to finance capital infrastructure for both general and utility services. In the postwar years, with the advent of separate municipal enterprise funds, utility rate revenue became the basic pledge for issuance of municipal revenue bonds for utility capital. To this day, utility rate revenue is the primary means of gathering significant capital funds for most utilities.

Primarily beginning in the 1950s and 1960s, the federal government and many state governments began various successions of grant and low-interest loan programs for water and wastewater infrastructure financial assistance. The availability of federal grant funding has grown and shrunk over the years, and for the most part has been replaced by low-interest revolving loan programs capitalized with a combination of federal and state funds. Most remaining grant funding today is typically narrowly targeted at economically distressed or disadvantaged communities. While some states appropriate funds for water and wastewater infrastructure assistance programs, most state-originated assistance is limited to what amounts to an extension of the state’s credit rating to eligible regional and local entities, which typically cannot borrow at more attractive terms on the open market.

However, these more traditional financial approaches have not been sufficient to meet the burgeoning infrastructure needs over time, nor have they addressed the growth-related utility rate increases that were being borne by existing customers. With the rapid growth of the 1970s and 1980s, many utilities searched for new and more innovative capital financing techniques, and various approaches were first implemented by utilities and cities under home rule powers in somewhat of an ad hoc nature. Concerns about these new approaches, perceived abuses, and lack of governmental direction on basic authority, acceptable methods, and public process led many state legislatures to enact laws to address impact fees, utility exactions from developers, creation of special utility districts, and other utility funding–related issues.

Given the tremendous infrastructure needs facing water and wastewater utilities, it is important to provide an appropriate *array* of utility funding tools that are effective and seek to minimize undesirable impacts upon various interest groups or development patterns. There are numerous methods available for financing water and wastewater infrastructure. In most cases, several of these tools are employed at the same time by a utility.

The following are some of the more commonly used sources of capital funds or assets in use today:

- Utility rate revenue
- Utility rate surcharges
- Ad valorem tax revenue
- Impact (capital recovery) fee
- Availability of service (standby) fee
- Service-related fees
- Developer exaction or dedication
- Subsidized low-interest loan or grant

Each alternative funding approach may have a different means of being levied upon affected parties, and this can have a bearing upon how effective the tool is in producing revenue, who initially and ultimately bears the cost, and its affordability to consumers. A brief description of the various funding tools and how they are typically levied or charged is shown in Table 32–1.

Detailed information on design, implementation, effectiveness, and impact issues for each funding tool would consume an entire book, so an overview of the major funding tools is provided in the following sections, along with references to ‘‘how-to’’ methodologies in other publications. Also, care should be taken to determine the state legal authority for the levy of certain fees and charges.

Utility Rates

The AWWA characterizes utility rates as a periodic charge for service that generates ‘‘sufficient total revenue to ensure proper operation and maintenance of the system, development and perpetuation of the system, and maintenance of the utility’s financial integrity.’’[1] Other fees and charges may also be levied, but utility rate revenue is, in most cases, the most significant revenue stream of the utility.

TABLE 32-1. Description of Typical Water and Wastewater Infrastructure Funding Methods and Means of Levy

- **Utility rate revenue:** revenue gathered from periodic (usually monthly) billing of customers for utility service and used to address utility operating and capital funding, either in the form of debt service payments or cash funding of capital projects
- **Utility rate surcharge:** a special additional charge on the monthly utility bill intended to target capital recovery for certain items or target certain customers for an additional level of capital recovery reflective of unusual service costs associated with that customer group
- **Ad valorem tax revenue:** annual revenue gathered from a broad-based assessment of real property value that is used to address utility operating and/or capital funding needs, typically used today only by special water districts
- **Impact (capital recovery) fee:** a front-end payment or customer contribution typically assessed to new connections for the purposes of providing capital funding needs, typically used today only by special water districts
- **Availability of service (stand-by) fee:** a monthly charge to utility customers to recover capital-related and ongoing costs incurred by a utility when it is constructing facilities for the benefit of future customers; normally, this is applicable during the period when service is first made available to a possible customer and the time service actually begins
- **Tap fee:** a one-time charge to new utility connections made for purchase/installation of the water meter and/or making the water or wastewater customer service connection to the utility
- **Fire protection charge:** a periodic charge to selected special-need customers for providing public or private fire protection services, typically derived from an allocation of the general or customer-specific additional costs to the utility of providing high (fire) flow capabilities and other fire-related facilities (hydrants, standpipes, etc.) in the system
- **Demand contract charge:** similar to availability of service fees, demand contract charges are periodic payments where a significant (high volume) customer(s) may contract to pay the fixed costs related to a particular share of utility capacity attributable to their use
- **Developer exaction/dedication:** typically, capital received through a utility or local governmental policy that requires a land developer, at his own expense, to provide some degree of utility facilities that provide a particular service benefit to that development; this may also include oversizing of facilities for future development in the area for which the initial developer is reimbursed through the levy of ‘‘subsequent user’’ fees from later connections or some other methods of repayment
- **Grants/low-interest subsidized loan:** no cost or low-cost financial assistance usually received from a higher level of government; while nearly all utilities may be eligible for some type of assistance, utility eligibility is defined for each assistance program; investor-owned utilities have the most limited opportunities for public assistance; typically, the funding agency either expends public monies or extends its good credit rating to provide funds to the receiving utility at a interest rate or funding cost lower than can be obtained by the utility itself

There are two major methods of projecting revenue requirements in water rate making: the cash basis and the utility basis.[1] These methods primarily differ in what is considered allowable costs in the rate base. Most public utilities use the *cash basis* where water or wastewater rates are typically based upon projected cash flow of the pending ‘‘rate year,’’ debt service expenses are fully recovered, and a profit is usually not specifically considered. While some governmental utilities may transfer funds to other functions (such as general services), this is usually not rationalized as a profit but as a payment for general services (management, police, fire, etc.) rendered to the utility.

Most privately owned utilities come under some level of state regulation, given their provision of a basic public service and conferred monopoly status for a defined service area. Typically, one of the regulated aspects of these private utilities is rate setting, using the *utility basis* method. With this approach, most reasonable expenses are allowed on a known or anticipated basis, interest expenses are specifically allowed, capital recovery is provided for through a depreciation expense, and reasonable rate of return or profit is allowed based on the amount of invested equity that the owner has in the utility. Usually these for-profit utilities cannot fund specific debt service (principal and interest) payment amounts, but must instead recover capital-related expenses indirectly through allowed depreciation and interest expenses.

Once the rate base of allowed costs has been defined, there is an array of alternative water rate (flat, declining block, seasonal, etc.) or wastewater rate (flat, block charges based on metered water, average winter month, etc.) designs that may seek to achieve various cost-recovery and policy goals.[2] However, the AWWA does not endorse any substantial departure from cost-of-service-based rates to achieve social objectives.[1]

Utility Rate Surcharges

Utility rate surcharges can take a variety of names and forms, although the generally common feature of these additional monthly billing charges is that certain types of customers are being narrowly targeted for special cost recovery due to unusual utility service provision or rate stabilization.[2] Additional charges for enhanced fire protection and special recovery of bonded water distribution and wastewater collection line capital costs from residents of districts that have been annexed by municipalities (normally borne by developers in typical city subdivisions) are two examples of unusual service costs that can be addressed through surcharges. Another form may entail a line item surcharge for all utility customers to specifically identify the collection of funds for a particular purpose, such as water supply acquisition.

Impact Fees

With continuing growth pressures and evolving attitudes, this type of capital charge gained widespread use by municipalities during the early 1980s to make ‘‘growth pay for itself.’’ This particular funding tool goes by many names: impact fees, capital recovery fees, facility charges, plant investment fees, system development charges, and so on. The AWWA characterizes this type of charge as a program of contributions of capital by a developer or new customers connecting to a water system.[3] However, care should be taken to distinguish the water-related impact fee’s typical one-time, up-front payment at time of connection or occupancy from that of developer exactions or other more minor connection or tap fees. This particular fee has been the subject of much debate, and several state legislatures have defined specific policy, methods, and processes for how these fees can be designed and adopted.

Availability of Service Fees

AWWA describes an availability charge (standby fee) as a levy designed to recover capital-related cost and other ongoing costs incurred by a utility when it is constructing facilities for the benefit of potential future users.[3] When levied, it is usually part of the utility's general water rate structure and is applicable from the time service is first made available to a potential customer (or when the levy is first enacted) until the time service goes into effect. It is essentially a charge for having utility service available but not using it.

Ad Valorem Taxes

While municipalities have taxing authority, nearly all modern-day municipal water and wastewater utilities employ utility rates and fees as the primary basis for funding the operations, maintenance, and capital needs of the system. In most states, the practical use of ad valorem (i.e., ''at value'' property assessment) taxes for funding water and wastewater utility purposes is limited to special water districts. This funding mechanism is most typically used where there is a need to rely, in part, on funding from undeveloped property benefiting from the presence of utilities. In lower-density areas that are just developing, property taxation may be an essential part of the funding mix to attract utility capital funds at reasonable rate of interest.

Developer Exactions / Dedications

Exactions are government requirements that developers dedicate land or other facilities for public use or improvements, or pay a fee in lieu of dedication. Within the water and wastewater utility business, these types of capital provisions are typically defined in what is known as utility extension policies.[3]

In many states, there are requirements to provide a level of municipal services to the newly annexed areas similar to those of other municipal areas with similar situations within a defined period of time. In these cases, any major infrastructure needed to serve a new development will be provided at city expense, although the cost of water distribution and wastewater collection lines ''internal'' to a new development are usually borne by the developer or owner. For requests to provide service outside of the municipal boundaries, the developer may be required to fund the cost of major water and/or wastewater approach mains to the development or other facilities (such as lift stations) that may provide unique service to the development. In some cases, there may be extension refund policies or agreements that allow for near- or longer-term reimbursement of these capital costs to the developer. In the instances where the city will require oversizing of the extended facilities to ultimately serve other development, the city may cost-participate in the construction (usually on a pro rata capacity basis) or require the developer to initially fund the entire extension project(s) with an agreement of subsequent reimbursement.

For-profit water corporations generally have extension policies that require the payment of a determined amount by the party requesting service. In many cases, this will constitute a ''footage'' charge for extending current service line lengths. If the service extension request is large or unusual, other capital charges may be applicable. When capital is acquired in this manner, this as defined as ''contributed capital'' and is usually not eligible for earning a rate of return.

Grants / Subsidized Low-Interest Loans

Federal and state financial assistance programs have varied over the years, evolving from substantial grant programs to a greater emphasis on loan and revolving loan assistance programs. Today, financial involvement of most states is primarily focused in federal/state- capitalized, below-market-interest, *state revolving loan fund* (*SRF*) *programs*, the extension of the state's credit rating in making low-interest loans, or limited grant programs for special purposes such as economically distressed areas.

Broader eligibility for such federal/state assistance is generally limited to political subdivisions of the states and nonprofit water and sewer corporations with IOUs being able to access some assistance only from the state Drinking Water SRF. Also, there may be public policy initiatives that are associated with accessing funds from these sources, such as environmental review and promotion of water conservation. So while not strictly a rate or fee funding tool, these public financial assistance programs can be an important source of lower-cost, and sometimes no-cost, funding for the provision of capital infrastructure.

REFERENCES

1. AWWA, *Manual of Water Supply Practices M1: Water Rates,* American Water Works Association, 1991.
2. AWWA, *Manual of Water Supply Practices M34: Alternative Rates,* American Water Works Association, 1992.
3. AWWA, *Manual of Water Supply Practices M26: Water Rates and Related Charges,* American Water Works Association, 1986.

APPENDIX A

Financial Statistics Tables

TABLE A-1. Values of 1 Amortized

Number of Periods	Periodic Interest Rate (i)																
	2.0%	2.5%	3.0%	3.5%	4.0%	4.5%	5.0%	5.5%	6.0%	6.5%	7.0%	7.5%	8.0%	8.5%	9.0%	9.5%	10.0%
2	0.5150	0.5188	0.5226	0.5264	0.5302	0.5340	0.5378	0.5416	0.5454	0.5493	0.5531	0.5569	0.5608	0.5646	0.5685	0.5723	0.5762
3	0.3468	0.3501	0.3535	0.3569	0.3603	0.3638	0.3672	0.3707	0.3741	0.3776	0.3811	0.3845	0.3880	0.3915	0.3951	0.3986	0.4021
4	0.2626	0.2658	0.2690	0.2723	0.2755	0.2787	0.2820	0.2853	0.2886	0.2919	0.2952	0.2986	0.3019	0.3053	0.3087	0.3121	0.3155
5	0.2122	0.2152	0.2184	0.2215	0.2246	0.2278	0.2310	0.2342	0.2374	0.2406	0.2439	0.2472	0.2505	0.2538	0.2571	0.2604	0.2638
6	0.1785	0.1815	0.1846	0.1877	0.1908	0.1939	0.1970	0.2002	0.2034	0.2066	0.2098	0.2130	0.2163	0.2196	0.2229	0.2263	0.2296
7	0.1545	0.1575	0.1605	0.1635	0.1666	0.1697	0.1728	0.1760	0.1791	0.1823	0.1856	0.1888	0.1921	0.1954	0.1987	0.2020	0.2054
8	0.1365	0.1395	0.1425	0.1455	0.1485	0.1516	0.1547	0.1579	0.1610	0.1642	0.1675	0.1707	0.1740	0.1773	0.1807	0.1840	0.1874
9	0.1225	0.1255	0.1284	0.1314	0.1345	0.1376	0.1407	0.1438	0.1470	0.1502	0.1535	0.1568	0.1601	0.1634	0.1668	0.1702	0.1736
10	0.1113	0.1143	0.1172	0.1202	0.1233	0.1264	0.1295	0.1327	0.1359	0.1391	0.1424	0.1457	0.1490	0.1524	0.1558	0.1593	0.1627
11	0.1022	0.1051	0.1081	0.1111	0.1141	0.1172	0.1204	0.1236	0.1268	0.1301	0.1134	0.1367	0.1401	0.1435	0.1469	0.1504	0.1540
12	0.0946	0.0975	0.1005	0.1035	0.1066	0.1097	0.1128	0.1160	0.1193	0.1226	0.1259	0.1293	0.1327	0.1362	0.1397	0.1432	0.1468
13	0.0881	0.0910	0.0940	0.0971	0.1001	0.1033	0.1065	0.1097	0.1130	0.1163	0.1197	0.1231	0.1265	0.1300	0.1336	0.1372	0.1408
14	0.0826	0.0855	0.0885	0.0916	0.0947	0.0978	0.1010	0.1043	0.1076	0.1109	0.1143	0.1178	0.1213	0.1248	0.1284	0.1321	0.1357
15	0.0778	0.0808	0.0838	0.0868	0.0899	0.0931	0.0963	0.0996	0.1030	0.1064	0.1098	0.1133	0.1168	0.1204	0.1241	0.1277	0.1315
16	0.0737	0.0766	0.0796	0.0827	0.0858	0.0890	0.0923	0.0956	0.0990	0.1024	0.1059	0.1094	0.1130	0.1166	0.1203	0.1240	0.1278
17	0.0700	0.0729	0.0760	0.0790	0.0822	0.0854	0.0887	0.0920	0.0954	0.0989	0.1024	0.1060	0.1096	0.1133	0.1170	0.1208	0.1247
18	0.0667	0.0697	0.0727	0.0758	0.0790	0.0822	0.0855	0.0889	0.0924	0.0959	0.0994	0.1030	0.1067	0.1104	0.1142	0.1180	0.1219
19	0.0638	0.0668	0.0698	0.0729	0.0761	0.0794	0.0827	0.0862	0.0896	0.0932	0.0968	0.1004	0.1041	0.1079	0.1117	0.1156	0.1195
20	0.0612	0.0641	0.0672	0.0704	0.0736	0.0769	0.0802	0.0837	0.0872	0.0908	0.0944	0.0981	0.1019	0.1057	0.1095	0.1135	0.1175

21	0.0588	0.0618	0.0649	0.0680	0.0713	0.0746	0.0780	0.0815	0.0850	0.0886	0.0923	0.0960	0.0998	0.1037	0.1076	0.1116	0.1156
22	0.0566	0.0596	0.0627	0.0659	0.0692	0.0725	0.0760	0.0795	0.0830	0.0867	0.0904	0.0942	0.0980	0.1019	0.1059	0.1099	0.1140
23	0.0547	0.0577	0.0608	0.0640	0.0673	0.0707	0.0741	0.0777	0.0813	0.0850	0.0887	0.0925	0.0964	0.1004	0.1044	0.1084	0.1126
24	0.0529	0.0559	0.0590	0.0623	0.0656	0.0690	0.0725	0.0760	0.0797	0.0834	0.0872	0.0911	0.0950	0.0990	0.1030	0.1071	0.1113
25	0.0512	0.0543	0.0574	0.0607	0.0640	0.0674	0.0710	0.0745	0.0782	0.0820	0.0858	0.0897	0.0937	0.0977	0.1018	0.1060	0.1102
26	0.0497	0.0528	0.0559	0.0592	0.0626	0.0660	0.0696	0.0732	0.0769	0.0807	0.0846	0.0885	0.0925	0.0966	0.1007	0.1049	0.1092
27	0.0483	0.0514	0.0546	0.0579	0.0612	0.0647	0.0683	0.0720	0.0757	0.0795	0.0834	0.0874	0.0914	0.0956	0.0997	0.1040	0.1083
28	0.0470	0.0501	0.0533	0.0566	0.0600	0.0635	0.0671	0.0708	0.0746	0.0785	0.0824	0.0864	0.0905	0.0946	0.0989	0.1031	0.1075
29	0.0458	0.0489	0.0521	0.0554	0.0589	0.0624	0.0660	0.0698	0.0736	0.0775	0.0814	0.0855	0.0896	0.0938	0.0981	0.1024	0.1067
30	0.0446	0.0478	0.0510	0.0544	0.0578	0.0614	0.0651	0.0688	0.0726	0.0766	0.0806	0.0847	0.0888	0.0931	0.0973	0.1017	0.1061
31	0.0436	0.0467	0.0500	0.0534	0.0569	0.0604	0.0641	0.0679	0.0718	0.0758	0.0798	0.0839	0.0881	0.0924	0.0967	0.1011	0.1055
32	0.0426	0.0458	0.0490	0.0524	0.0559	0.0596	0.0633	0.0671	0.0710	0.0750	0.0791	0.0832	0.0875	0.0917	0.0961	0.1005	0.1050
33	0.0417	0.0449	0.0482	0.0516	0.0551	0.0587	0.0625	0.0663	0.0703	0.0743	0.0784	0.0826	0.0869	0.0912	0.0956	0.1000	0.1045
34	0.0408	0.0440	0.0473	0.0508	0.0543	0.0580	0.0618	0.0656	0.0696	0.0737	0.0778	0.0820	0.0863	0.0907	0.0951	0.0995	0.1041
35	0.0400	0.0432	0.0465	0.0500	0.0536	0.0573	0.0611	0.0650	0.0690	0.0731	0.0772	0.0815	0.0858	0.0902	0.0946	0.0991	0.1037
36	0.0392	0.0425	0.0458	0.0493	0.0529	0.0566	0.0604	0.0644	0.0684	0.0725	0.0767	0.0810	0.0853	0.0898	0.0942	0.0988	0.1033
37	0.0385	0.0417	0.0451	0.0486	0.0522	0.0560	0.0598	0.0638	0.0679	0.0720	0.0762	0.0805	0.0849	0.0894	0.0939	0.0984	0.1030
38	0.0378	0.0411	0.0445	0.0480	0.0516	0.0554	0.0593	0.0633	0.0674	0.0715	0.0758	0.0801	0.0845	0.0890	0.0935	0.0981	0.1027
39	0.0372	0.0404	0.0438	0.0474	0.0511	0.0549	0.0588	0.0628	0.0669	0.0711	0.0754	0.0798	0.0842	0.0887	0.0932	0.0978	0.1025
40	0.0366	0.3098	0.0433	0.0468	0.0505	0.0543	0.0583	0.0623	0.0665	0.0707	0.0750	0.0794	0.0839	0.0884	0.0930	0.0976	0.1023

TABLE A-2. Amount of 1 at Compound Interest
$s = (1 + i)^n$

Number of Periods (n)	Periodic Discount Rate (i)																
	2.0%	2.5%	3.0%	3.5%	4.0%	4.5%	5.0%	5.5%	6.0%	6.5%	7.0%	7.5%	8.0%	8.5%	9.0%	9.5%	10.0%
2	1.0404	1.0506	1.0609	1.0712	1.0816	1.0920	1.1025	1.1130	1.1236	1.1342	1.1449	1.1556	1.1664	1.1772	1.1881	1.1990	1.2100
3	1.0612	1.0769	1.0927	1.1087	1.1249	1.1412	1.1576	1.1742	1.1910	1.2079	1.2250	1.2423	1.2597	1.2773	1.2950	1.3129	1.3310
4	1.0824	1.1038	1.1255	1.1475	1.1699	1.1925	1.2155	1.2388	1.2625	1.2865	1.3108	1.3355	1.3605	1.3859	1.4116	1.4377	1.4641
5	1.1041	1.1314	1.1593	1.1877	1.2167	1.2462	1.2763	1.3070	1.3382	1.3701	1.4026	1.4356	1.4693	1.5037	1.5386	1.5742	1.6105
6	1.1262	1.1597	1.1941	1.2293	1.2653	1.3023	1.3401	1.3788	1.4185	1.4591	1.5007	1.5433	1.5869	1.6315	1.6771	1.7238	1.7716
7	1.1487	1.1887	1.2299	1.2723	1.3159	1.3609	1.4071	1.4547	1.5036	1.5540	1.6058	1.6590	1.7138	1.7701	1.8280	1.8876	1.9487
8	1.1717	1.2184	1.2668	1.3168	1.3686	1.4221	1.4775	1.5347	1.5938	1.6550	1.7182	1.7835	1.8509	1.9206	1.9926	2.0669	2.1436
9	1.1951	1.2489	1.3048	1.3629	1.4233	1.4861	1.5513	1.6191	1.6895	1.7626	1.8385	1.9172	1.9990	2.0839	2.1719	2.2632	2.3579
10	1.2190	1.2801	1.3439	1.4106	1.4802	1.5530	1.6289	1.7081	1.7908	1.8771	1.9672	2.0610	2.1589	2.2610	2.3674	2.4782	2.5937
11	1.2434	1.3121	1.3842	1.4600	1.5395	1.6229	1.7103	1.8021	1.8983	1.9992	2.1049	2.2156	2.3316	2.4532	2.5804	2.7137	2.8531
12	1.2682	1.3449	1.4258	1.5111	1.6010	1.6959	1.7959	1.9012	2.0122	2.1291	2.2522	2.3818	2.5182	2.6617	2.8127	2.9715	3.1384
13	1.2936	1.3785	1.4685	1.5640	1.6651	1.7722	1.8856	2.0058	2.1329	2.2675	2.4098	2.5604	2.7196	2.8879	3.0658	3.2537	3.4523
14	1.3195	1.4130	1.5126	1.6187	1.7317	1.8519	1.9799	2.1161	2.2609	2.4149	2.5785	2.7524	2.9372	3.1334	3.3417	3.5629	3.7975
15	1.3459	1.4483	1.5580	1.6753	1.8009	1.9353	2.0789	2.2325	2.3966	2.5718	2.7590	2.9589	3.1722	3.3997	3.6425	3.9013	4.1772
16	1.3728	1.4845	1.6047	1.7340	1.8730	2.0224	2.1829	2.3553	2.5404	2.7390	2.9522	3.1808	3.4259	3.6887	3.9703	4.2719	4.5950
17	1.4002	1.5216	1.6528	1.7947	1.9479	2.1134	2.2920	2.4848	2.6928	2.9170	3.1588	3.4194	3.7000	4.0023	4.3276	4.6778	5.0545
18	1.4282	1.5597	1.7024	1.8575	2.0258	2.2085	2.4066	2.6215	2.8543	3.1067	3.3799	3.6758	3.9960	4.3425	4.7171	5.1222	5.5599
19	1.4568	1.5987	1.7535	1.9225	2.1068	2.3079	2.5270	2.7656	3.0256	3.3086	3.6165	3.9515	4.3157	4.7116	5.1417	5.6088	6.1159
20	1.4859	1.6386	1.8061	1.9898	2.1911	2.4117	2.6533	2.9178	3.2071	3.5236	3.8697	4.2479	4.6610	5.1120	5.6044	6.1416	6.7275

21	1.5157	1.6796	1.8603	2.0594	2.2788	2.5202	2.7860	3.0782	3.3996	3.7527	4.1406	4.5664	5.0338	5.5466	6.1088	6.7251	7.4002
22	1.5460	1.7216	1.9161	2.1315	2.3699	2.6337	2.9253	3.2475	3.6035	3.9966	4.4304	4.9089	5.4365	6.0180	6.6586	7.3639	8.1403
23	1.5769	1.7646	1.9736	2.2061	2.4647	2.7522	3.0715	3.4262	3.8197	4.2564	4.7405	5.2771	5.8715	6.5296	7.2579	8.0635	8.9543
24	1.6084	1.8087	2.0328	2.2833	3.5633	2.8760	3.2251	3.6146	4.0489	4.5331	5.0724	5.6729	6.3412	7.0846	7.9111	8.8296	9.8497
25	1.6406	1.8539	2.0938	2.3632	2.6658	3.0054	3.3864	3.8134	4.2919	4.8277	5.4274	6.0983	6.8485	7.6868	8.6231	9.6684	10.8347
26	1.6734	1.9003	2.1566	2.4460	2.7725	3.1407	3.5557	4.0231	4.5494	5.1415	5.8074	6.5557	7.3964	8.3401	9.3992	10.5869	11.9182
27	1.7069	1.9478	2.2213	2.5316	2.8834	3.2820	3.7335	4.2444	4.8223	5.4757	6.2139	7.0474	7.9881	9.0490	10.2451	11.5926	13.1100
28	1.7410	1.9965	2.2879	2.6202	2.9987	3.4297	3.9201	4.4778	5.1117	5.8316	6.6488	7.5759	8.6271	9.8182	11.1671	12.6939	14.4210
29	1.7758	2.0464	2.3566	2.7119	3.1187	3.5840	4.1161	4.7241	5.4184	6.2107	7.1143	8.1441	9.3173	10.6528	12.1722	13.8998	15.8631
30	1.8114	2.0976	2.4273	2.8068	3.2434	3.7453	4.3219	4.9840	5.7435	6.6144	7.6123	8.7550	10.0627	11.5583	13.2677	15.2203	17.4494
31	1.8476	2.1500	2.5001	2.9050	3.3731	3.9139	4.5380	5.2581	6.0881	7.0443	8.1451	9.4116	10.8677	12.5407	14.4618	16.6662	19.1943
32	1.8845	2.2038	2.5751	3.0067	3.5081	4.0900	4.7649	5.5473	6.4534	7.5022	8.7153	10.1174	11.7371	13.6067	15.7633	18.2495	21.1138
33	1.9222	2.2589	2.6523	3.1119	3.6484	4.2740	5.0032	5.8524	6.8406	7.9898	9.3253	10.8763	12.6760	14.7632	17.1820	19.9832	23.2252
34	1.9607	2.3153	2.7319	3.2209	3.7943	4.4664	5.2533	6.1742	7.2510	8.5092	9.9781	11.6920	13.6901	16.0181	18.7284	21.8816	25.5477
35	1.9999	2.3732	2.8139	3.3336	3.9461	4.6673	5.5160	6.5138	7.6861	9.0623	10.6766	12.5689	14.7853	17.3796	20.4140	23.9604	28.1024
36	2.0399	2.4325	2.8983	3.4503	4.1039	4.8774	5.7918	6.8721	8.1473	9.6513	11.4239	13.5115	15.9682	18.8569	22.2512	26.2366	30.9127
37	2.0807	2.4933	2.9852	3.5710	4.2681	5.0969	6.0814	7.2501	8.6361	10.2786	12.2236	14.5249	17.2456	20.4597	24.2538	28.7291	34.0039
38	2.1223	2.5557	3.0748	3.6960	4.4388	5.3262	6.3855	7.6488	9.1543	10.9467	13.0793	15.6143	18.6253	22.1988	26.4367	31.4584	37.4043
39	2.1647	2.6196	3.1670	3.8254	4.6164	5.5659	6.7048	8.0695	9.7035	11.6583	13.9948	16.7853	20.1153	24.0857	28.8160	34.4469	41.1448
40	2.2080	2.6851	3.2620	3.9593	4.8010	5.8164	7.0400	8.5133	10.2857	12.4161	14.9745	18.0442	21.7245	26.1330	31.4094	37.7194	45.2593

TABLE A–3. Values of 1 at Compound Discount
a = $(1 + i)^{-n}$

Number of Periods (n)	Periodic Discount Rate (i)																
	2.0%	2.5%	3.0%	3.5%	4.0%	4.5%	5.0%	5.5%	6.0%	6.5%	7.0%	7.5%	8.0%	8.5%	9.0%	9.5%	10.0%
2	0.9612	0.9518	0.9426	0.9335	0.9246	0.9157	0.9070	0.8985	0.8900	0.8817	0.8734	0.8653	0.8573	0.8495	0.8417	0.8340	0.8264
3	0.9423	0.9286	0.9151	0.9019	0.8890	0.8763	0.8638	0.8516	0.8396	0.8278	0.8163	0.8050	0.7938	0.7829	0.7722	0.7617	0.7513
4	0.9238	0.9060	0.8885	0.8714	0.8548	0.8386	0.8227	0.8072	0.7921	0.7773	0.7629	0.7488	0.7350	0.7216	0.7084	0.6956	0.6830
5	0.9057	0.8839	0.8626	0.8420	0.8219	0.8025	0.7835	0.7651	0.7473	0.7299	0.7130	0.6966	0.6806	0.6650	0.6499	0.6352	0.6209
6	0.8880	0.8623	0.8375	0.8135	0.7903	0.7679	0.7462	0.7252	0.7050	0.6853	0.6663	0.6480	0.6302	0.6129	0.5963	0.5801	0.5645
7	0.8706	0.8413	0.8131	0.7860	0.7599	0.7348	0.7107	0.6874	0.6651	0.6435	0.6227	0.6028	0.5835	0.5649	0.5470	0.5298	0.5132
8	0.8535	0.8207	0.7894	0.7594	0.7307	0.7032	0.6768	0.6516	0.6274	0.6042	0.5820	0.5607	0.5403	0.5207	0.5019	0.4838	0.4665
9	0.8368	0.8007	0.7664	0.7337	0.7026	0.6729	0.6446	0.6176	0.5919	0.5674	0.5439	0.5216	0.5002	0.4799	0.4604	0.4418	0.4241
10	0.8203	0.7812	0.7441	0.7089	0.6756	0.6439	0.6139	0.5854	0.5584	0.5327	0.5083	0.4852	0.4632	0.4423	0.4224	0.4035	0.3855
11	0.8043	0.7621	0.7224	0.6849	0.6496	0.6162	0.5847	0.5549	0.5268	0.5002	0.4751	0.4513	0.4289	0.4076	0.3875	0.3685	0.3505
12	0.7885	0.7436	0.7014	0.6618	0.6246	0.5897	0.5568	0.5260	0.4970	0.4697	0.4440	0.4199	0.3971	0.3757	0.3555	0.3365	0.3186
13	0.7730	0.7254	0.6810	0.6394	0.6006	0.5643	0.5303	0.4986	0.4688	0.4410	0.4150	0.3906	0.3677	0.3463	0.3262	0.3073	0.2897
14	0.7579	0.7077	0.6611	0.6178	0.5775	0.5400	0.5051	0.4726	0.4423	0.4141	0.3878	0.3633	0.3405	0.3191	0.2992	0.2807	0.2633
15	0.7430	0.6905	0.6419	0.5969	0.5553	0.5167	0.4810	0.4479	0.4173	0.3888	0.3624	0.3380	0.3152	0.2941	0.2745	0.2563	0.2394
16	0.7284	0.6736	0.6232	0.5767	0.5339	0.4945	0.4581	0.4246	0.3936	0.3651	0.3387	0.3144	0.2919	0.2711	0.2519	0.2341	0.2176
17	0.7142	0.6572	0.6060	0.5572	0.5134	0.4732	0.4363	0.4024	0.3714	0.3428	0.3166	0.2925	0.2703	0.2499	0.2311	0.2138	0.1978
18	0.7002	0.6412	0.5874	0.5384	0.4936	0.4528	0.4155	0.3815	0.3503	0.3219	0.2959	0.2720	0.2502	0.2303	0.2120	0.1952	0.1799
19	0.6864	0.6255	0.5703	0.5202	0.4746	0.4333	0.3957	0.3616	0.3305	0.3022	0.2765	0.2531	0.2317	0.2122	0.1945	0.1783	0.1635
20	0.6730	0.6103	0.5537	0.5026	0.4564	0.4146	0.3769	0.3427	0.3118	0.2838	0.2584	0.2354	0.2145	0.1956	0.1784	0.1628	0.1486

21	0.6598	0.5954	0.5375	0.4856	0.4388	0.3968	0.3589	0.3249	0.2942	0.2665	0.2415	0.2190	0.1987	0.1803	0.1637	0.1487	0.1351
22	0.6468	0.5809	0.5219	0.4692	0.4220	0.3797	0.3418	0.3079	0.2775	0.2502	0.2257	0.2037	0.1839	0.1662	0.1502	0.1358	0.1228
23	0.6342	0.5667	0.5067	0.4533	0.4057	0.3634	0.3256	0.2919	0.2618	0.2349	0.2109	0.1895	0.1703	0.1531	0.1378	0.1240	0.1117
24	0.6217	0.5529	0.4919	0.4380	0.3901	0.3477	0.3101	0.2767	0.2470	0.2206	0.1971	0.1763	0.1577	0.1412	0.1264	0.1133	0.1015
25	0.6095	0.5394	0.4776	0.4231	0.3751	0.3327	0.2953	0.2622	0.2330	0.2071	0.1842	0.1640	0.1460	0.1301	0.1160	0.1034	0.0923
26	0.5976	0.5262	0.4637	0.4088	0.3607	0.3184	0.2812	0.2486	0.2198	0.1945	0.1722	0.1525	0.1352	0.1199	0.1064	0.0945	0.0839
27	0.5859	0.5134	0.4502	0.3950	0.3468	0.3047	0.2678	0.2356	0.2074	0.1826	0.1609	0.1419	0.1252	0.1105	0.0976	0.0863	0.0763
28	0.5744	0.5009	0.4371	0.3817	0.3335	0.2916	0.2551	0.2233	0.1956	0.1715	0.1504	0.1320	0.1159	0.1019	0.0895	0.0788	0.0693
29	0.5631	0.4887	0.4243	0.3687	0.3207	0.2790	0.2429	0.2117	0.1846	0.1610	0.1406	0.1228	0.1073	0.0939	0.0822	0.0719	0.0630
30	0.5521	0.4767	0.4120	0.3563	0.3083	0.2670	0.2314	0.2006	0.1741	0.1512	0.1314	0.1142	0.0994	0.0865	0.0754	0.0657	0.0573
31	0.5412	0.4651	0.4000	0.3442	0.2965	0.2555	0.2204	0.1902	0.1643	0.1420	0.1228	0.1063	0.0920	0.0797	0.0691	0.0600	0.0521
32	0.5306	0.4538	0.3883	0.3326	0.2851	0.2445	0.2099	0.1803	0.1550	0.1333	0.1147	0.0988	0.0852	0.0735	0.0634	0.0548	0.0474
33	0.5202	0.4427	0.3770	0.3213	0.2741	0.2340	0.1999	0.1709	0.1462	0.1252	0.1072	0.0919	0.0789	0.0677	0.0582	0.0500	0.0431
34	0.5100	0.4319	0.3660	0.3105	0.2636	0.2239	0.1904	0.1620	0.1379	0.1175	0.1002	0.0855	0.0730	0.0624	0.0534	0.0457	0.0391
35	0.5000	0.4214	0.3554	0.3000	0.2534	0.2143	0.1813	0.1535	0.1301	0.1103	0.0937	0.0796	0.0676	0.0575	0.0490	0.0417	0.0356
36	0.4902	0.4111	0.3450	0.2898	0.2437	0.2050	0.1727	0.1455	0.1227	0.1036	0.0875	0.0740	0.0626	0.0530	0.0449	0.0381	0.0323
37	0.4806	0.4011	0.3350	0.2800	0.2343	0.1962	0.1644	0.1379	0.1158	0.0973	0.0818	0.0688	0.0580	0.0489	0.0412	0.0348	0.0294
38	0.4712	0.3913	0.3252	0.2706	0.2253	0.1878	0.1566	0.1307	0.1092	0.0914	0.0765	0.0640	0.0537	0.0450	0.0378	0.0318	0.0267
39	0.4619	0.3817	0.3158	0.2614	0.2166	0.1797	0.1491	0.1239	0.1031	0.0858	0.0715	0.0596	0.0497	0.0415	0.0347	0.0290	0.0243
40	0.4529	0.3724	0.3066	0.2526	0.2083	0.1719	0.1420	0.1175	0.0972	0.0805	0.0668	0.0554	0.0460	0.0383	0.0318	0.0265	0.0211

APPENDIX B

Periodic Table of the Elements

PERIODIC TABLE OF THE ELEMENTS

1a	2a	3b	4b	5b	6b	7b	8			1b	2b	3a	4a	5a	6a	7a	0
1 +1 H 1.0079																	2 0 He 4.0026
3 +1 Li 6.94	4 +2 Be 9.01218			Atomic Number → Symbol → Atomic Weight →	50 +2 +4 Sn 118.69	← Oxidation States						5 +3 B 10.81	6 +4 C 12.011	7 +5 +3 N 14.0067 -3	8 O 15.9994 -2	9 F 18.998403 -1	10 0 Ne 20.17
11 +1 Na 22.98977	12 +2 Mg 24.305											13 +3 Al 26.98154	14 +4 Si 28.0855	15 +5 +3 P 30.97376 -3	16 +6 +4 S 32.06 -2	17 Cl 35.453 -1	18 0 Ar 39.948
19 +1 K 39.0983	20 +2 Ca 40.08	21 Sc 44.9559	22 Ti 47.9	23 V 50.941	24 +6 +3 Cr 51.996	25 +7 +4 +2 Mn 54.938	26 +3 +2 Fe 55.847	27 +3 +2 Co 58.9332	28 +3 +2 Ni 58.71	29 +2 +1 Cu 63.546	30 +2 Zn 65.38	31 +3 Ga 69.72	32 +4 +2 Ge 72.59	33 +5 +3 As 74.9216	34 +6 +4 Se 78.96 -2	35 Br 79.904 -1	36 0 Kr 83.8
37 +1 Rb 85.467	38 +2 Sr 87.62	39 Y 88.9059	40 Zr 91.22	41 Nb 92.9064	42 Mo 95.94	43 Tc 98.9062	44 Ru 101.07	45 Rh 102.9055	46 Pd 106.4	47 +1 Ag 107.868	48 +2 Cd 112.41	49 +3 In 114.82	50 +4 +2 Sn 118.69	51 +5 +3 Sb 121.75	52 +6 +4 Te 127.6 -2	53 I 126.9045 -1	54 0 Xe 131.3
55 +1 Cs 132.9054	56 +2 Ba 137.33	57 La 138.9055	72 Hf 178.49	73 Ta 180.947	74 W 183.85	75 Re 186.2	76 Os 190.2	77 Ir 192.22	78 Pt 195.09	79 Au 196.9665	80 +2 +1 Hg 200.59	81 +3 +1 Ti 204.37	82 +4 +2 Pb 207.2	83 +5 +3 Bi 208.9808	84 +6 +4 Po (210)	85 At (210)	86 0 Rn (222)
87 +1 Fr (223)	88 +2 Ra 226.0254	89 Ac (227)	104 Ku* (261)	105 Ha* (262)													

Lanthanum Series	58 Ce 140.12	59 Pr 140.9	60 Nd 144.24	61 Pm (145)	62 Sm 150.4	63 Eu 151.96	64 Gd 157.2	65 Tb 158.93	66 Dy 162.50	67 Ho 164.93	68 Er 167.26	69 Tm 168.93	70 Yb 173.04	71 Lu 174.97
Actinium Series	90 Th 232.0	91 Pa 231.0	92 U 238.0	93 Np 237.0	94 Pu (244)	95 Am (243)	96 Cm (247)	97 Bk (247)	98 Cf (251)	99 Es (254)	100 Fm (257)	101 Md (256)	102 No (254)	103 Lr (257)

Mass numbers of the most stable or most abundant isotopes are shown in parentheses.
The elements to the right of the bold lines are called the non metals and the elements to the left of the bold line are called the metals.
Common oxidation numbers are given for the representative elements and some transition elements.
* Kurchatovium and Hahnium are tentative names for these elements.

APPENDIX C

Atomic Weights

Name	Symbol	Atomic Number	Atomic Weight (amu)
Actinium	Ac	89	(227)
Aluminum	Al	13	26.98154
Americium	Am	95	(243)
Antimony	Sb	51	121.7_5
Argon	Ar	18	39.94_8
Arsenic	As	33	74.9216
Astatine	At	85	(~ 210)
Barium	Ba	56	137.3_4
Berkelium	Bk	97	(247)
Beryllium	Be	4	9.01218
Bismuth	Bi	83	208.9806
Boron	B	5	10.81
Bromine	Br	35	79.904
Cadmium	Cd	48	112.4
Calcium	Ca	20	40.08
Californium	Cf	98	(251)
Carbon	C	6	12.011
Cerium	Ce	58	140.12
Cesium	Cs	55	132.9055
Chlorine	Cl	17	35.453
Chromium	Cr	24	51.996
Cobalt	Co	27	58.9332
Copper	Cu	29	63.546
Curium	Cm	96	(247)
Dysprosium	Dy	66	162.5
Einsteinium	Es	99	(254)
Erbium	Er	68	167.26
Europium	Eu	63	151.96
Fermium	Fm	100	(257)
Fluorine	F	9	18.9984
Francium	Fr	87	(223)
Gadolinium	Gd	64	157.2_5
Gallium	Ga	31	69.72
Germanium	Ge	32	72.5_9
Gold	Au	79	196.9665

Name	Symbol	Atomic Number	Atomic Weight (amu)
Hafnium	Hf	72	178.4_9
Helium	He	2	4.00260
Holmium	Ho	67	164.9303
Hydrogen	H	1	1.0080
Indium	In	49	114.82
Iodine	I	53	126.9045
Iridium	Ir	77	192.2_2
Iron	Fe	26	55.84_7
Krypton	Kr	36	83.80
Lanthanium	La	57	138.905_5
Lawrencium	Lr	103	(257)
Lead	Pb	82	207.2
Lithium	Li	3	6.941
Lutetium	Lu	71	174.97
Magnesium	Mg	12	24.305
Manganese	Mn	25	54.9380
Mendelevium	Md	101	(256)
Mercury	Hg	80	200.5_9
Molybdenum	Mo	42	95.9_4
Neodymium	Nd	60	144.2_4
Neon	Ne	10	20.17_9
Neptunium	Np	93	237.0482
Nickel	Ni	28	58.7_1
Niobium	Nb	41	92.9064
Nitrogen	N	7	14.0067
Nobelium	No	102	(254)
Osmium	Os	76	190.2
Oxygen	O	8	15.999_4
Palladium	Pd	46	106.4
Phosphorous	P	15	30.9738
Platinum	Pt	78	195.0_9
Plutonium	Pu	94	(244)
Polonium	Po	84	(~210)
Potassium	Kr	19	39.10_2
Praseodymium	Pr	59	140.907_7
Promethium	Pm	61	(145)
Protactinium	Pa	91	231.0359
Radium	Ra	88	226.0254
Radon	Rn	86	(~222)
Rhenium	Re	75	186.207
Rhodium	Rh	45	102.9055
Rubidium	Rb	37	85.467_8
Ruthenium	Ru	44	101.0_7
Samarium	Sm	62	150.4
Scandium	Sc	21	44.9559
Selenium	Se	34	78.9_6
Silicon	Si	14	28.08_6
Silver	Ag	47	107.868
Sodium	Na	11	22.9898
Strontium	Sr	38	87.62

Name	Symbol	Atomic Number	Atomic Weight (amu)
Sulfur	S	16	32.06
Tantalum	Ta	73	180.947_9
Technetium	Tc	43	98.9062
Tellurium	Te	52	127.6_0
Terbium	Tb	65	158.9254
Thallium	Tl	81	204.3_7
Thorium	Th	90	232.0381
Thulium	Tm	69	168.9342
Tin	Sn	50	118.6_9
Titanium	Ti	22	47.9_0
Tungsten	W	74	183.8_5
Uranium	U	92	238.029
Vanadium	V	23	50.941_4
Xenon	Xe	54	131.3
Ytterbium	Yb	70	173.0_4
Yttrium	Y	39	88.9059
Zinc	Zn	30	65.3_7
Zirconium	Zr	40	91.22

Based on the assigned relative atomic mass of $^{12}C = 12$

APPENDIX D

Metric Conversion Units

TABLE D–1. Base Units

Quantity	Unit Name	Unit Symbol
Length	meter	m
Mass	kilogram	kg
Time	second	s
Electric current	ampere	A
Thermodynamic temperature	kelvin	K
Amount of substance	mole	mol
Luminous intensity	candela	cd

TABLE D–2. Examples of Derived Units

Quantity	Unit Name	Unit Symbol	Expressed in Terms of SI Units
Velocity			m/s
Acceleration			m/s^2
Force	Newton	N	$kg\ m/s^2$
Pressure, stress, modulus	Pascal	Pa	N/m^2
Energy work, quantity of heat	Joule	J	N m
Power	Watt	W	J/s
Volume	liter	L	$10^{-3}m^3$

TABLE D–3. Most Frequently Used Prefixes

Multiples			Sub-Multiples		
Factor	Prefix	Symbol	Factor	Prefix	Symbol
10^9	giga	G	10^{-3}	milli	m
10^6	mega	M	10^{-6}	micro	μ*
10^3	kilo	k	10^{-9}	nano	nn

*The upright (roman) type style is preferred when available, as with all other prefixes. We sometimes use u for μ.

TABLE D–4. Metric Units and Conversion Factors

Unit Type	Current Unit		Metric Unit
Conversion	Multiply	By	To Obtain
Length	inches (in)	25.40	millimeters (mm)
Length	feet (ft)	0.3048	meters (m)
Length	yards (yd)	0.9144	meters (m)
Length	miles (mi)	1.609	kilometers (km)
Area	square inches (in^2)	645.2	square millimeter (mm^2)
Area	square feet (ft^2)	0.09290	square meters (m^2)
Area	acre (ac)	0.4047	hectares (ha)
Area	square miles (mi^2)	2.590	square kilometers (km^2)
Volume	gallons (gal)	3.785	Liter (L)
Volume	cubic feet (ft^3)	0.02832	cubic meters (m^3)
Volume	cubic yards (yd^3)	0.7646	cubic meters (m^3)
Volume	acre-feet (ac-ft)	1233	cubic meter (m^3)
Mass	pound (lb)	0.4536	kilogram (kg)
Power/Energy	calories (cal)	4.187	joules (J)
Energy	Btu	1.055	kilojoules (kJ)
Energy	horsepower (hp)	0.7457	kilowatts (kW)
Pressure	atmosphere (atm)	101.3	kilopascal (kPa)
Pressure	feet water (ft H_20)	0.3048	meters of head (m H_20)
Pressure	lbs/square inch (psi)	6.895	kilopascal (kPa)
Pressure	inches mercury (in Hg)	3.377	kilopascal (kPa)
Flow	mgd	3.785	megaliter per day (ML/d)
Flow	gpm	3.785	liter per minute (L/min)
Flow	cfs	0.02832	cubic meter per second (m^3/s)
Flow	cfm (liquid)	0.4719	liter per second (L/s)
Flow	cfm (gas)	0.02832	cubic meter per minute (m^3/min)
Velocity	ft/sec	0.3048	meter per second (m/s)
Velocity	ft/min	0.3048	meter per minute (m/min)
Loading rates	gpd/sq ft	0.0016984	m^3/m^2h or m/h
Loading rates	gpm/sq ft	2.444	m^3/m^2 .h or m/h
Loading rates	lb/d/sq ft	4.882	kg/m^2.d
Loading rates	lb/1000 cu ft	0.01602	kg/m^3
Loading rates	lb/1000 cu ft/day	0.01602	kg/m^3 .d

Note: Currently both ML/d and m^3/d are used in the industry—use ML/d for simplicity.

APPENDIX E

Physical Properties of Water

Temp., °F	Temp., °C	Specific Volume, cu ft/lb	39.2°F Reference	60°F Reference	68°F Reference	Wt. in lb/cu ft	Vapor Pressure, psi abs
32	0	0.01602	1.000	1.001	1.002	62.42	0.088
35	1.7	0.01602	1.000	1.001	1.002	62.42	0.100
40	4.4	0.01602	1.000	1.001	1.002	62.42	0.1217
50	10.0	0.01603	0.999	1.001	1.002	62.38	0.1781
60	15.6	0.01604	0.999	1.000	1.001	62.34	0.2563
70	21.1	0.01606	0.998	0.999	1.000	62.27	0.3631
80	26.7	0.01608	0.996	0.998	0.999	62.19	0.5069
90	32.2	0.01610	0.995	0.996	0.997	62.11	0.6982
100	37.8	0.01613	0.993	0.994	0.995	62.00	0.9492
120	48.9	0.01620	0.989	0.990	0.991	61.73	1.692
140	60.0	0.01629	0.983	0.985	0.986	61.39	2.889
160	71.1	0.01639	0.977	0.979	0.979	61.01	4.741
180	82.2	0.01651	0.970	0.972	0.973	60.57	7.510
200	93.3	0.01663	0.963	0.964	0.966	60.13	11.526
212	100.0	0.01672	0.958	0.959	0.960	59.81	14.696
220	104.4	0.01677	0.955	0.956	0.957	59.63	17.186
240	115.6	0.01692	0.947	0.948	0.949	59.10	24.97
260	126.7	0.01709	0.938	0.939	0.940	58.51	35.43
280	137.8	0.01726	0.928	0.929	0.930	58.00	49.20
300	148.9	0.01745	0.918	0.919	0.920	57.31	67.01

APPENDIX F

Compatibility of Various Chemicals

This appendix contains tables to provide the design engineer with information regarding the compatibility of chemicals and materials.

When designing chemical piping systems, use Tables F–1, F–2, and F–3 for chemical compatibility recommendations. Table F–1 presents plastic pipe chemical compatibility for PVC, CPVC, polypropylene (PP), PVDF, and polyethylene (PE). Table F–2 presents plastic seat/seal chemical compatibility for teflon, viton, EPDM, buna-N, hypalon, nylon, and neoprene. Table F–3 presents 304, 316, and 416 stainless steel, hastelloy C, cast iron, steel, bronze, monel, and titanium.

In Tables F–1, F–2, and F–3, the following definitions apply:

A = Very good resistance, being used successfully
B = Moderate resistance, use with caution
X = Not recommended

Table F–4 shows chemicals that are considered incompatible and should be kept apart for storage and handling. In Table F–4 an X denotes incompatible chemicals.

REFERENCES

1. Harrington's Engineering Handbook, Harrington Industrial Plastics, Inc., 1997.
2. Materials Selection Guide, BIF, 1985.
3. Corrosion Data Guide, Atlas Stainless Steels, 1992.
4. Materials Selection Chart, Milton Roy, 1991.
5. Materials Selection Guide, Grinnell, 1988.
6. Materials Selection Guide, Durco.
7. Chemical Comparison Table, Fab-Tech Inc., 1997.

TABLE F–1. Plastic Pipe Chemical Compatibility

Concentration	CHEMICALS	PVC	CPVC	PP	PVDF	PE
	References:	1,2,4,6	1,7	1,2,4	1,5,6	1,4
<10%	Alum (Aluminum Sulfate)	A	A	A	A	A
50%	Alum (Aluminum Sulfate)	A	A	A	A	—
	Aluminum Phosphate	—	—	—	—	A
15%	Ammonia	B	A	A	A	X
25%	Ammonia	B	A	A	A	—
99%	Ammonia, aqua	X	X	B	A	—
100%	Ammonia, anhydrous	B	A	A	A	X
30%	Ammonium Phosphate	A	A	A	A	A
	Calcium Carbonate	A	A	A	A	A
5%	Calcium Hydroxide	B	A	A	A	A
	Calcium Hypochlorite	B	B	A	A	A
	Calcium Oxide (Lime)	B	A	A	A	—
	Chlorinated Water	B	A	X	B	—
	Chlorine Gas (dry)	X	X	X	A	X
	Chlorine Gas (wet)	X	X	X	A	X
	Chlorine Liquid	X	X	X	A	X
	Diesel Fuel	B	B	B	A	—
	Ethylene Glycol	A	A	A	A	X
100%	Ferric Chloride, anhydrous	A	A	A	A	A
45%	Ferric Chloride	A	A	A	A	A
	Ferric Sulfate	A	A	A	A	A
30%	Ferrous Chloride	A	A	A	A	A
	Ferrous Sulfate	A	A	A	A	A
	Hydraulic Oil	A	—	X	A	—
10%	Hydrochloric Acid	A	A	A	A	A
20%	Hydrochloric Acid	A	A	B	A	A
25%	Hydrochloric Acid	A	A	A	A	A
37%	Hydrochloric Acid	A	A	X	A	A
5%	Hydrogen Peroxide	B	X	A	A	A
10%	Hydrogen Peroxide	B	A	B	A	A
30%	Hydrogen Peroxide	B	A	B	A	A
50%	Hydrogen Peroxide	B	A	B	A	—
90%	Hydrogen Peroxide	X	X	X	X	—
100%	Hydrogen Peroxide	A	A	B	A	—
	Isopropanol (Isopropyl Alcohol)	A	A	A	A	—
	Methanol	B	A	A	A	A
	Mineral Oil	A	A	X	A	X
	Natural Gas	A	B	B	A	—
	Oxygen Gas	A	A	A	A	—
	Ozone	B	B	B	A	X
10%	Phosphoric Acid	A	A	A	A	A
20%	Phosphoric Acid	A	A	A	A	A
40%	Phosphoric Acid	B	A	A	B	A
50%	Phosphoric Acid	A	A	A	A	A
80%	Phosphoric Acid	X	A	X	A	B
85%	Phosphoric Acid	X	B	X	X	B
100%	Phosphoric Acid	X	B	B	X	—
	Polymer, dry (water dilution)	A	A	X	—	X
	Polymer, emulsion (oil based)	A	A	X	—	X

TABLE F–1. (*Continued*)

Concentration	CHEMICALS	PVC	CPVC	PP	PVDF	PE
	References:	1,2,4,6	1,7	1,2,4	1,5,6	1,4
	Polymer, Mannich (water based)	A	A	X	—	X
	Polymer, solution (water based)	A	A	X	—	X
	Potassium Carbonate (Potash)	A	A	A	A	A
	Sodium Bicarbonate	A	A	A	A	A
	Sodium Bisulfite	A	A	A	A	A
	Sodium Carbonate (Soda Ash)	A	A	A	A	A
5%	Sodium Fluoride	A	A	A	A	A
10%	Sodium Hydroxide	A	—	A	A	—
15%	Sodium Hydroxide	A	A	A	A	X
20%	Sodium Hydroxide	A	A	A	B	X
30%	Sodium Hydroxide	A	A	A	X	X
50%	Sodium Hydroxide	A	A	B	X	X
70%	Sodium Hydroxide	A	A	A	X	X
5%	Sodium Hypochlorite	A	A	A	A	A
20%	Sodium Hypochlorite	B	A	X	A	X
	Sulfur Dioxide Gas (dry)	A	X	B	A	B
	Sulfur Dioxide Gas (wet)	A	X	B	A	B
10%	Sulfuric Acid	A	A	A	A	B
30%	Sulfuric Acid	A	A	A	A	B
50%	Sulfuric Acid	A	A	B	A	X
60%	Sulfuric Acid	A	A	B	A	X
70%	Sulfuric Acid	A	A	B	A	X
80%	Sulfuric Acid	X	X	X	A	X
90%	Sulfuric Acid	X	X	X	A	X
95%	Sulfuric Acid	X	X	X	A	X
98%	Sulfuric Acid	X	X	X	A	X
100%	Sulfuric Acid	X	X	X	A	X
	Water, Acid Mine	B	A	A	A	—
	Water, Deionized	A	A	A	A	A
	Water, Demineralized	A	A	A	A	A
	Water, Distilled	A	A	A	A	A
	Water, Potable, 180 deg F	X	A	A	A	X
	Water, Potable, 225 deg F	X	X	X	A	X
	Water, Salt	B	A	A	A	A
	Water, Sewage	A	A	A	A	A

TABLE F-2. Plastic Seat/Seal Chemical Compatibility

Concentration	CHEMICALS	Teflon	Viton	EPDM	Buna N	Hypalon	Nylon	Neoprene
	References:	1,2,5,6	1,2,4,5,6	1,4,5,6	1,5	1,2,6	5	1
<10%	Alum (Aluminum Sulfate)	A	A	A	A	A	B	150
50%	Alum (Aluminum Sulfate)	A	A	A	—	A	A	A
	Aluminum Phosphate	—	—	—	—	—	—	—
15%	Ammonia	A	X	A	B	X	A	A
25%	Ammonia	A	X	A	A	X	A	A
99%	Ammonia, aqua	A	X	A	—	A	—	—
100%	Ammonia, anhydrous	A	X	A	B	X	B	200
30%	Ammonium Phosphate	A	A	B	B	B	X	140
	Calcium Carbonate	A	A	B	B	B	A	100
5%	Calcium Hydroxide	A	A	A	B	A	A	70
	Calcium Hypochlorite	A	B	B	X	A	X	X
	Calcium Oxide (Lime)	A	B	B	A	A	B	160
	Chlorinated Water	A	X	X	X	X	X	X
	Chlorine Gas (dry)	A	B	X	X	X	X	X
	Chlorine Gas (wet)	A	X	X	X	X	X	X
	Chlorine Liquid	A	X	—	X	X	—	—
	Diesel Fuel	A	B	X	B	X	A	X
	Ethylene Glycol	A	A	A	A	A	A	160
100%	Ferric Chloride, anhydrous	A	A	A	A	A	—	160
45%	Ferric Chloride	A	A	B	X	B	X	
	Ferric Sulfate	A	A	A	A	A	X	200
30%	Ferrous Chloride	A	A	A	A	A	X	80
	Ferrous Sulfate	A	B	A	A	B	X	200
	Hydraulic Oil	A	A	X	B	B	A	70
10%	Hydrochloric Acid	A	B	B	X	A	A	X
20%	Hydrochloric Acid	A	B	B	X	A	B	80
25%	Hydrochloric Acid	A	B	B	X	A	X	X
37%	Hydrochloric Acid	A	B	X	X	B	X	X

5%	Hydrogen Peroxide	A	B	X	—	X	—	—
10%	Hydrogen Peroxide	A	B	X	X	X	X	X
30%	Hydrogen Peroxide	A	B	X	X	X	X	X
50%	Hydrogen Peroxide	A	B	X	X	B	X	X
90%	Hydrogen Peroxide	A	B	X	X	B	X	X
100%	Hydrogen Peroxide	A	B	X	X	B	X	X
	Isopropanol (Isopropyl Alcohol)	A	A	A	X	A	B	70
	Methanol	A	X	B	A	A	B	140
	Mineral Oil	A	A	X	B	B	A	70
	Natural Gas	A	A	X	B	A	A	140
	Oxygen Gas	A	A	A	B	A	A	200
	Ozone	A	A	A	X	A	X	80
10%	Phosphoric Acid	A	A	B	X	B	A	120
20%	Phosphoric Acid	A	A	B	X	B	—	70
40%	Phosphoric Acid	A	A	B	X	B	B	70
50%	Phosphoric Acid	A	A	B	X	B	—	X
80%	Phosphoric Acid	A	A	B	X	B	—	X
85%	Phosphoric Acid	A	A	B	X	B	—	X
100%	Phosphoric Acid	A	B	B	X	X	—	X
	Polymer, dry (water dilution)	A	A	X	A	X	—	—
	Polymer, emulsion (oil based)	A	A	X	A	X	—	—
	Polymer, Mannich (water based)	A	A	X	A	X	—	—
	Polymer, solution (water based)	A	A	X	A	X	—	—
	Potassium Carbonate (Potash)	A	A	A	A	A	A	160
	Sodium Bicarbonate	A	A	A	A	A	A	160
	Sodium Bisulfite	A	A	A	B	B	A	140
	Sodium Carbonate (Soda Ash)	A	A	A	A	A	B	140

TABLE F-2. **(*Continued*)**

Concentration	CHEMICALS	Teflon	Viton	EPDM	Buna N	Hypalon	Nylon	Neoprene
	References:	1,2,5,6	1,2,4,5,6	1,4,5,6	1,5	1,2,6	5	1
5%	Sodium Fluoride	A	A	A	B	B	B	70
10%	Sodium Hydroxide	A	X	B	—	A	—	—
15%	Sodium Hydroxide	A	X	B	A	A	—	160
20%	Sodium Hydroxide	A	X	B	B	A	A	160
30%	Sodium Hydroxide	A	X	B	B	A	—	160
50%	Sodium Hydroxide	A	X	B	X	A	B	160
70%	Sodium Hydroxide	A	X	B	X	B	X	100
5%	Sodium Hypochlorite	A	B	B	X	A	X	X
20%	Sodium Hypochlorite	A	B	X	X	B	—	70
	Sulfur Dioxide Gas (dry)	A	X	B	X	X	B	X
	Sulfur Dioxide Gas (wet)	A	X	A	X	X	—	X
10%	Sulfuric Acid	A	A	B	X	B	X	100
30%	Sulfuric Acid	A	A	X	X	B	X	100
50%	Sulfuric Acid	A	A	X	X	B	X	100
60%	Sulfuric Acid	A	A	X	X	X	X	100
70%	Sulfuric Acid	A	A	X	X	X	X	X
80%	Sulfuric Acid	A	B	X	X	X	X	X
90%	Sulfuric Acid	A	B	X	X	X	X	X
95%	Sulfuric Acid	A	B	X	X	X	X	X
98%	Sulfuric Acid	A	B	X	X	X	X	X
100%	Sulfuric Acid	A	B	X	X	X	X	X
	Water, Acid Mine	A	A	B	B	A	A	160
	Water, Deionized	A	A	A	A	A	—	160
	Water, Demineralized	A	A	A	A	A	A	160
	Water, Distilled	A	A	A	A	A	—	160
	Water, Potable, 180 deg F	A	X	A	A	A	A	X
	Water, Potable, 225 deg F	A	X	B	X	X	X	X
	Water, Salt	A	A	A	A	A	A	160
	Water, Sewage	A	A	A	A	A	A	160

TABLE F–3. Metal Chemical Compatibility

Concentration	CHEMICALS	316 SS	304 SS	Hastelloy C	Iron/ Cast Iron	Steel	Bronze	416 SS	Monel 400	Titanium
	References:	1,2,3,4,5,6	1,3,4	1,2,4,6	2,4	2,4,6	5	5	6	1
<10%	Alum (Aluminum Sulfate)	A	B	A	X	X	X	B	B	A
50%	Alum (Aluminum Sulfate)	B	X	A	X	X	—	—	B	A
	Aluminum Phosphate	—	—	—	—	—	—	—	—	—
15%	Ammonia	A	B	A	—	A	X	B	—	—
25%	Ammonia	B	B	—	—	—	—	—	—	—
99%	Ammonia, aqua	A	A	A	A	A	—	—	—	—
100%	Ammonia, anhydrous	A	A	A	A	A	X	B	B	—
30%	Ammonium Phosphate	A	A	A	X	X	X	B	A	—
	Calcium Carbonate	A	A	B	—	B	—	—	—	A
5%	Calcium Hydroxide	A	A	A	A	A	X	A	A	A
	Calcium Hypochlorite	X	X	B	X	X	—	B	X	—
	Calcium Oxide (Lime)	X	X	A	A	A	—	—	—	—
	Chlorinated Water	X	X	B	X	X	—	—	X	—
	Chlorine Gas (dry)	B	X	A	— (a)	A	X	X	A	—
	Chlorine Gas (wet)	X	X	A	— (a)	X	—	X	X	—
	Chlorine Liquid	X	—	A	X (a)	X	—	—	—	—
	Diesel Fuel	A	—	—	—	—	B	A	—	—
	Ethylene Glycol	A	A	A	A	A	A	A	A	—
100%	Ferric Chloride, anhydrous	X	X	X	X	X	—	—	X	—
45%	Ferric Chloride	X	X	B	X	X	X	X	X	—
	Ferric Sulfate	A	B	A	X	X	X	—	X	—
30%	Ferrous Chloride	X	X	B	X	X	—	X	X	—
	Ferrous Sulfate	A	X	A	X	X	X	B	—	—
	Hydraulic Oil	A	—	A	A	A	A	B	—	—
10%	Hydrochloric Acid	X	X	B	X	X	X	X	X	—
20%	Hydrochloric Acid	X	X	B	X	X	X	X	X	—
25%	Hydrochloric Acid	X	X	B	X	X	X	X	X	—
37%	Hydrochloric Acid	X	X	B	—	X	X	X	X	—

TABLE F-3. (*Continued*)

Concentration	CHEMICALS	316 SS	304 SS	Hastelloy C	Iron/ Cast Iron	Steel	Bronze	416 SS	Monel 400	Titanium
	References:	1,2,3,4,5,6	1,3,4	1,2,4,6	2,4	2,4,6	5	5	6	1
5%	Hydrogen Peroxide	A	B	B	X	X	—	—	A	—
10%	Hydrogen Peroxide	A	X	B	X	X	—	—	A	—
30%	Hydrogen Peroxide	X	X	B	X	X	—	—	X	—
50%	Hydrogen Peroxide	X	X	X	—	X	—	—	X	—
90%	Hydrogen Peroxide	X	B	X	—	X	X	X	X	—
100%	Hydrogen Peroxide	X	B	X	—	X	—	—	X	—
	Isopropanol (Isopropyl Alcohol)	A	A	A	B	B	A	A	B	—
	Methanol	A	A	A	A	A	A	—	A	—
	Mineral Oil	B	A	A	A	A	—	—	—	—
	Natural Gas	A	A	—	—	—	A	A	—	—
	Oxygen Gas	A	A	A	—	—	A	A	—	—
	Ozone	A	A	A	—	—	—	B	—	—
10%	Phosphoric Acid	B	B	A	X	X	—	X	X	—
20%	Phosphoric Acid	B	B	A	X	X	X	—	X	—
40%	Phosphoric Acid	B	B	A	X	X	—	—	X	—
50%	Phosphoric Acid	B	B	A	X	X	—	—	X	—
80%	Phosphoric Acid	B	B	A	X	X	—	—	X	—
85%	Phosphoric Acid	B	A	X	X	X	—	—	X	—
100%	Phosphoric Acid	A	A	X	—	X	—	—	X	—
	Polymer, dry (water dilution)	—	—	—	—	—	—	—	—	—
	Polymer, emulsion (oil based)	—	—	—	—	—	—	—	—	—
	Polymer, Mannich (water based)	—	—	—	—	—	—	—	—	—
	Polymer, solution (water based)	—	—	—	—	—	—	—	—	—
	Potassium Carbonate (Potash)	A	A	A	B	B	A	—	—	—
	Sodium Bicarbonate	A	A	A	A	A	B	A	A	—
	Sodium Bisulfite	A	A	A	X	X	—	—	X	—
	Sodium Carbonate (Soda Ash)	A	A	A	B	B	B	B	A	—

5%	Sodium Fluoride	B	B	B	X	X	X	—	A	—
10%	Sodium Hydroxide	A	A	A	A	A	—	—	A	—
15%	Sodium Hydroxide	A	A	A	—	A	—	—	A	—
20%	Sodium Hydroxide	A	A	A	B	B	X	X	A	—
30%	Sodium Hydroxide	A	A	A	—	B	—	—	A	—
50%	Sodium Hydroxide	X	B	A	B	B	X	X	A	—
70%	Sodium Hydroxide	X	B	A	—	X	X	X	A	—
5%	Sodium Hypochlorite	X	B	A	X	X	X	—	X	—
20%	Sodium Hypochlorite	X	X	A	X	X	—	—	X	—
	Sulfur Dioxide Gas (dry)	A	A	A	A	A	X	B	—	—
	Sulfur Dioxide Gas (wet)	B	B	A	A	A	—	—	—	—
10%	Sulfuric Acid	X	X	A	X	X	X	X	A	—
30%	Sulfuric Acid	X	X	A	X	X	X	X	A	—
50%	Sulfuric Acid	X	X	A	X	X	X	X	A	—
60%	Sulfuric Acid	X	X	A	X	X	X	X	A	—
70%	Sulfuric Acid	X	X	A	X	X	X	X	X	—
80%	Sulfuric Acid	X	X	A	X	X	X	X	X	—
90%	Sulfuric Acid	X	X	A	X	X	X	X	X	—
95%	Sulfuric Acid	X	X	A	X	X	X	X	X	—
98%	Sulfuric Acid	X	X	A	B	X	X	X	X	—
100%	Sulfuric Acid	B	X	A	B	X	X	X	X	—
	Water, Acid Mine	A	A	A	—	—	—	—	—	—
	Water, Deionized	A	A	A	X	X	—	—	—	—
	Water, Demineralized	A	A	A	X	X	A	—	—	—
	Water, Distilled	—	A	A	—	—	—	—	—	—
	Water, Potable, 180 deg F	A	A	A	—	—	A	A	—	—
	Water, Potable, 225 deg F	A	—	—	—	—	A	A	—	—
	Water, Salt	X	X	X	X	X	B	X	—	—
	Water, Sewage	A	B	A	—	—	A	B	—	—

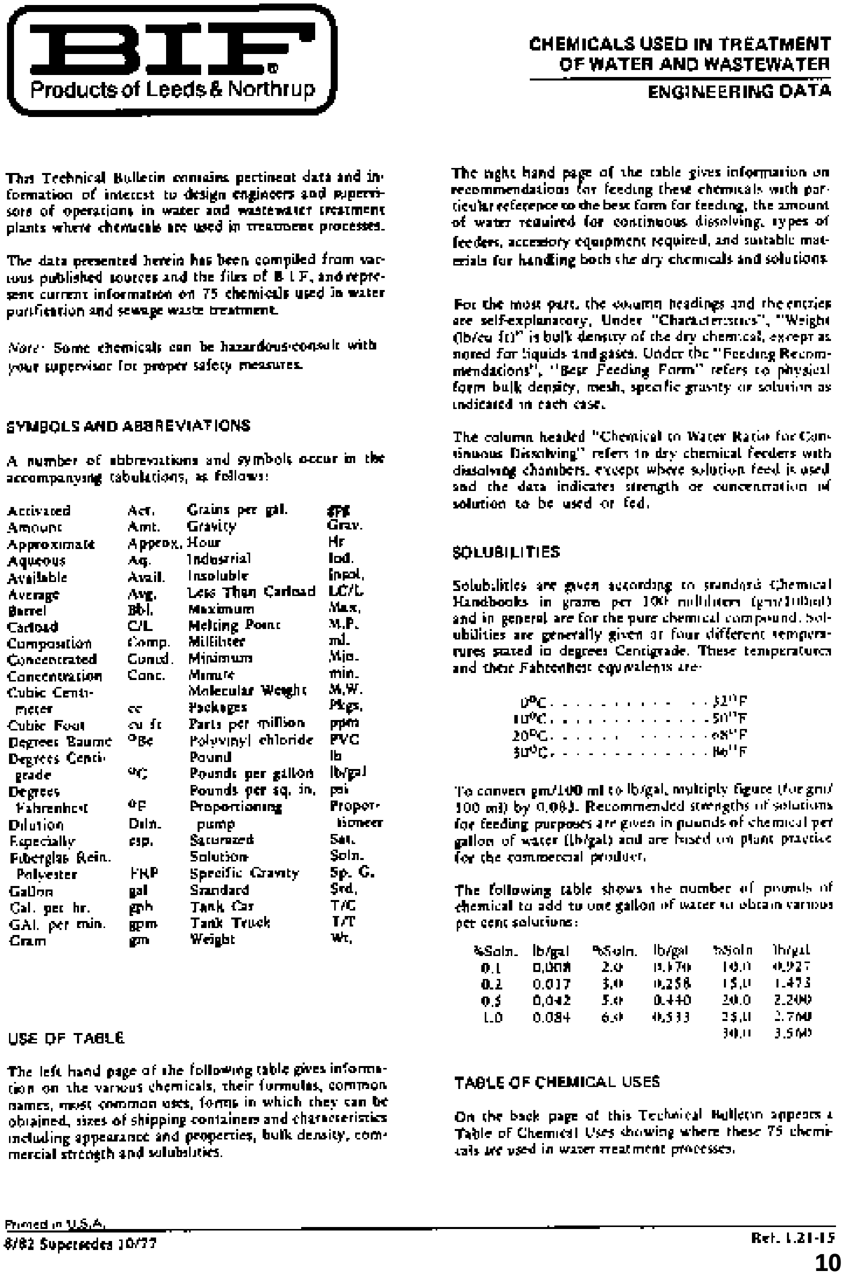
TABLE F-4. Compatibility of Commonly Used Chemicals

		Alum	PACl	HFA	FSA	$FeCl_3$	H_2SO_4	PAC	Polymers	Silicate	NH_3	NH_4OH	CaOH	Soda Ash	NaOH	$KMnO_4$	Cl_2	ClO_2	O_3	NaOCl	H_2O_2
Aluminum Sulfate	Alum			X			X			X	X	X	X	X	X					X	
Polyaluminum Chloride	PACl			X			X			X	X	X	X	X	X			X		X	
Hydrofluoric Acid	HFA	X	X			X	X	X	X	X	X	X	X	X	X	X				X	X
Hydrofluosilicic Acid	FSA							X	X	X	X	X	X	X	X	X				X	
Ferric Chloride	$FeCl_3$			X						X	X	X	X	X	X	X	X	X	X	X	X
Sulfuric Acid	H_2SO_4	X	X	X						X	X	X	X	X	X	X				X	X
Powdered Activated Carbon	PAC			X	X											X	X	X	X	X	X
Polymers	Polymers			X	X											X	X	X	X	X	X
Sodium Silicate	Silicate	X	X	X	X	X	X									X	X			X	
Anhydrous Ammonia	NH_3	X	X	X	X	X	X						X	X	X	X	X	X	X	X	X
Ammonium Hydroxide	NH_4OH	X	X	X	X	X	X						X	X	X	X	X	X	X	X	X
Calcium Hydroxide	CaOH	X	X	X	X	X	X				X	X									X
Sodium Carbonate	Soda Ash	X	X	X	X	X	X				X	X									X
Sodium Hydroxide	NaOH	X	X	X	X	X	X				X	X									X
Potassium Permanganate	$KMnO_4$			X	X	X	X	X	X	X	X	X									X
Chlorine	Cl_2					X		X	X	X	X	X									
Chlorine Dioxide	ClO_2					X		X	X		X	X									
Ozone	O_3					X		X	X		X	X									
Sodium Hypochlorite	NaOCl	X	X	X	X	X	X	X	X	X	X	X									
Hydrogen Peroxide	H_2O_2			X		X	X	X	X		X	X	X	X	X	X					

X = INCOMPATIBLE

APPENDIX G

Chemicals Used in Treatment of Water and Wastewater



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CHEMICALS USED IN TREATMENT
OF WATER AND WASTEWATER
ENGINEERING DATA

This Technical Bulletin contains pertinent data and information of interest to design engineers and supervisors of operations in water and wastewater treatment plants where chemicals are used in treatment processes.

The data presented herein has been compiled from various published sources and the files of B-I-F, and represent current information on 75 chemicals used in water purification and sewage waste treatment.

Note: Some chemicals can be hazardous-consult with your supervisor for proper safety measures.

SYMBOLS AND ABBREVIATIONS

A number of abbreviations and symbols occur in the accompanying tabulations, as follows:

Activated	Act.
Amount	Amt.
Approximate	Approx.
Aqueous	Aq.
Available	Avail.
Average	Avg.
Barrel	Bbl.
Carload	C/L
Composition	Comp.
Concentrated	Concd.
Concentration	Conc.
Cubic Centimeter	cc
Cubic Foot	cu ft
Degrees Baume	°Be
Degrees Centigrade	°C
Degrees Fahrenheit	°F
Dilution	Diln.
Especially	esp.
Fiberglas Rein. Polyester	FRP
Gallon	gal
Gal. per hr.	gph
GAl. per min.	gpm
Gram	gm
Grains per gal.	gpg
Gravity	Grav.
Hour	Hr
Industrial	Ind.
Insoluble	Insol.
Less Than Carload	LC/L
Maximum	Max.
Melting Point	M.P.
Milliliter	ml.
Minimum	Min.
Minute	min.
Molecular Weight	M.W.
Packages	Pkgs.
Parts per million	ppm
Polyvinyl chloride	PVC
Pound	lb
Pounds per gallon	lb/gal
Pounds per sq. in.	psi
Proportioning pump	Proportioneer
Saturated	Sat.
Solution	Soln.
Specific Gravity	Sp. G.
Standard	Std.
Tank Car	T/C
Tank Truck	T/T
Weight	Wt.

USE OF TABLE

The left hand page of the following table gives information on the various chemicals, their formulas, common names, most common uses, forms in which they can be obtained, sizes of shipping containers and characteristics including appearance and properties, bulk density, commercial strength and solubilities.

The right hand page of the table gives information on recommendations for feeding these chemicals with particular reference to the best form for feeding, the amount of water required for continuous dissolving, types of feeders, accessory equipment required, and suitable materials for handling both the dry chemicals and solutions.

For the most part, the column headings and the entries are self-explanatory. Under "Characteristics", "Weight (lb/cu ft)" is bulk density of the dry chemical, except as noted for liquids and gases. Under the "Feeding Recommendations", "Best Feeding Form" refers to physical form bulk density, mesh, specific gravity or solution as indicated in each case.

The column headed "Chemical to Water Ratio for Continuous Dissolving" refers to dry chemical feeders with dissolving chambers, except where solution feed is used and the data indicates strength or concentration of solution to be used or fed.

SOLUBILITIES

Solubilities are given according to standard Chemical Handbooks in grams per 100 milliliters (gm/100ml) and in general are for the pure chemical compound. Solubilities are generally given at four different temperatures stated in degrees Centigrade. These temperatures and their Fahrenheit equivalents are:

0°C . . . . 32°F
10°C . . . . 50°F
20°C . . . . 68°F
30°C . . . . 86°F

To convert gm/100 ml to lb/gal, multiply figure (for gm/100 ml) by 0.083. Recommended strengths of solutions for feeding purposes are given in pounds of chemical per gallon of water (lb/gal) and are based on plant practice for the commercial product.

The following table shows the number of pounds of chemical to add to one gallon of water to obtain various per cent solutions:

%Soln.	lb/gal	%Soln.	lb/gal	%Soln	lb/gal
0.1	0.008	2.0	0.170	10.0	0.927
0.2	0.017	3.0	0.258	15.0	1.473
0.5	0.042	5.0	0.440	20.0	2.200
1.0	0.084	6.0	0.533	25.0	2.780
				30.0	3.560

TABLE OF CHEMICAL USES

On the back page of this Technical Bulletin appears a Table of Chemical Uses showing where these 75 chemicals are used in water treatment processes.

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CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
ACTIVATED CARBON (Nuchar, Norit, Darco, Carbodur) Decolorizing, Taste and odor Removal Dosage between 5 to 80 ppm	Powder Granules	Bags - 35 lb (3x21x39 in) Drums - 5 lb & 25 lb Bulk - C/L	Black powder, about 400 mesh Dusty, smoulders if ignited Arches in hoppers, floodable(1) Do not mix with $KMnO_4$, hypochlorite or ClO_2. pH varies, usually weakly acidic	Powder 8 to 28 Avg. 12	10% C (Bone charcoal) to 90% C (Wood charcoal)	Insoluble - forms a slurry
ACTIVATED SILICA SiO_2 (Silica Soln.) Coagulation Aid Dosage - 1 to 10 ppm - Raw H_2O (5-25 ppm-Waste)	Produced at site as needed from sodium silicate and activating agent	—	Clear often opalescent Na_2SiO_3 may be activated by alum, ammonium sulfate, chlorine, carbon dioxide. Sodium bicarbonate, sulfuric acid or Na_2SiF_6	41°Be sod. silicate Approx. 11.6 lb/gal	41°Be sodium silicate is used and diluted to 1% SiO_2 before use as coag. aid	Act. Silica is a colloidal soln. will gel at high conc.
ALUMINUM AMMONIUM SULFATE $Al_2(SO_4)_3(NH_4)_2SO_4 \cdot 24H_2O$ (Ammonia Alum) Coagulation, especially with pressure filters	Lump Granular Ground Powder	Fiber Drums Kegs - 100-400 lb Bags - 100 lb Bulk	Colorless crystals Strong astringent taste 1% Soln. - pH 3.5	60 to 70	11% Al_2O_3	3.9 at 0°C 9.5 at 10°C 15.1 at 20°C 20.0 at 30°C
ALUMINUM CHLORIDE $AlCl_3 \cdot 6H_2O$ Sludge Conditioner	Solution 15 to 35%	Carboys Rubber lined tank trucks	Clear pale yellow liquid Aq. Soln. is acid Always fed as solution.	Sp. Gr. 1.5 to 1.16	32°Be or 35% $AlCl_3$	Completely Miscible
ALUMINUM POTASSIUM SULFATE $Al_2(SO_4)_3 \cdot K_2SO_4 \cdot 24H_2O$ (Potassium Alum) Coagulation	Lump Granular Rice Ground Powder	Bags - 100 lb Bbl. - 350 & 350 lb Drums - 100 lb & 350 lb Bulk - C/L	White crystals Low even solubility M.P. - about 200°F 1% Soln. pH 3.3 Above 60-65°C loses $9H_2O$ Crystals are efflorescent.	60 to 70	10 to 11% Al_2O_3	5.7 at 0°C 7.6 at 10°C 11.4 at 20°C 16.6 at 30°C
ALUMINUM SULFATE $Al_2(SO_4)_3 \cdot 14H_2O$ Alum, Filter Alum Coagulation at pH 5.5 to 8.0 Dosage between 0.5 to 9 gpg. Precipitate PO_4 (90-110 ppm)	Lump Granular Rice Ground Powder	Bags - 100 lb & 200 lb Bbl. - 325 lb & 400 lb Drums - 25 lb, 100 lb & 250 lb Bulk - C/L	Light tan to grey-green Dusty, astringent Only slightly hygroscopic 1% Soln. - pH 3.4	60 to 75 (Powder is lighter) To calculate hopper capacities, use 60	98% plus or 17% Al_2O_3 (Min.)	72.5 at 0°C 78.0 at 10°C 87.3 at 20°C 101.6 at 30°C
ALUM $Al_2(SO_4)_3 \cdot XH_2O$ Liquid 1 gal 36°Be=5.38 lb of dry Alum - 60°F Coag. at pH 5.5 to 8.0 Sludge Conditioner Precipitate PO_4	Solution 32.2°Be to 37°Be	Manufactured near site, 6000-8000 gal steel T/C; 2000-4000 gal rubber lined steel tank trucks. High freight cost precludes distant shipment	Light green to light brown Soln. F.P. or crystallization point for: 35.97°Be= 4°F 36.95°Be= 27°F 37.7°Be= 60°F 1% Soln. - pH 3.4 Visc. 36°Be at 60°F= 25 cp.	36°Be Sp. G. = 1.33 or 11.1 lb/gal at 60°F	At 60°F 32.2°Be 7.25% Al_2O_3 35.97°Be 8.23% Al_2O_3 37°Be 8.5% Al_2O_3	Completely Miscible

(1) "Floodability" as used in this table with dry ch... means that under some conditions, the material entrains air and becomes "fluidized" so that it will flow through small openings, like water.

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPE OF FEEDERS FOR	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS (2)
Powder - with bulk density of 12 lb/cu ft Slurry - 1 lb/gal	According to its bulkiness and wettability, a 10 to 15% solution would be maximum concentration	Volumetric Helix Slurry Rotodip *Diaphragm Metering Pumps Hydraulic 1700 & Mechanical 1200	Washdown type wetting tank Vortex mixer Hopper agitators Non-flood rotors Dust collectors Large storage cap. for liquid feed Tank agitators Transfer Pumps If no surface mixer, need surface jets	316 ss, rubber, bronze, Monel, Hastelloy C, FRP(3) Saran, Hypalon, Titanium
0.6 to 1.0% Soln. to prevent gel formation Dilute for transport by eductor or pump	Diluted "SiO_2" to process. Max. (100 gal aging tank) Capacity Factor = 141 Alum 16.4 gpm H_2SO_4 24.9 gpm $(NH_4)_2SO_4$ 9.9 gpm Cl_2 15.3 gpm $NaHCO_3$ 9.9 gpm	Liquid (continuous) Silica activator or batch systems Diaphragm metering Pump - Series 1700 & 1200 & 5700 Rotodip	Flushing equipment Batch mixing tanks Storage or feeding tank Aging tank	Iron, steel, rubber, stainless steel, Viton, Hypalon (dilute), Tyril
Lump for pressure filters with solution pot. Ground for dry feeders	0.5 lb/gal	Usually fed in pot type feeders for pressure filters Gravimetric or volumetric feeders may be used - see aluminum sulfate feeders	Dissolvers	Lead, rubber, ceramic, PVC-1 Saran, FRP, Uscolite, Carp. 20 ss, Hypalon, Halar, Ti
As shipped. Solution does dissociate.	—	Solution Rotodip Diaphragm Metering Pumps Series 1200 & 1700 5700	Storage tanks Transfer pumps	Carp. 20 ss (10% at room temp.) (pits), rubber, FRP, PVC-1, Vinyl, Saran, Hastelloy B, Hypalon, ceramic, Teflon, Tyril, Uscolite, Ti (25%, Rm Noryl, Derakane Halar, Polypropylene
Ground	0.5 lb/gal	Usually fed in pot type feeders for pressure filters Gravimetric or volumetric feeders may be used - see aluminum sulfate feeders	—	Lead-lined tanks, rubber, ceramic, FRP, Saran, Hypalon, Epoxy, Carp. 20 ss, Uscolite
Ground, granular or rice. Powder is dusty, arches and is floodable	0.5 lb/gal Dissolver detention time 5 minutes for ground (10 minutes for granules)	Volumetric Helix Solution Diaphragm metering pump Series 1200 & 1700 & 5700	Dissolver Mechanical mixer Scales for volumetric feeders Dust Collectors	Lead, rubber, FRP, PVC-1, 316 ss, Carp. 20 ss, Vinyl, Hypalon, Epoxy, Ni-Resist glass, ceramic, Polyethylene, Tyril, Uscolite, Ti, Noryl Derakane, Halar, Polypropylene
Full strength under controlled temp. or dilute to avoid crystallization Minimize surface evap. - causes flow problems Keep % dry Alum below 50% to avoid crystallization	Dilute to between 3 to 15% - according to application or conditions, mixing, etc.	Solution Rotodip Diaphragm Metering Pump Series 1200 & 1700 & 5700	Tank gauges or scales Transfer pumps Storage tank Temperature control Eductors or dissolvers for dilution	Lead or rubber-lined tanks, Duriron, FRP, Saran, PVC-1, Vinyl, Hypalon, 316 ss, Carp. 20 ss, Tyril, Ti, Derakane, Halar, Polypropylene

*Requires Ball Valves

(2) Iron and steel can be used with chemicals in the dry state unless the chemical is deliquescent or very hygroscopic, or in a dampish form and thus corrosive to some degree.

(3) FRP, in every case, refers to the chemically resistant grade (bisphenol A+) of Fiberglass Rein. Polyester.

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
AMINES, NEUTRALIZING 1. Benzylamine (amino-toluene) C_7H_9N 2. Cyclohexylamine (Hexamine) $C_6H_{13}N$ 3. Morpholine (Morlex) C_4H_9NO (91% Soln.) Boiler water treatment Corrosion control	Liquids Solutions	1. 30 and 55 gal steel drums 2. 55 gal drums 3. 1 gal tin can, 5 and 55 gal iron drum	Colorless liquids, pH 10, caustic, toxic 1. Boiling Point is 184°C 2. 135°C Flash Point is 90°F (Open cup) 3. 128°C Flash Point is 100°C (open cup) Hygroscopic liquid	Sp. G. (room temperature) 1. -0.97 2. -0.86 3. -1.002	98 to 99 %	Completely Miscible
AMMONIA ANHYDROUS NH_3 (Ammonia) Chlorine - ammonia treatment Anaerobic digestion Nutrient	Colorless, liquified gas	Steel cylinders - 50, 100, 150 lb T/C - 50,000 lb Green Gas Label	Pungent, irritating odor; liquid causes burns Freezing Point is -107.9°F Boiling Point is -28°F. S.Gr. (gas) 0.59 at 70°C and 1 atm. MCA Warning Label Visc. liquid = 0.27 cps at 33°C	Sp. G. of liquid is 0.68 at -28°F	99 to 100% NH_3	% by weight 47.3 at 0°C 40.6 at 10°C 34.6 at 20°C 29.1 at 30°C
AMMONIA, AQUA NH_4OH (Ammonium hydroxide, Ammonia water, Ammonium hydrate) Chlorine - ammonia treatment pH control Nutrient	Technical Certified Pure Solution 16°Be 20°Be 26°BE	Carboys - 5, 10 gal Drums - 375, 750 lb T/C - 8000 gal	Water white solution, strongly alkaline, causes burns: irritating vapor Unstable, store in cool place and tight container MCA Warning Label Vent feeding systems	At 60°F 26°Be S. Gr. 0.8974	16°Be 10.29% NH_3 20°Be 17.76% NH_3 26°Be 29.4% NH_3	Completely miscible
AMMONIUM CHLORIDE NH_4Cl (Sal ammoniac, Ammonium muriate) Source of NH_3 Chloramines Nutrient	Fine crystals	Barrels Multiwall paper sack	White crystals Saline taste Hygroscopic Sublimes at 350°C Endothermic pH 5.5 (1% Soln.)	45 to 55	99.5%	29.4 at 0°C 33.3 at 10°C 37.2 at 20°C 41.4 at 30°C
AMMONIUM SILICO-FLUORIDE $(NH_4)_2SiF_6$ (Ammonium Fluosilicate) Fluoridation for dose 1 ppm 13.3 lb/million gal H_2O	Free flowing fine crystals Commercial grade fine crystals	Pkgs. - 4 to 26.5 lb Kegs - 100 lb Bbl. or Drums - 400 lb.	Odorless, colorless crystals Poison pH 3.5 (1% Soln.)	70 to 80	98% (62.7% Fluoride)	12 at 0°C 19 at 20°C 50 at 100°C
AMMONIUM SULFATE $(NH_4)_2SO_4$ (Sulfate of Ammonia) Chlorine - ammonia treatment Anaerobic digestion Activation of Silica	Small crystals Superdry crystals Damp crystals	Boxes - 25 lb Kegs - 100 lb Bags - 100 lb Bbl. - 300, 400 lb Bulk	White to brown sugar size crystals Tends to cake (Add $CaSO_4$ to prevent caking) Decomposes above 280°C $PH_{0.1M}$ = 5.5	Dry 49 - 64 41 - 54 Damp 60	99% (21% NH_3-N)	70.6 at 0°C 75.4 at 20°C 78.0 at 30°C
BARIUM CARBONATE $BaCO_3$ (Witherite) Boiler water treatment to precipitate out sulfates	Powder	Boxes - 25 lb Kegs - 100 lb Bags - 100 lb	White, heavy cohesive powder Poisonous	52 to 78	98 to 99%	0.0022 at 18°C 0.0065 at 100°C
BENTONITE $H_2(Al_2O_3.Fe_2O_3.3MgO).4SiO_2.nH_2O$ (Colloidal clay, Wilkinite, Volclay) Coagulation aid as a weighting agent	Powder Fine granules	Bags - 50, 100 lb Bulk - C/L	Tannish powder Can form a viscous colloidal solution 3% colloidal solution Viscosity = 4 to 10 cps at 72°F	Powder 45 to 60 Granules 65 to 75	—	Forms a colloidal solution

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
As received	Amines are used in small amounts mixed with other additives, etc.	Solution Diaphragm Metering Pump Series 1200 Series 1700 Series 5700	—	Steel, stainless steel, TFE
Dry gas or as aqueous solution - see ammonia, aqua	—	Gas feeder	Scales	Steel, Ni-Resist, Monel, 316 ss, Neoprene
Full strength	—	Solution Diaphragm Metering Pump Series 1200 & 1700 & 5700	Scales Drum handling equipment or storage tanks Transfer pumps	Iron, steel, rubber, Hypalon, 316 ss, Tyril (room temp. to 28%)
Crystals (No lumps or massed crystals) Add TCP or other anti-caking agent	—	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Special agitators if material becomes massed	316 (plus) rubber, PVC-I, Saran, Vinyl, FRP, Halar, Glass, Carp. 20 SS Hypalon, Ti, Noryl (140°F), TFE, Polypropylene
Free flowing fine crystals or solution	Dry feed 0.5 lb/gal Solution feed up to 1.5 lb/gal	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Dustless bag loading hopper or dust filter. Non-flood feeder, if hopper holds more than 2 bags, with tight seal	316 ss, rubber, Vinyl, PVC-I, Saran, Uscolite, Hypalon, Monel, Halar
Superdried, or $CaSO_4$ coated If high moisture content, use solution feed Solutions "climb"	Dry feed 0.5 lb/gal maximum Solution feed 1.0 lb/gal	Gravimetric L-I-W Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Hopper agitators (spec.)- if dampish	Damp crystals or solution: 316 ss, Hastelloy C, D, rubber, FRP, PVC-I, Vinyl, Hypalon, Tyril, Monel, Uscolite, Ti, Halar, Polypropylene
Powder plus Hi-Sil(4) or a slurry	Slurry feed, 15 to 20% to a 25% maximum slurry.	Gravimetric Belt Volumetric Helix Rotodip (slurry)	Hopper agitators (heavy) and/or vibrator	Slurry Rubber, Glass, Steel, Ceramic, 316 ss, Hypalon Uscolite, PVC-I, Tyril
Granules	0.258 lb/gal (a 3% colloidal solution)	Gravimetric Belt Volumetric Helix Solution (Colloidal) Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Hopper agitators for powder Non-flood rotor mechanism for powder Polypak	Iron, Steel, Hypalon, Tyril, TFE

(4) Hi-Sil is a hydrated silica additive – for improved flow

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft (Bulk Density)	COMMERCIAL STRENGHT	SOLUBILITY IN WATER gm/100 ml
BROMINE Br_2 Swimming pool disinfection Algae control	Liquid	Glass bottles 6.5 lb Earthenware bottles White Label	Dark red liquid Boiling point is 58°C Burns skin Irritating fumes Fire hazard Corrosive Vaporizes at room temperature	Sp. G. 3.119 at 20/15°C 26 lb/gal	99.5%	4.17 at 0°C 3.13 at 30°C
CALCIUM CARBONATE $CaCO_3$ (Calcite, Limestone, Whiting, Chalk) Stabilization of water Neutralizing agent Corrosion prevention Waste treatment	Powder Granules	Bags - 50 lb Drums Bulk - C/L	White amorphous powder	Powder 35 to 50 Granules 100 to 115	96 to 99%	0.0013 at 25°C 0.002 at 100°C (In CO_2 free H_2O)
CALCIUM FLUORIDE CaF_2 (Fluorspar, Fluorite) Fluoridation—For 1 ppm Fluoride, use 18 lb/million gal H_2O	Granular and powder Granular powder Powder Powder (micronized)	Bags - 125 lb Bbl - 500 lb Bulk - C/L	Tan or white granules and powder M.P. 1350°C Toxic Do not breathe dust	Bulk Density varies: 95 to 123 85 to 117 62 to 83 51 to 69	85 to 98%	0.0016 at 18°C 0.0017 at 26°C (Hard to wet the fine powder)
CALCIUM HYDROXIDE $Ca(OH)_2$ (Hydrated Lime, Slaked Lime) Coagulation, softening, pH adjustment Waste neutralization Sludge conditioning Precipitate PO_4	Light powder Powder	Bags - 50 lb Bbl - 100 lb Bulk - C/L (Store in dry place)	White, 200 - 400 mesh powder, free from lumps Caustic, irritant, dusty Sat. Soln. - pH 12.4 Absorbs H_2O and CO_2 from air to revert back to $CaCO_3$ 10% Slurry - 50 to 150 cp S. Gr. = 1.06	20 to 30 and 30 to 50 To calculate hopper capacity, use 25 to 35	$Ca(OH)_2$ 82 to 95% CaO 62 to 72%	0.18 at 0°C 0.16 at 20°C 0.15 at 30°C
CALCIUM HYPOCHLORITE $Ca(OCl)_2 \cdot 4H_2O$ (H.T.H., Perchloron, Pittchlor) Disinfection, slime control, deodorization	Granules Powder Pellets	Bbl - 415 lb Cans - 5, 15, 100, 100 lb Drums - 800 lb (Store dry and cool; avoid contact with organic matter)	White or yellowish white, hygroscopic corrosive Strong chlorine odor (Alkaline pH) Yellow Label - oxidizing agent	Granules 68 to 80 Powder 32 to 50	70% Available Cl_2	21.88 at 0°C 22.7 at 20°C 23.4 at 40°C
CALCIUM OXIDE CaO (Quicklime, Burnt Lime, Chemical Lime, Unslaked Lime) Coagulation, softening, pH adjustment Waste neutralization Sludge conditioning Precipitate PO_4	Pebble Lump Ground Pulverized Pellet Granules Crushed	Moisture-proof bags - 100 lb Wood barrel Bulk - C/L (Store dry - Max. 60 days, keep container closed)	White (light gray, tan) lumps to powder Unstable, caustic, irritant Slakes to hydroxide slurry evolving heat Air slakes to form $CaCO_3$ Sat. Soln. pH is 12.4	55 to 70 To calculate hopper capacity, use 60 Pulverized = 43 to 45	70 to 96% CaO (Below 85% can be poor quality)	Reacts to form $Ca(OH)_2$. See $Ca(OH)_2$ above (See footnote 7 below)
CARBON DIOXIDE CO_2 (Carbonic Acid Gas) (Dry Ice) Recarbonation in water softening Activation of Silica	Generated by burning fuel oil or gas, or from stack gases on site Dry Ice Liquefied gas under pressure	Steel cylinders for compressed gas - 150 lb Dry ice delivered as required and evaporated on site in large steel cylinders Green Label	Colorless, Odorless gas (liquefied under pressure) Snowlike solid below -78°C Sat. Soln. pH 4.0 at 20°C	Sp. G. with respect to air is 1.5 (gas)	99.5% in Cylinders 10 to 14% in combustion gases 100% from dry	0.348 at 0°C 0.145 at 25°C 0.097 at 40°C

(7) Each pound of CaO will slake to form 1.16 to 1.32 lb of $Ca(OH)_2$ (depending on purity) and from 2 to 12% grit.

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTION
Liquid, Anhydrous	—	Liquid Special feeder req'd.	—	Dry (<30 ppm H_2O): Monel, Carp. 20 ss. Lead (Chemical), Nickel Wet and Dry Teflon, Glass, Ceramic Tantalum
According to use, powder may flood	Slurry - 10% (Light to 20% conc. max.) (Heavy to 25% conc. max.)	Gravimetric Belt Volumetric Helix Slurry * Diaphram Metering Pump - Series 1200 & 1700	Hopper agitators Non - flood rotor - for fluid-like or floodable grades	Slurry: Iron, Steel, Rubber
Granules and powder	To a 10% max. slurry Very difficult to disperse the fine powder For use with alum, see Footnote 5 (is soluble in alum solution)	Volumetric Helix Slurry * Diaphram Metering Pump - Series 1200 & 1700	Washdown type wetting tank or Vortex mixer Dust collector Dust machinery Agitators	316 ss, Rubber, PVC-1 Saran, FRP
Finer particle sizes more reactive but more difficult to handle and feed, maintain slurry velocity greater than 3 FPS to avoid settling	Dry feed 0.5 lb/gal - max. Slurry - 0.93 lb/gal; i.e., a 10% slurry (Light to a 20% conc. max.) (Heavy to a 25% conc. max.)	Gravimetric Belt Volumetric Helix Slurry Rotodip * Diaphram Metering Pump - Series 1200 & 1700 (See footnote 6)	Hopper agitators Non-flood rotor under large hoppers Dust collectors	Rubber hose, Iron, Steel Concrete, Hypalon, PVC-1, No lead, Noryl, Derakane
Up to 3% Soln. max. (practical)	0.125 lb/gal makes 1% Soln. of available Cl_2	Liquid Diaphragm Metering Pump - 1200 & 1700 & 5700 Rotodip	Dissolving tanks in pairs with drains to draw off sediment Injection nozzle Foot Valve	Ceramic, Glass, Rubber-lined tanks, PVC-1 Tyril (rm. temp.), Hypalon, Vinyl, Uscolite (rm. temp.) Saran Hastelloy C (good). No tin Ti, (RM), Halar, Polypropylene, TFE
1/4 to 3/4 in. pebble lime, Pellets Ground lime arches and is floodable Pulverized will arch and is floodable Soft burned, porous best for slaking	2.1 lb/gal (range from 1.4 to 3.3 lb/gal (according to slaker etc.) Dilute after slaking to 0.93 lb/gal which is (10%) average slurry (35% is max. Dilution unless residue carry-over is not critical)	Gravimetric Belt Volumetric Helix	Hopper agitator and non-flood rotor for ground and pulverized lime Lime slaker	Rubber, Iron, Steel, Concrete, Hypalon PCV-1
Stack gases and generators are most economical Dry ice or compressed gas involves least equipment	Gas diffused in water under treatment	Generators and gas meter Underwater burners Gas feeders	Compressors and scrubbers for stack gases Diffusers	Dry Gas Iron, Steel Wet: 316 ss, FRP, Monel, Hypalon, PVC-1 Saran, Viton, Vinyl, Tyril (rm. temp.)

(5) Advances in the use of Fluorspar — F. J. Maier, JAWWA, January 1960

(6) When feeding rates exceed 100 lb/hr, economic factors may dictate use of calcium oxide (quicklime)

* Requires Ball Valves

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULAR COMMON NAME	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
CHLORINATED COPPERAS $Fe_2(SO_4)_3$-$FeCl_3$ (Chlorinated Ferrous Sulfate) Coagulation Sludge Conditioning	Produced at site by reaction of chlorine and $FeSO_4$ in solution	See chlorine and ferrous sulfate as reagents needed	Yellow solution Corrosive	—	—	—
CHLORINATED LIME $CaH2CaOCl_2.5H_2O$ -Variable formula- (Chloride of Lime, Bleaching Powder) Disinfection Slime Control	Powder White or Yellow Label	Drums - 100, 300, 800 lb Store cool, dry	White or yellowish-white, unstable, deteriorates: chlorine odor Tends to precipitate incrustants in hard water in feeders Do not mix dry mat with oil, grease, etc.	45 to 50	25 to 37% Available Cl_2	Forms hypochlorous acid (HOCl) with H_2O
CHLORINE Cl_2 (Chlorine Gas, Liquid Chlorine) Disinfection Slime Control Taste and odor control Waste treatment Activation of Silica (See Footnote[8])	Liquified gas under pressure	Steel Cylinders 100, 150 lb Ton containers T/C - 15 ton containers T/C - 16, 30, 55 tons Green Label	Greenish yellow gas liquified under pressure Pungent, noxious, corrosive gas heavier than air: health hazard	Sp. G. with respect to air 2.49	99.8% Cl_2	0.98 at 10°C 0.716 at 20°C 0.57 at 30°C
CHLORINE DIOXIDE ClO_2 Disinfection Taste and odor control (especially phenol) Waste treatment 0.5 to 5 lb $NaClO_2$ per million gal H_2O dosage	Generated as used from Cl_2 and $NaClO_2$ or from $NaClO_2$ and NaOCl plus acid Dissolved as generated	26.3% Avail. Cl_2	Yellow solution when generated in water Yellow-red gas, unstable, irritating, poisonous, explosive: keep cool - keep from light	--	Use 2 lb of $NaClO_2$ to 1 lb of Cl_2, or equal conc. of $NaClO_2$ and NaOCl plus acid (max. 2% each plus Dil. water)	0.29 at 21°C.
COAGULANT AIDS (See Footnote[9]) Dry - potable H_2O High M.W. Synthetic Polymers Separan NP10 potable grade, Magnifloc 990, Purifloc N17 Ave. dosage - 0.1 to 1ppm	Powdered, flakey granules	Multiwall paper bags	White flake powder pH varies	27 to 35	—	Colloidal solution
COAGULANT AIDS (See Footnote[9]) Dry-potable H_2O High M.W. synthetic Polymers Separan NP10 & 20 AP30, Magnifloc 972, Superfloc 16 Average Dosage 0.1 to 5 ppm	Powdered, flakey granules	Multiwall paper bags	White flake powder ph varies	NP10 27 to 35 30 to 37 NP20 32 to 35 AP30 40 to 50 Superfloc 16 26 to 33 Magnifloc 972 28 to 34	—	Colloidal solution

(8) For small dosages of chlorine use calcium hypochlorite or sodium hypochlorite

(9) Information available on many other coagulant aids (or flocculent aids) from, BIF "Polypak" BIF "Polypak" Polymer selection Guide Ref. 39.25-1, Nalco, Calgon, Drew, Betz, North American Mogul, American Cyanamid, Dow, etc.

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTION
1 part Cl_2 to 7.8 parts $FeSO_4$ by weight Mix solution discharge of dry feed machine ($FeSO_4$) with solution discharge from Chlorinator in reaction chamber	Ferrous sulfate 0.5 lb/gal Chlorine 1 lb to 40-50 gal or more	See ferrous sulfate See chlorine	Rubber lined reaction tank	Rubber, Ceramic, Hypalon, PVC-1, Vinyl, TFE, Polypropylene
2% solution of available Cl_2 - Maximum Settle out solids before pumping	About 0.25 lb/gal makes 1% solution of available Cl_2	<u>Liquid</u> Diaphragm metering pump - Series 1200 & 1700 Rotodip	Dissolving tanks or pans with drains to remove sludge	Rubber, Stoneware, Glass, PVC-1 Vinyl, Saran,
Gas - Vaporized from liquid	1 lb to 45-50 gal or more	Gas chlorinator	Vaporizers for high capacities Scales Residue Analyzer	<u>Anhydrous Liquid or Gas</u> Steel, Copper, Black Iron <u>Wet Gas</u> Tygon, Hastelloy C, PVC-1, (good), silver, tantalum, Ti <u>Chlorinated Brine</u> Saran, Stoneware, Carp. 20 ss, Hastelloy C, [illegible] PVC-1, Viton, [illegible] Ti, Derakane
Solution from generator - Mix discharge from Chlorinator and $NaClO_2$ solution or add acid to mixture of $NaClO_2$ and NaOCl. Use equal concentrations - 2% max	Chlorine water must contain 500 ppm or over of Cl_2 and have a pH of 3.5 or less. Water use depends on method of preparation.	<u>Solution</u> Diaphragm metering pump, Series 1200 & 1700 & 5700	Dissolving tanks or crocks	For solutions with 3% ClO_2: Ceramic, Glass, Hypalon, PVC-1, Saran, Vinyl, Teflon, Ti (fair), TFE, Polypropylene
Powdered, flakish granules	Max. conc. - 1% Feed over stream or vigorous vortex - (too fast mixing will break up colloidal growth) 1/2 hour detention	<u>Volumetric</u> Helix <u>Solution (Colloidal)</u> Diaphragm metering pump, Series 1200 & 1700 & 5700	Special dispersing procedure	Steel, Rubber, TFE, Hypalon, Tyril, Derakane, Non-corrosive, but no zinc Same as for H_2O of similar pH or according to its pH, Polypropylene
Powdered, flakish granules	Same as potable water grades	Same as potable water grades	Same as potable water grades	Same as potable water grades

CHEMICAL	CHARACTERISTICS		SHIPPING DATA			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
COPPER SULFATE $CuSO_4 \cdot 5H_2O$ (Blue Vitriol, Blue Stone, Cupric Sulfate) Algae control in reservoirs Root control in sewers 0.1 to 10 ppm dosage	Lump Crystal Powder	Bags - 100 lb Bbl - 450 lb Drums	Clear blue crystals or pale blue powder Slowly efflorescing Poisonous pH above 3	Crystal 75 to 90 Powder 60 to 68	99%	19.3 at 0°C 25.2 at 15°C 26.3 at 20°C 31.1 at 30°C
DIATOMACEOUS EARTH SiO_2 Plus Oxides Infusorial earth, Diatom, Celite, Dicalite, Celatom, Supercel, Speedex, Speed Flow Filter Aid	Natural Calcined Flux Calcined	Bags - 50 lb Bulk	Natural Gray-white bulky powder Calcined Pinkish bulky powder Flux Calcined White bulky powder Do not breathe excessive amount of dust	Natural 5 to 12 8 to 18 Calcined 6 to 13 Flux Calcined 10 to 18 15 to 25	—	Insoluble-forms a slurry
o-DICHLOROBENZENE $C_6H_4Cl_2$ (1, 2 - Dichlorobenzene) (Emulsified grade available) Orano, tech. 80% Cafon 88.5% Chloroben, Benichlor Odor control in sewage and waste treatment	Liquid Emulsified liquid	55 gal steel drum Tank car	Clear liquid Sp. G. 1.3 at 15° C/15° C Keep 32°F Flash point (pure liquid) (Open Cup) is 167°F Ignition Point is 1199°F Freezing Point is -18 to 22° C Toxic	About 10 lb/gal	Regular ortho - 80% para - 20% Pure ortho - 99.5% para - 0.5%	Insoluble (emulsified grade is miscible with H_2O)
DI-SODIUM PHOSPHATE $Na_2HPO_4 \cdot 12H_2O$ (DSP, Secondary Sodium Phosphate, Dibasic Sod. Phos.) Also Na_2HPO_4—Anhydrous Sodium Phosphate Scale and corrosion control Boiler water conditioning, precipitates Ca and Mg	Crystals Anhydrous	Bags - 100 lb Kegs - 125 lb Drums - 25, 100, 125 lb Bbl - 325, 350 lb	White, hygroscopic M.P. is 34.6°C for (94°F) hydrate Store dry in tightly closed container 1% Soln. - pH 8.4 to 9.0 (approx.) Anhydrous is hygroscopic Hydrate is efflorescent	Crystal Hydrate 80 to 90 Anhydrous 53 to 62	Crystal 19 to 19.5% P_2O_5 Anhydrous 48% P_2O_5	Hydrate 4.2 at 0°C 19.0 at 20°C 52.4 at 30°C Anhydrous 1.7 at 0°C 3.6 at 10°C 7.7 at 20°C 20.8 at 30°C
DOLOMITIC HYDRATED LIME $Ca(OH)_2 \cdot Mg(OH)_2$ (dihydrated) $Ca(OH)_2 \cdot MgO$ (monohydrated) Silica removal Acid waste neutralization	Powder (Light or heavy)	Bags - 50 lb Bbl Bulk - C/L	White powder, free from lumps Caustic, dusty Store dry Sat. Soln. - pH 12.4	30 to 50 To calculate hopper capacity use 40	Ave $Ca(OH)_2$ 60% MgO 33% Variable Comp	See $Ca(OH)_2$
DOLOMITIC LIME $CaO \cdot MgO$ (MgO content varies) Dolomitic Quicklime Silica removal Acid neutralization, including waste treatment	Pebble Crushed Lump Ground Pulverized	Bags - 50, 60 lb Bbl Bulk - C/L	White (light gray, tan) lumps to powder Unstable, caustic irritant Slakes to hydroxide slurry evolving heat Sat. Soln. - pH 12.4	Pebble 60 to 65 Ground 50 to 75 Lump 50 to 65 Powder 37 to 65 Average 60	CaO 55 to 57.5% MgO 37.6 to 40.5%	Slakes to form hydrated lime slurry plus MgO. (MgO slakes at 212°F) See $Ca(OH)_2$ solubility
DOLOMITIC LIMESTONE $CaCO_3 \cdot MgCO_3$ (Dolomite) Neutralization	Lump or crushed Granule Ground Powder	Bags - 50 lb Drums Bulk- C/L	White, gray, tan Sat. Soln. - pH 9 to 9.5	Granule 93 to 124 or 85 to 105 Powder 63 to 75	Varies	Approx. same as $CaCO_3$

FEEDING RECOMMENDATIONS				
BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Granules or Ground (sugar) or power for float application	0.25 lb/gal Retention time, 3/8" crystals - 10.0 minutes Fine crystals - 5.0 minutes Powder - 2.3 minutes Temp. 50°F and efficient mixing	Volumetric Helix Solution Diaphragm metering pump - series 1200 & 1700 & 5700 Rotodip	Dissolving tanks For direct application from boxes: Dissolving tank, pump, spray or crystal spreader, or bags	Rubber, ceramic, 316 ss PVC-1, Vinyl, Hypalon FRP, Viton, Uscolite, Saran, Epoxy, Tyril, Polypropylene, TFE
Flash calcined, with higher bulk density	1 lb/gal, depending on bulkiness. a 15 - 20% slurry is the maximum possible	Volumetric Helix Gravimetric Belt Slurry *Diaphragm pump - series 1200 & 1700	Slurry tank Positive agitation	Iron, Steel, Rubber, FRP, Tyril, Hypalon
Liquid	——	Liquid Diaphragm metering pump - series 1200 & 1700 & 5700		Steel, 440 ss, 316 ss, Viton (slight swelling with emulsified grade. Could be less resistance with pure liquid). TFE (RM) (very slight swelling also)
Solution (crystal grade) 0.3 lb/gal Anhydrous grade 0.12 lb/gal	Good, efficient mixing needed for anhydrous (tends to ball)	Solution Diaphragm metering pump - series 1200 & 1700 & 5700	Solution tank	Iron, Steel, Rubber, 304 ss, 316 ss, PVC-1 Uscolite, Ni-Resist, Hypalon, Tyril, TFE, Polypropylene
Finer particle sizes more efficient but more difficult to handle and feed	Dry feed (continuous) 0.5 lb/gal max. Slurry (stock) 0.93 lb/gal - 10% Slurry (stock) Light to a 30% max. Heavy to a 25% max.	Gravimetric Belt Volumetric Helix Slurry Rotodip *Diaphragm metering pump - series 1200 & 1700	Hopper agitators Non-flood rotor under large hopper Dust collectors	Iron, Steel, Rubber hose, Concrete, Hypalon, PVC-1, No lead. Requires carbide valves.
Pebble (small) or crushed to pass 3/4 in. mesh, fines included	2lb/gal (range 1.6 to 4.2 lb/gal Dilute after slaking to 0.93 lb/gal (10% slurry) max. Detention, temp., amount of water are critical for efficient slaking	Gravimetric Belt Volumetric Helix	Hopper agitator and non-flood rotor for ground and pulverized Lime slaker	See dolomitic-hydrated lime
Granule	——	Gravimetric Belt Volumetric Helix	Agitators (for finer grades) Dust collector (powder) Possibly a rotor	For slurry (powder): Steel, Iron, Rubber

*Requires Ball Valve

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
FERRIC CHLORIDE $FeCl_3$ - Anhydrous $FeCl_3 \cdot 6H_2O$ = Crystal $FeCl_3$ Solution (Ferrichlor, Chloride of Iron) Coagulation pH 4 to 11 Dosage - 0.3 to 3 gpg (Sludge cond. 1.5 to 4.5% $FeCl_3$ by weight of solids Precipitate PO_4	Solution Lumps - sticks (crystals) Granules	Solution Carboys - 5, 13 gal Truck, T/C Crystal Keg - 100, 400, 450 lb Drums - 150, 350, 650 lbs	Solution - Dark brown syrup. Crystals - Yellow brown lumps Anhydrous - Green black. Very hygroscopic. *staining corrosive in liquid form. 1% Soln. pH 2.0 40% Soln. = 7 cp visc. & Sp. G. = 1.42	Solution 11.2 to 12.4 lb Crystal 60 to 64 Anhy. 45 to 60	Solution 35 to 45% $FeCl_3$ Crystal 60% $FeCl_3$ Anhy 96 to 97% $FeCl_3$	Solution Completely Miscible Crystals 64.42 81.9 at 10°C 91.1 at 20°C Anhy. 74.4 at 0°C
FERRIC SULFATE $Fe_2(SO_4)_3 \cdot 3H_2O$ (Ferrifloc) $Fe_2(SO_4)_3 \cdot 2H_2O$ (Ferriclear) (Iron Sulfate) Coag. pH 4-6 & 8.8 - 9.2 Dosage - 0.3 to 3 gpg Precipitate PO_4	Granules	Bags - 100 lb Drums - 400 425 lb Bulk - C/L	$2H_2O$ Red brown $3H_2O$ Red Gray Cakes at high R. H. Corrosive in Soln: Store dry in tight containers Stains	70 - 72	$3H_2O$ 68% $Fe_2(SO_4)_3$ 18.5% Fe $2H_2O$ 76% $Fe_2(SO_4)_3$ 21% Fe	Very Soluble
FERROUS SULFATE $FeSO_4 \cdot 7H_2O$ (Copperas, Iron Sulfate, Sugar Sulfate, Green Vitriol) Coagulation at pH 8.8 to 9.2 Chrome reduction in waste treatment. Sewage odor control. Precipitate PO_4	Granules Crystals Powder Lumps	Bags - 100 lb Bbl - 400 lb Bulk	Fine greenish crystals M. P. is 64°C. Oxidizes in moist air. Efflorescent in dry air. Masses in storage at higher temp. Soln. is acid.	63 to 66	55% $FeSO_4$ 20% Fe	32.8 at 0°C 37.5 at 10°C 48.5 at 20°C 60.2 at 30°C
FLUOSILICIC ACID H_2SiF_6 (Hydrofluosilicic Acid) Fluoridation - for 1 ppm use 35.5 lb. per 1.0 million gal H_2O (of 30% H_2SiF_6) or 3.3 gal./mil. - gal.	Liquid (Tech & C.P.)	Kegs - 5 gal. pitch-lined Drums 50 gal. rubber-lined Bbl-420 lb T/C rubber-lined White Label	Clear, colorless, slightly fuming, corrosive, toxic liquid. 1% Soln. - pH 1.2 Attacks glass Vent storage tanks	30% is 10.4 to 10.5 lb/gal and Sp. G. = 1.27	15 to 30% (12 to 24% F)	Miscible
HYDRAZINE H_2NNH_2 + Water. (Diamine Hydrazine Base) Corrosion inhibitor Scavenger for O_2	Solutions of Hydrazine Hydrazine 35% Tech. 44.8% Purified 54.4% Purified 64% (All hydrazine hydrates)	Glass carboys Steel drums	Colorless fuming hygroscopic liquid M. P. 2°C. Flash Pt. (OC) 126°F Reducing agent. Vapor flammable Toxic. Keep from eyes. Corrosive liquid, White Label. Fire hazard with organic mat. Keep from oxidizers and acids.	Anhy density 1.008 at 20°C. 8.38 lb/gal	Anhy. to 99% pure Also a 35% water solution (Deoxy-sol.)	Miscible with water in all proportions
HYDROCHLORIC ACID HCl (Muriatic Acid) Neutralization of alkaline waste. Cleaner Regenerate deionizer	Concentrated - 37% 18, 20, and 22° 22°Be Tech. U.S.P. XVI N.F. C.P.	Bottles Carboys Drums Tank cars	Clear, colorless or slightly yellow, fuming, pungent liquid. Highly corrosive to metals. Avoid skin contact. Toxic. pH of 1 normal is 0.1 White Label Releases H_2 gas from metals.	Conc. - 37% 9.95 lb/gal 18°Be 9.5 lb/gal 20°Be 9.65 lb/gal 22°Be 9.8 lb/gal	Concentrated 37% - 38% HCl 18°Be 27.9% HCl 20°Be 31.5% HCl 22°Be 35.2% HCl	Miscible with water in all proportions
HYDROFLUORIC ACID HF Fluoridation - not usually recommended for use	Liquid	Drums - steel 20, 30, 100 gal. Bulk - T/C White Label	Clear, colorless, toxic, dangerous liquid. Corrosive to steel below 60% Conc. pH approx. 2.0 Attacks glass. Anhy. B.P. = 68°F.	——	30% 48% 52% 60% Anhy. 99%	Miscible

FEEDING RECOMMENDATIONS				
BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Solution or any dilution up to 45% $FeCl_3$ solution (Anhy. form has a high heat of soln.)	Anhydrous to form 45% - 5.59 lb/gal 40% - 4.75 lb/gal 35% - 3.96 lb/gal 30% - 3.24 lb/gal 20% - 1.98 lb/gal 10% - .91 lb/gal (Multiply $FeCl_3$ by 1.666 to obtain $FeCl_3 \cdot 6H_2O$) at 20°C	Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Storage tanks for Liquid Dissolving tanks for lumps or granules	Rubber, Glass, Ceramics Hypalon, Saran, PVC-1, (3) FRP, Vinyl Hast C (good to fair-below 40%) Uscolite, Epoxy, Tyril (RM) Ti, Noryl, Kynar, Derakane, Halar, Polypropylene, TFE
Granules	2 lb/gal (Range) 1.4 to 2.4 lb/gal for 20 min. detention (Warm water permits shorter detention) Water insolubles can be high	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Dissolver w/motor – driven mixer and water control. Vapor remover solution tank.	316 SS, Rubber, Glass, Ceramics, Hypalon Saran, PVC-1, Vinyl, Carp 20 SS (3) FRP, Epoxy, Tyril, Ti, Halar Derakane, Polyethylene, Polypropylene, TFE
Granules	0.5 lb/gal (Dissolve detention time 5 min. Minimum)	Gravimetric L-I-W Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Dissolvers Scales	Rubber, (3)FRP, PVC-1 Vinyl, Epoxy, Hypalon, Uscolite, Ceramic, Carp. 20 SS Tyril, Polypropylene, TFE
Liquid direct from containers do not dilute (unless investigated - depends on free silica in acid, etc.) (10)	—	Liquid Diaphragm Metering Pump - Series 1200 &1700 & 5700 Rotodip	Scales to check weight fed	Rubber, PVC1 Saran, Vinyl, TFE, Hypalon, Carp. Hypalon, Carp. 20 Hast. C, Viton, Halar, Polypropylene
Dilute Solution	Can dilute to 0.2 lb/gal stock or feed solution	Solution (Special) Diaphragm Metering Pump - Series 1200 & 1700	—	No molybdenum or Cu (will catalyze and decomp.) Glass, 304 SS, Ceramic, 347 SS, Polyethylene, Teflon, (no welding slag fillings either). 316 SS Suitable for low concentrations less than 3%. TFE (RM) Hypalon (to 35%) (to 100°F)
As received (less fuming with diluted acid)	—	Liquid Diaphragm Metering Pump - Series 1200 & 1700 & 5700		Metals not generally recommended. - Hast. C (caution) Rubber, Polyethylene (3) FRP, Vinyl, Hypalon, (to 120°F), Viton, Halar, Tyril (RM), Ti (RM) Noryl (20% Derakane, TFE Polypropylene (RM)
Solution (60%) as received	—	Diaphragm Metering Pump-Series 1200 & 1700 Special precautions for equipment and materials, chemical not generally recommended because of handling problems		Monel (50%, air free), Rubber (to 52%) Derakane (10%), Epoxy, Carp. 20 (50% air) Hast C (10 & 60%) Viton, Tantalum (Steel 60%) Hypalon (to 48%) Saran, Polyethylene

(10) Bellack, E & Maier, F. "Dilution of Fluosilicic Acid," Feb. 1956, JAWWA, PG. 199

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULAR COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
IODINE I_2 Disinfection-Swimming Pool 1 - 3 ppm	Lumps Granules Flakes	Bottles 100 - 200 lb. kegs Drums	Blue, black, lustrous volatile bluish granules. M.P. is 113.5°C Toxic Sp. G. - 4.93 Violet vapor Readily sublimes	164 - 184 (granules)	—	0.03 at 25°C 0.09 at 50°C (soluble in KI soln).
MAGNESIUM HYDROXIDE $Mg(OH)_2$ Defluoridation	Heavy powder Light powder	Wooden barrel or drums Glass bottles Carboys	White powder Sp. G. = 2.36 Absorbs CO_2 in presence of H_2O. Suspension is called milk of magnesia	Heavy Powder 30 - 45 24 - 34 Light Powder 14 - 23	—	0.0009 at 18°C
MAGNESIUM OXIDE MgO (Magnesia, Magox, Calcined Magnesite) Defluoridation Removal Fe (pH 7.1) 2 ppm MgO each ppm Fe plus diatomite Boiler water treatment Water softening	Heavy powder Light powder Granule	Fiber drums Multiwall paper sacks Tonnage lots Bulk	White powder M.P. 2800°C Absorbs CO_2 and moisture. pH is alkaline	Heavy Powder 57 - 91 44 - 70 40 - 58 Light Powder 17 - 29 16 - 25 7.0 - 11.6	Reagent is 98% min.	0.00062 (cold H_2O) Hydrates very slowly to $Mg(OH)_2$
OCTADECYLAMINE $C_{18}H_{37}NH_2$ Filming amine Hagafilm, Nalco 353 & 354 Ocrafilm 80 & 120, Permacel 90 & 180 Calgon FL-20 Boiler Water - corrosion control, condensate and feed-water systems	Solution Liquid Flake	Liquids 5 & 50 gal. drums Dry 55 lb drum	Liquids or solutions (mainly) Sp. G. < 1.0 Visc. 600 to 6000 cp pH 8.0	< 8.34 lb/gal.	—	Most are dispersible in hot H_2O
OZONE O_3 Taste and Odor Control Disinfection, Waste treatment 1 to 5 ppm-odor 0.5 to 1 ppm-disinfection Color Removal CN & Phenol Oxidation	Gas Liquid	Generated at site by action of electric discharge through dry air-0.5 to 1% produced	Colorless-bluish gas or blue liquid Toxic - do not breathe-Explosive fire hazard-keep from oil or readily combustible materials.	Density of Gas is 2.1 Liquid Sp. G. is 1.71 (S. is 1.71 at -183°C	1 - 2%	49.4 cc at 0°C
PHOSPHORIC ACID, ORTHO H_3PO_4 Boiler water softening Reduce alkalinity Cleaning boilers. Nutrient Feeding	50%, 75%, 85% 90% Anhy. Commercial Tech. Food N.F.	Bottles, 1-5 lb. 5, 6-1/2, 13 gal. carboys 55 gal drums & barrels. Tank cars and trucks.	Clear, colorless liquid F.P. (50%) is -40% B.P. (50%) is 108°C pH (0.1N) is 1.5 15 to 30 cp. viscosity accord to % Avoid skin contact MCA warning label Can form H_2 with some metals.	50% 11.2 lb/gal 75% 13.3 lb gal 85% 14.1 lb/gal	50, 75 and 85% conc.	Liquid miscible with water in all proportions
POTASSIUM IODIDE KI With I_2 Disinfection	Crystal Powder	Fiber drums	White crystals M.P. = 680°C Toxic	Crystal 84 - 100 Powder 62 - 85	Reagent is 99.5% min.	127.5 at 0°C 136.0 at 10°C 144.0 at 20°C 152.0 at 30°C

FEEDING RECOMMENDATIONS				
BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
	I_2+KI-1 to 1.33 ratio dilute (batch soln.) Investigate (Dissolve in KI soln. tablets in alcohol, etc.)	Solution Diaphragm Metering Pump - Series 1200 & 1700	Vibrator (if needed)	10% Soln. 316 St. (pass) Carp. 20 SS, Alum. (3)FRP, Glass, Stoneware, Vinyl Lining. Viton.
Same as MgO	Slurry Feed: 5 - 10% conc. (Light to a 20% maximum) (Heavy to a 30% maximum)	Gravimetric Belt Volumetric Helix Slurry *Diaphragm Metering Pump Series 1200 & 1700	Agitators Rotor Dust Collector Vortex Mixer Eductor or Dissolver-mixer.	Steel, Iron, Rubber, Hypalon,
Heavy powder (feeding) (Light powder is more reactive) (Fresh MgO important for Si removal)	Slurry Feed: 10% conc. (Light - 10 to 15% maximum) (Heavy to a 25% maximum)	Gravimetric Belt Volumetric Helix *Diaphragm Metering Pump Series 1200 & 1700	Agitators Rotor Dust Collector. Vortex Mixer Eductor or equivalent or Dissolver-mixer,	Steel, Iron, Rubber Requires Carbide Valves.
Diluted Solutions	—	Solution Diaphragm Metering Pump Series 1200 & 1700 & 5700	—	Steel, St. Steel, TFE
As generated. Approx. 1% Ozone in air	Gas diffused in water under treatment.	Ozonator	Air drying equipment Diffusers	Glass, 316 SS, Ceramics, Aluminum Teflon
50 to 75% conc. (85% is syrupy) (100% is crystalline)	—	Liquid Diaphragm Metering Pump Series 1200 & 1700 & 5700		316 St. (to [illegible] F), Rubber, (3)FRP, Vinyl, PVC-1, Hypalon, Viton, Carp 20 SS Hast. C, Noryl (to 75%), TFE, Polypropylene
Crystal	See I_2	—	—	Vinyl Lining, (First 316 St., Hast C., Saran, Hypalon, PVC-1, (3)FRP, Carp. 20

*Requires Ball Valves

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
POTASSIUM PERMANGANATE $KMnO_4$ Cairox Taste odor control 0.5 to 4.0 ppm Removes Fe & Mn at 1-1 and up to 2-1 ratio, Respectively	Crystal	U.S.P. 25, 110, 125 lb steel keg Tech. 25, 110, 600 lb steel drum	Purple crystals Sp. G. - 2.7 Decomposes 240°C Can cake up at high relative humidity Strong oxidant. Toxic Keep from organics. Yellow Label.	86 - 102	Tech. is 97% min. Reagent is 99% min.	2.8 at 0°C 3.3 at 10°C 5.0 at 20°C 7.5 at 30°C
SODIUM ALUMINATE $Na_2Al_2O_4$, Anhy. (soda alum) Ratio Na_2O/Al_2O_3 1/1 or 1.15/1 (high purity) Also $Na_2Al_2O_4 \cdot 3H_2O$ hydrated form Coagulation Boiler H_2O treatment.	Ground (pulverized) Crystals Liquid, 27°Be Hydrated Anhydrous	Ground Bags - 50, 100 lb. Drums Liquid Drums	High Purity White Standard Gray. Hygroscopic Aq. Soln. is alkaline Exothermic heat of solution pH = 11 32% is 110°F at 70°F and Sp.G =1.47	High Purity 50 Standard 60	High Purity Al_2O_3 45% $Na_2Al_2O_4$ 72% Standard Al_2O_3 55% $Na_2Al_2O_4$ 88-90%	Hydrated 80 at 75°F Standard 6 - 8% Insolubles Anhydrous 3 lb/gal at 60°F 3 lb/gal at 60°F
SODIUM BICARBONATE $NaHCO_3$ (Baking Soda) Activation of Silica pH adjustment Treat acidic wastes Digester pH	U.S.P. C.P. Commercial Pure Powder Granules	Bags - 100 lbs Bbl - 112, 400 lb Drums 25 lb Kegs	White powder Slightly alkaline 1% soln. pH 8.2 Unstable in soln. (Decomposes into CO_2 and Na_2CO_3) Decomposes 100°F.	59 - 62	99% $NaHCO_3$	6.9 at 0°C 8.2 at 10°C 9.6 at 20°C 10.0 at 30°C (5.5% is practical)
SODIUM BISULFITE, ANHYDROUS $Na_2S_2O_5$ ($NaHSO_3$) Sodium Pyrosulfite, Sodium metabisulfite) Dechlorination-About 1.4 ppm for each ppm Cl_2 Reducing agent in waste treatment (as Cr)	Crystals Crystals plus powder Solution (3.25 to 44.9%)	Bags - 100 lb. Drums - 100 400 lb.	White to slight yellow Sulfurous odor Slightly hygroscopic Store dry in tight container Forms $NaHSO_3$ in soln. 1% soln. pH 4.6 Vent Soln. tanks	74 - 85 and 55 - 70	97.5% to 99% $Na_2S_2O_5$ SO_2 65.3%	54 at 20°C 81 at 100°C
SODIUM CARBONATE Na_2CO_3 (Soda Ash -58% Na_2O) Water Softening, pH Adjustment Neutralization	Dense granules Med Gran. & Pwd. Light Powder	Bags - 100 lb Bbl - 100 lb Drums - 25, 100 lb Bulk - C/L	White, alkaline Hygroscopic-can cake up. 1% soln. pH 11.2	Dense 65 Medium 40 Light 30	99.2% Na_2CO_3 58% Na_2O	7.0 at 0°C 12.5 at 10°C 21.5 at 20°C 38.8 at 30°C
SODIUM CHLORIDE NaCl (Salt, Common Salt) Regeneration of ion exchange materials (zeolites) in water softening	Rock Powder Crystal Granules	Bags - 100 lb. Bbl Drums 25 lb. Bulk - C/L	Colorless crystals or white powder, slightly hygroscopic, corrosive when moist. Impure form very hygroscopic - will cake up.	Rock 50 to 60 Crystal 58 to 70 Powder 48 - 60	98%	35.7 at 0°C 35.8 at 10°C 36.0 at 20°C 36.3 at 30°C
SODIUM CHLORITE $NaClO_2$ (Technical Sodium Chlorite) Disinfection, Taste and Odor Control. Ind. Waste Treatment (with Cl_2 produces ClO_2)	Powder Flakes Crystals (Tech. and Analytical) Solution (about 40%) Crystallizes about 95°F.	Drums 100 lb. (Do not let $NaClO_2$ dry out on combustible materials)	Tan or white crystals or powder. Hygroscopic, poisonous Powerful oxidizing agent. Explosive on contact with organic matter. Store in metal containers only. Oxidizer Liq. - White Label Solid - Yellow Label	65 - 75	Technical 81% 78% (min.) 124% Available Cl_2 Analytical 98.5% 153% Available Cl_2	34 at 5°C 39 at 17°C 46 at 30°C 55 at 60°C

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Crystals plus anticaking additive	1.0% conc. (2.0% max.) 5-min. Retention with Mech. Mixer & Water > 55°F, if water < 55°F, 10 min.	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Dissolving Tank, Mixer, mechanical	Steel, Iron (neutral & alkaline) 316 St., PVC-1, (3)FRP, Hypalon Lucite, Rubber (alkaline) Noryl, Kynar, Halar Derakane, TFE, Polypropylene
Granular or soln. as received Std. grade produces sludge on dissolving	Dry 0.5 lb/gal Soln. dilute as desired	Gravimetric Belt Volumetric Helix Solution Rotodip Diaphragm - Metering Pump - Series 1200 & 1700 & 5700	Hopper agitators for dry form	Iron, Steel, Rubber, 316 St. s., Concrete, Hypalon, TFE, Polypropylene
Granules or Powder plus TCP (0.4%)	0.3 lb/gal.	Gravimetric Belt Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Hopper agitators and Non-flood rotor for powder, if large storage hopper.	Iron & Steel (dilute soln.), (caution), Rubber, St. Steel, Saran, Hypalon, Tyril, TFE, Polypropylene
Crystals (Do not let set.) Storage difficult	0.5 lb/gal.	Volumetric Helix Solution Rotodip Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Hopper Agitators for powdered grades Vent dissolver to outside	Glass Carp. 20 SS, PVC-1 Uscolite, 316 st., (3)FRP, Tyril, Hypalon, TFE, Polypropylene
Dense	Dry Feed 0.25 lb/gal for 10 min. detention time & 0.5 lb/gal for 20 min. Solution Feed 1.0 lb/gal Warm H_2O and/or efficient mixing can reduce detention time if mat. has not set around too long and formed lumps - to 5 min.	Gravimetric Belt Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Rotor for light forms to prevent flooding. Large dissolvers Bin agitators for medium or light grades & very light grades.	Iron, Steel, Rubber, Hypalon, Tyril, Ti Halar, TFE, Polypropylene
—	Saturated brine usually made up for regeneration of ion-exchange material.	Gravimetric Volumetric Belt Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Series Solution Tanks	(3)FRP, Glass, PVC-1, Saran, Vinyl, Hast. C, Hypalon, Halar 316 St. (Fair), Duraflow Carp. 20 SS, Tyril, Ti, Noryl, Kynar, TFE, Polypropylene
Solution as rec'd.	Batch Solutions 0.12 to 2 lb/gal.	Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Chlorine feeder and chlorine dioxide generator	Glass, Saran, PVC-1, Vinyl Tygon (3) FRP, Has. C (fair), Hypalon, Tyril, Ti, Noryl, Derakane, Halar (100°F), TFE, Polypropylene

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
SODIUM CHROMATE Na_2CrO_4 $Na_2CrO_4.4H_2O$ Sodium Dichromate $Na_2Cr_2O_7$ $Na_2Cr_2O_7.2H_2O$ (Sodium Bichromate) Corrosion control	Granules Powder Solution Tech. Reagent	Chromate 100 lb bags 100, 200, 400, lb steel drums Dichromate 50, 100 lb bags 100, 400 lb drums	Chromate, Anhy. Yellow crystals Deliquescent M.P. is 792°C pH 8.4 to 9.2 Dichromate Anhy. Orange Xls M.P. 357°C pH 3.1 to 4.4 Dichromate $2H_2O$ Orange Xls Deliquescent pH 3.1 to 4.4 All are toxic	Chromate Anhy. 93-113 Hydrate 53-63 Dichromate Anhy. 8-100 Hydrate 63-99	Chromate Anhy. 39-42% soln. 98.8% tech. Dichromate $(2H_2O)$ 69% soln. 99.8% tech. 99.9% reagent	Na_2CrO_4 57 at 60°F $Na_2CrO_4.4H_2O$ 88 at 60°F $Na_2Cr_2O_7$ 111 at 60°F $Na_2Cr_2O_7.2H_2O$ 126 at 60°F
SODIUM FLUORIDE NaF (Fluoride) Flouridation - For 1 ppm Feed 18.3 lb. per 1 Million gallons H_2O (98% NaF)	Granular Crystal Powder	Bags - 100 lb Drums - 25, 125, 375 lb (Bags shipped only by car load or truck, LCL by drum only) White Label	White or tinted Nile Blue for identification Store dry and separately from other chemicals. Poisonous 1% Soln. is pH 6.5 4% Soln. is pH 7.6	Powder 65 to 100 Granular Crystals 90 to 106	NaF 95 to 98% F 43 to 44%	4.0 at 0°C 4.03 at 15.5°C 4.05 at 20°C 4.1 at 25°C 5.0 at 100°C
SODIUM HEXA-META PHOSPHATE $(NaPO_3)_6$ (Calgon, Glassy Phosphate Sodium Polyphos, Micromet Metafos, Nalco 519, Quadrafos) Corrosion and Red Water control. Stabilization, Water Well Yield Adjustment. Sequester Ions	"Glass" (Powder or flake more expensive) (Unadjusted form less expensive)	Bag - 100 lb Drums - 100, 300, 320 lb	Powdered & Granular grades. Also broken glass grade. slightly hygroscopic 0.25% soln. -pH 6.8 - 7.0 (unadjusted) Adjusted with soda ash to → pH 8	Glass 64 to 100 Powder & Granular 44-60	67% P_2O_5 Min.	Infinitely Soluble.
SODIUM HYDROXIDE NaOH (Caustic Soda, Soda Lye) pH Adjustment, Neutralization	Flakes Lumps Powder Solution	Drums - 25, 50, 350, 400, 700 lb Bulk-Solution in T/C Liquid White label	White flakes, Granules or pellets, Deliquescent, caustic poison. Dangerous to handle. 1% Soln. pH 12.9 50% Soln. will crystallize at 54°F and Viscosity = 80 cp (Rm) & Sp. G. = 1.53	Pellets 60 to 70 Flakes 46 to 62	Solid 98.9% NaOH 74.76% Na_2O Solution 12 to 50% NaOH	42 at 0°C 51.5 at 10°C 109 at 20°C 119 at 30°C
SODIUM HYPOCHLORITE NaOCl (Javelle Water, Bleach Liquor, Chlorine Bleach) Disinfection, Slime control, Bleaching	Solution White or Yellow Label	Carboys - 5, 13, gal. Drums - 30 gal. Bulk - 1300, 1800, 2000 gal % T	Yellow liquid strongly alkaline, store in cool place, protect from light and vent containers at intervals. Can be stored about 60 days under proper conditions. Sp.G. = 1.21 (15%)	15% 10.2 lb/gal 12.5% 10 lb/gal	15% NaOCl = 1.25 lb. Cl_2/gal 12.5% NaOCl = 1.04 lb. Cl_2/gallon	Completely Miscible
SODIUM NITRATE $NaNO_3$ (Soda, Niter, Chile Saltpeter) Waste Treatment Source of Oxygen Boiler H_2O Treatment Odor Control	Crude Purified C.P. U.S.P.	Multiwall Paper Sacks Bags up to 100 lb	Colorless crystals or white pellets Sp.G. - 2.26 M.P. - 308°C Deliquescent Fire Hazard - oxidizing agent Yellow Label	Crystals 82 - 97 Pellets 75 - 83	Crude 92% C.P. 99%+	73 at 0°C 80 at 20°C 96 at 30°C (when dissolved lowers temp. of water.)
SODIUM PENTACHLOROPHENATE C_6Cl_5ONa $C_6Cl_5ONa.H_2O$ Pentachlorophenate "Santobrite", "Dowicide G" (monohydrate) Slime & Algae Control Biocide	Anhydrous Flakes Pellets Briquettes Monohydrate beads Flakes Ground	Flake 50 lb Burlap 60 lb Fiber drums Pellets 300 lb Fiber drum Ground 100 lb drum	Tan Flakes, etc. pH 9.0 Stable when stored dry. Converted to insoluble phenol at pH 6.8 TOXIC - Keep from skin, eyes. Do not breathe dust.	Powder 20 - 25 Pellets 45 - 55 Briquettes 50 - 58 Monohydrate 20- 25	88% Min.	Anhy. 15.0 at 4°C 20.0 at 9.5°C 40.0 at 18°C Monohydrate 33.0 at 25°C

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Solution	—	Solution Diaphragm metering Pump - Series 1200 & 1700 & 5700	—	316 S. St., Glass, Rubber, PVC-1, Uscolite, Hypalon, Tyril, Steel (Caution), TFE Polypropylene
Granular (Powder arches, floods, is dusty)	Dry Feed 1 lb to 12 gal (1%) 5-10 min. detention Solution Feed 1 lb to 3 gal or more	Volumetric Helix Solution Diaphragm metering Pump Series 1200 & 1700 & 5700	Vibration or Agitator for powder grades. Dust Control Equipment Large Dissolvers. Softeners for water used in making solutions for pumping, if hardness > 50 ppm (Hard H_2O will form CaF_2)	Rubber, 316 st., PVC-1, Saran, (1)FRP, Hypalon, Tyril, TFE, Polypropylene
Solution	1 lb/gal in solution feeder (Above 2 lb/gal becomes syrupy)	Solution Diaphragm metering Pump - Series 1200 & 1700 & 5700	Tray or basket below surface of dissolving tank. Agitation after solution.	316 st., Rubber Ceramics, Vinyl, PVC-1, (3)FRP, Saran, Hypalon, Tyril, TFE, Polypropylene
Solution feed	NaOH has a high heat of solution	Solution *Diaphragm metering Pump - Series 1200 & 1700 & 5700 Rotodip		Cast Iron, Steel, For no contam. use, Rubber, PVC-1, 316 St., Hypalon, TFE, Polypropylene (RM)
Solution up to 16% Available Cl_2Conc.	1.0 gal of 12.5% (Avail. Cl_2) Soln. to 12.5 gal. of water gives a 1% Available Cl_2 soln.	Solution Diaphragm metering Pump - Series 1200 & 1700 & 5700 Rotodip	Solution tanks Foot Valves Water Meters Injection Nozzles	Rubber, Glass, Tyril, Saran, PVC-1, Vinyl, PVC-1, Vinyl, Ti, Hast. C., Hypalon-Durakane (170°F) Halar (100°F), TFE, Polypropylene (RM)
Crystals or pellets (coated) (keep dry)	10% conc. 5 min. detention	Gravimetric Belt Volumetric Helix Solution Diaphragm metering Pump - Series 1200 & 1700 & 5700	Dissolving Tank, Mixer	Steel (dry), St. steel Hypalon, Tyril, TFE, Polypropylene
Solution	Anhydrous Do not exceed a 20% Solution. Generally used in a 1/4 to 2% soln.	Solution Diaphragm metering Pump - Series 1200 & 1700 & 5700	—	Iron, Steel, Brass, Hypalon, Tyril, TFE, Polypropylene

*Requires Ball Valves

CHEMICAL	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON USE NAME	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGHT	SOLUBILITY IN WATER gm/100 ml
SODIUM PHOSPHATE, MONO Anhydrous - NaH_2PO_4 Boiler H_2O Treatment pH Control	Powder	100 lb. paper bag 125 lb. drum 350 lb. barrel	White powder Deliquescent pH 4.1	51 - 71	98% (58% P_2O_5)	57.9 at 0°C 69.9 at 10°C 85.2 at 20°C 107.0 at 30°C
SODIUM PHOSPHATE, MONO Hydrated - $NaH_2PO_4H_2O$ Boiler, H_2O Treatment pH control	Crystal	Same as mono. phosphate, anhy.	White crystals Loses H_2O at 100°C Cakes up at room R.H. pH 4.5	49 - 73	97% (50% P_2O_5)	64.6 at 0°C 80.4 at 10°C 98.0 at 20°C 123.0 at 30°C
SODIUM SILICATE Na_2SiO_3 41°Be: Na_2O:3.22SiO_2 (Water Glass) Coagulation aid as activated silica. Corrosion control	Liquid (Various ratios of Na_2O, SiO_2 are available)	Drums - 1, 5, 55 gal Bulk - T/T, T/C	Opaque, viscous. Conc. solution may freeze about 25°F. 1% Soln. pH 12.7 Viscosity = 180 cp Sp. G=1.39 (68°F)	41°Be 11.6 lb/gal 42.2°Be 11.73 lb/gal	41°Be 8.9% Na_2O 28.7% SiO_2 42.2°Be 9.16% Na_2O 29.5% SiO_2	Completely miscible
SODIUM SILICOFLUORIDE Na_2SiF_6 (Sodium Fluosilicate) Fluoridation - for 1 ppm F use 13.0 lbs per 1 million gallons H_2O	Granular Powdered Granular Granular, Powd.	Bags - 100 lb Drums - 25, 125, 375 lb. Bags shipped only in carloads or trucks LCL in drums, only White Label	White or tinted Nile blue for identification Poison; store separately from other chemicals. Sat. Soln. pH 3.0	Granular 85 - 105 Pwd. Gran. 60 - 98 Gran. & Pwd. 70 - 100	98.5 (60%F)	0.44 at 0°C 0.762 at 25°C 0.98 at 37.8°C 1.52 at 65.6°C 2.45 at 100°
SODIUM SULFITE Na_2SO_3 (Sulfite) Dechlorination in disinfection, reducing agent in waste treatment. Oxygen removal in boiler water treatment. About 1.8 ppm for each ppm Cl_2 in dechlorination.	Granules (crystals) Powder	Bags Bbl Drums Kegs	White crystals or powder Saline Sulfurous taste 1% Soln. pH is 9.8 Soln. gives off SO_2	Powder 54 - 85 Granules 84 - 107	93 to 99%	14 at 0°C 20 at 10°C 27 at 20°C 36 at 30°C
SODIUM THIOSULFATE $Na_2S_2O_3 \cdot 5H_2O$ (Hypo, Sodium Hyposulfite) Dechlorination in disinfection, reducing agent for Cl_2. 1 to 1.5 ppm for each ppm Cl_2 (dechlorination)	Standard Crystals Rice Granules	Bags Bbl Drums Kegs	White translucent crystals; cooling bitter taste, 1% Soln. pH 6.5-8.0 Efflorescent - dry air & slightly deliq. in damp air. Loses all H_2O at 200°C M.P. about 120°F.	53 - 60	98 to 99%	74.7 at 0°C. 301 at 60°C
SULFAMIC ACID HSO_3NH_2 pH control Cleaning scale	Granule Crystallin (Damp)	100 lb Fiber drum (Polyethylene liner)	White granules, M.P. at 176°C pH is 5.2	71 - 81	Crystallin 99.% plus Granular is less pure.	14.0 at 0°C 18.0 at 10°C 21.0 at 20°C 25.0 at 30°C
SULFUR DIOXIDE SO_2 Dechlorination in disinfection, Filter bed cleaning. About 1 ppm SO_2 for each ppm Cl_2, in dechlorination. Waste treatment - Cr +6 reduction.	Liquified Gas under pressure	Steel cylinders 100, 150, 200 lb. Green Label	Heavier than air Colorless gas, suffocating odor, corrosive, poison Acid in solution dissolves to form H_2SO_3 Density of liquid is 89.60 lb/cu ft. at 32°F.	—	100% SO_2	760mm 22.8 at 0°C 16.2 at 10°C 11.3 at 20°C 7.8 at 30°C

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Powder, dry	Good, efficient mixing required. ("Feed to Ball")	Gravimetric Volumetric Helix Belt Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Agitators Rotor-possibly Dust Collector	316 SS, Carp. 20 SS Hypalon Rubber, FRP PVC-1, Vinyl, Tyril, TFE, Polypropylene
Crystals, dry	—	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Vibrator-possible	Same as for the anhydrous Form.
Solution Feed as received to dilution tanks See Activated Silica	See Activated Silica	Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Scales to check loss in weight of drums as silicate is fed	Cast Iron, Steel, Rubber, Hypalon, TFE, Polypropylene
Optimum Mesh Mesh %Og 60 - 1 (max.) 100 - 9 (Max.) 200 - 35 325 - 15 - 30 pan - 15 - 30	1 lb to 120 gal of water (0.1%) 10 min. detention with < 60°F H_2O mechanical mixer (5 min. with warm H_2O) with right cond. 1 lb to 60 gal H_2O (0.2%)	Volumetric Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Hopper Agitators for fine powder Vibrator for heavier, powdered grade. Large dissolving chamber, or special dissolving arrangement	316 Ss, Rubber, PVC1 Vinyl, Saran, Hypalon, Viton Tyril, Halar, Derakane TFE, Polypropylene
Solution 2 to 12.5%	3% Solution 5 - 10 min. retention	Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Tanks Vent. Soln. Tanks	Cast Iron, steel For no contam: 316 st., PVC-1, Saran, FRP, Vinyl, Hypalon, Tyril TFE, Polypropylene
Solution	—	Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	—	Cast Iron, Steel No Contam: 316 st., PVC-1 Saran, Tyril, FRP, Vinyl, Hypalon, TFE, Polypropylene
Granule	1 - 3% made up, can be made stronger	Volumetric - Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700	Damp-proof, agitation or possibly vibrate. No aid needed for dry granules.	316 st., wood, Polyethylene, glass, Teflon, Hypalon, Carp.20 S.S. (Pumps), Tyril, TFE, Polypropylene
Gas	1 lb to 1 gal. —	Gas Rotameter SO_2 Feeder		Wet gas: Glass, Carp. 20 S.S., PVC-1 Ceramics, 316 (d), Viton, Hypalon

CHEMICALS	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME USE	GRADES OR AVAILABLE FORMS	CONTAINERS AND REQUIREMENTS	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft. (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
SULFURIC ACID H_2SO_4 (Oil of Vitriol, Vitriol) pH Adjustment, Activation of Silica Neutralization of Alkaline wastes.	Liquid 66°Be 60°Be 50°Be	Bottles Carboys 5, 13 gal Drums 55, 100 gal Bulk T/T, T/C White Label	Syrupy liquid, very corrosive, hygroscopic. Store dry and cool in tight container. pH = 1.2 for 0.1N Soln. + 0.3 for 1N Soln. Visc. = 22 cp Sp. G. = 1.8	66°Be 15.1 lb/gal 60°Be 14.2 lb/gal 66°Be	66°Be 93.2% H_2SO_4 60°Be 77.7% H_2SO_4 50°Be 62.2% H_2SO_4	Completely miscible
TETRA-SODIUM PYROPHOSPHATE $Na_4P_2O_7 \cdot 10H_2O$ (TSPP, Alkaline Sodium Pyrophosphate) $Na_4P_2O_7$ - Anhydrous Boiler Water Treatment	Crystal Powder C.P. Pure	Bags - 100, 200 lb Bbl - 350 lb Drums - 25, 100, 300 & 350 lb Kegs - 125 lb	Crystal hydrate is efflorescent in dry air. Anhy. is slightly hygroscopic. Crystal loses water at 100° C. 1% Soln. pH 10.8 (Approx.)	Crystal 50-70 Powder 46-66	Crystal Hydrate 34% P_2O_5 Anhydrous 53-57% P_2O_5	Hydrate 5.4 at 0°C 7.0 at 20.7°C Anhy. 2.6 at 25°C
TRICALCIUM PHOSPHATE $Ca_3(PO_4)_2$ $[Ca_5OH(PO_4)_3]$ Calcium Phosphate, tribasic Defluoridation	Granular Powder Tech. C.P. N.F. Pure	60 lb bag 150 lb drum	White powder Sp. G. = 3.18 M.P. = 1670°C pH 6-7	21 - 33	96%	0.0025 (Cold H_2O) (decomposes in hot H_2O)
TRI-SODIUM PHOSPHATE Anhy. Na_3PO_4 (TSP) $Na_3PO_4 \cdot 12H_2O$ $Na_3PO_4 \cdot H_2O$ Boiler water treatment Cleaner	Crystals, coarse medium, std. Powder C.P. Commercial Highest Purity	Bags - 100, 200 lb Bbl - 325, 400 lb Kegs - 125 lb	Colorless crystals. Crystal, efflorescent. May cake under high R.H. Monohydrate, Anhy. are hygroscopic. Loses 11 H_2O at 100°C 1% Soln. - pH 11.9 Crystal M.P. = 73°C	Crystal 55 to 60 Mono-Hydrate 65 Anhy. 50-70	Crystal 19% P_2O_5 Anhy. 42% P_2O_5 Hydrate, Mono 38.5% P_2O_5	Hydrated $(12H_2O)$ 3.5 at 0°C 9.5 at 10°C 15.5 at 20°C 46.4 at 30°C Anhy. 1.5 at 0°C 4.1 at 10°C 11.0 at 20°C 20.0 at 30°C

FEEDING RECOMMENDATIONS

BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPE OF FEEDERS	ACESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
Solution at desired dilution. H_2SO_4 has a high heat of solution	Dilute to any desired concentration, but always add acid to water. NEVER water to acid	Liquid Diaphragm Metering Pump - Series 1200 & 1700 Rotodip & 5700	Dilution tanks	Conc. >85% Steel, Iron, PVC-I (Good), TFE (KM) Viton 40 to 85%: Carp. 20, PVC-I Viton, TFE 2 to 40%: Carp. 20, PVC-I glass, PVC1, Viton, Noryl, Derakane (70%), TFE
Crystal, powder or solution (0.3 lb/gal)	0,3 lb/gal, 20 min. detention. Good, efficient mixing for Anhy. product. ("Tends to ball")	Gravimetric - Belt Volumetric - Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Hopper agitator and non-flood rotor for Anhy. powder. Solution tank for solution feed. Cont. soln. feed not to practical unless warm H_2O used	Cast Iron, steel rubber, Hypalon, Tyril, TFE, Polypropylene
Powder	20% Slurry	Gravimetric - Belt Volumetric - Helix Slurry Diaphragm Metering Pump Series 1200 & 1700	Agitation Dust collector Rotor, possibly	Steel, Iron, rubber, TFE
Crystal, powder or solution Do not store crystal grade over one day.	0.3 lb/gal. Good, efficient mixing needed for Anhy. & mono. ("Tends to ball") Warm water will help.	Volumetric - Helix Solution Diaphragm Metering Pump - Series 1200 & 1700 & 5700 Rotodip	Hopper agitation for powder. Solution tank for solution feed.	Cast iron, steel, rubber, Hypalon, Tyril, TFE

CHEMICAL	WATER TREATMENT													SEWAGE TREATMENT						INDUSTRIAL WASTE TREATMENT										
	Algae Control	Boiler Water Treatment	Coagulation**	Color Removal	Corrosion and Scale Control	Dechlorination	Disinfection	Fluoridation	Iron & Manganese Removal	pH Control	Softening	Taste and Odor Control	Miscellaneous	B.O.D. Removal	Coagulation**	Condition-Dewater Sludge	Disinfection	Odor Control	Miscellaneous	B.O.D. Removal	Coagulation**	Chlorination	Condition-Dewater Sludge	Dechlorination	Flotation	Neutralization-Acid	Neutralization-Alkali	Oxidation	pH Control	Reduction
Activated Carbon	x			x		x						x																		
Activated Silica			x												x						x									
Alum. Ammonium Sulfate			x				1					1																		
Aluminum Chloride Soln.			(x)												(x)	x					(x)		x							
Alum. Potassium Sulfate			x																											
Aluminum Sulfate			x						x				2		x						x									
Alum. Liquid			x												x						x									
Amines Neutr.		x								x																				
Ammonia, Anhydrous							1					1							3											
Ammonia, Aqua							1					1							3											
Ammon. Chloride							1												17											
Ammonium Silicofluoride								x																						
Ammonium Sulfate							1					1	2						3											
Barium Carbonate		x																												
Bentonite			x	x																	x									
Bromine	(x)						4																							
Calcium Fluoride								(x)																						
Calcium Hydroxide			x	x	x		(x)		x	x	x				x	x		x			x		x		x	x			x	

CHEMICALS USED IN WATER SEWAGE AND WASTE TREATMENT PROCESSES

*Checks indicate normal uses; Checks in parenthesis (x) indicate potential applications, usually dependent on the economics involved.

**Followed by sedimentation.

1 - Chlorine - ammonia treatment for disinfection and tastes and odor control.

2 - Activation of silica.

3 - Sludge digestion control.

4 - Swimming pool disinfection.

Calcium Hypochlorite							x						6				x	x				x								
Calcium Oxide			x	x	x		(x)		x	x	x				x	x		x			x		x		x	x			x	
Carbon Dioxide										x	7		2												(x)		x		x	
Chlorinated Copperas			x												x	(x)					x		(x)							
Chlorinated Lime							x						6				x													
Chlorine				x			x		x			x	6	x			x	x	c	x		x						x		
Chlorine Dioxide				x			x		x			x		(x)			(x)			(x)								(x)		
Copper Sulfate	x											x																		
Diatomaceous Earth				x			x		x			x		x			x	x		x										
Disodium Phosphate		x			x					x	x																			
Dolomitic Hydrated Lime		10								x	10		11								x		x		x	x			x	
Dolomitic Lime		10								x	10		11								x		x		x	x			x	
Dolomitic Limestone																										x				
Ferric Chloride			x												x	x		x			x		x							
Ferric Sulfate			x												x	x					x		x							
Ferrous Sulfate			x												x			x			x									x
Fluosilicic Acid								x																						
Hydrazine		x			x								19																	
Hydrochloric Acid											12		(2)														x		(x)	
Hydrofluoric Acid								(x)																				x		
Iodine							4																							
Mag. Hydroxide								11					11														x			
Mag. Oxide								11					11														x			
o-Dichlorobenzene																		x												
Octadecylamine		x																												

5 - Slurry up-flow coagulation and sedimentation for solids and B.O.D. removal.

6 - Slime control.

7 - Recarbonation in water softening.

8 - Ponding control in activated sludge.

9 - Bulking control in activated sludge.

10 - Silica removal.

11 - Fluoride removal. Also accomplished by magnesium oxide (or hydroxide) and tri-calcium phosphate.

12 - Regeneration of ion exchange materials (zeolites).

13 - To produce chlorine dioxide.

14 - For activated silica coagulation aid.

CHEMICAL	WATER TREATMENT													SEWAGE TREATMENT						INDUSTRIAL WASTE TREATMENT										
	Algae Control	Boiler Water Treatment	Coagulation**	Color Removal	Corrosion and Scale Control	Dechlorination	Disinfection	Fluoridation	Iron & Manganese Removal	pH Control	Softening	Taste and Odor Control	Miscellaneous	B.O.D. Removal	Coagulation**	Condition-Dewater Sludge	Disinfection	Odor Control	Miscellaneous	B.O.D. Removal	Coagulation**	Chlorination	Condition-Dewater Sludge	Dechlorination	Flotation	Neutralization-Acid	Neutralization-Alkali	Oxidation	pH Control	Reduction
Ozone				x			x					x		x						x										
Phosphoric Acid		(x)									16								17								(x)			
Potassium Iodide							±																							
Pot. Permanganate									x			x																		
Sodium Aluminate		x	x	x									(10)		x						x									
Sodium Bicarbonate										x			2												(x)				(x)	
Sodium Bisulfite						x							19											x						x
Sodium Carbonate					x					x	x															x			x	
Sodium Chloride											12																			
Sodium Chlorite (13)				x			x		x			x	13	(x)			(x)			(x)								(x)		
Sodium Chromate					x																									
Sodium Dichromate					x																									
Sodium Fluoride								x																						
Sod. Hexametaphosphate					x							x	18																	
Sodium Hydroxide		x	(x)		x					x	x				x	x					(x)		(x)		(x)	(x)			x	

CHEMICALS USED IN WATER SEWAGE AND WASTE TREATMENT PROGRESSES

15 - Source of O_2
16 - Boiler H_2O Soft
17 - Nutrient
a - See Notes 1 and 11
b - See Notes 2,6 & 13
c - See Notes. 3,6,8 & 9
18 - Sequester ions
19 - O_2 Scavenger

Sodium Hypochlorite							x					x	6	(x)			(x)	x		(x)		x						x		
Sodium Nitrate		(x)																x	15											
Sodium Pentachlorophenate	x												6																	
Sod. Phos. Mono Anhy.		x								x																				
Sod. Phos. Mono Hyd.		x								x																				
Sodium Silicate			14		x																14									
Sodium Silicofluoride								x					2																	
Sodium Sulfite		x			x	x							19											x						x
Sodium Thiosulfate						x																		x						(x)
Sulfamic Acid					x					x																				
Sulfur Dioxide						x							(2)											x						x
Sulfuric Acid										x			2														x		x	
Tetra-Sod. Pyrophosphate		x			x								18																	
Trical. Phosphate								11					11																	
Tri-Sodium Phosphate		x			x					x	x		16																	

CHEMICALS	SHIPPING DATA		CHARACTERISTICS			
FORMULA COMMON NAME	GRADES OR AVAILABLE FORMS	CONTAINERS AND EQUIPMENT	APPEARANCE AND PROPERTIES	WEIGHT lb/cu ft (Bulk Density)	COMMERCIAL STRENGTH	SOLUBILITY IN WATER gm/100 ml
Hydrogen Peroxide H_2O_2 Odor Control. 2 part H_2O_2 = 1 part H_2S. Lessen Sludge Bulking (60 to 400ppm) Treat Phenols SO_2, CN, Mercaptans and Refractory Organics Hypochlorite destruction. Lowers COD. Prevents denitrification. BOD removal control H_2S odor wastewater treatment	Liquid USP 3% Tech. 3, 6, 27.5, 30, 35,50 and 90%	30 - gallon Aluminum Drums Polyethylene (for up to 37% with fiber and steel over pack Amber glass bottles Carboys Tank Trucks Tank Cars	Colorless heavy liquid Unstable. Decomposition Catalyzed by metallic impurities, usually has inhibitor. pH 9 to 10. F.P. 35% = -27,4°F. F.P. 40% = -62°F. Visc. 50% 1,2 cp @20°C 40°C	35% Sp.G.= 1,132 9.4 lb/gal 50% Sp.G= 1,196 10 lb/gal (20°C)	% 35 or 50	Miscible
Oxygen O_2 Oxidation of municipal and industrial wastes. Activated Sludge Treatment	Gas Low Purity High Purity U.S.P. Research (99.99%) Extra Dry (99.6%)	Steel Cylinders Tank Cars	Colorless Odorless Tasteless Gas. Liquid at -183°C to slightly bluish. Sp.G. gas = 1.105 Sp.G. liquid=1.14 Nonliquified gas at 2200 psig at 70°F	Sp. Vol. 12.1 cu. ft/lb	High Purity 99.95% Research Grade 99.99%	1 Vol./32 Vol. Water at 32°F, or 9 mg/l at 20°C and 7.5 mg/l at 30°C and 11.2 mg/l at 100°C

FEEDING RECOMMENDATIONS				
BEST FEEDING FORM	CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	TYPES OF FEEDERS	ACCESSORY EQUIPMENT REQUIRED	SUITABLE HANDLING MATERIALS FOR SOLUTIONS
35 and 50% Liquid	—	Solution Diaphragm Pump 1700 Pump Series (glycol as hydraulic fluid)	Goggles Neoprene Gloves Protective Aprons of Dacron or Orlon	Al Alloy 1160, 1100. Tygon, Koroseal, Polyethylene, Derakane (to 30%) Viton (to 50%) 316 st. steel, to 50%, under certain conditions (Rm), Hast. C (Good)
On Site Generation, especially if using 50 T/D Smaller Amounts Delivered if source close by	Gas Diffused in Water under treatment	Unox Process, Air Products Systems: Airco, F30 System.	—	Glass, 316, Ceramics, Al Teflon. (no grease or oil or easily combustible substance

INDEX