Technologies and Costs for Control of Microbial Contaminants and Disinfection Byproducts

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List of Acronyms

AWWA	American Water Works Association
AWWARF	American Water Works Association Research Foundation
AWWSC	American Water Works Service Company
BAT	Best Available Technology
BCI	building cost index
BDOC	biodegradable organic carbon
BLS	Bureau of Labor Statistics
BV	bed volume
˚C	degrees Celsius
CAP	total capital costs
CDC	Centers for Disease Control and Prevention
CFE	combined filter effluent
CFR	Code of Federal Regulations
CIP	clean-in-place
cm	centimeter
CT	measured disinfectant residual × contact time
CWS	community water system
D/DBP	disinfectant/disinfection byproduct
DBP	disinfection byproduct
DBPR	Disinfectants and Disinfection Byproducts Rule
DES	designated flow
DNA	deoxyribonucleic acid
DOC	dissolved organic carbon
E&I	electrical and instrumentation
EA	economic analysis

EA	environmental assessment
EBCT	empty bed contact time
EIS	environmental impact statement
ENR	Engineering News Record
EPA	United States Environmental Protection Agency
ES	effective size
ESWTR	Enhanced Surface Water Treatment Rule
FACA	Federal Advisory Committee Act
FBRR	Filter Backwash Recycling Rule
fps	feet per second
ft	feet
ft^2 (sf or sq ft)	square feet
FTW	filter to waste
GAC	granular activated carbon
gfd	gallons of filtrate per day per square foot of membrane area
gpd	gallons per day
gpm	gallons per minute
GWUDI	ground water under the direct influence of surface water
HAA	haloacetic acid
HAA5	sum of five haloacetic acids
HAA6	sum of six haloacetic acids
HIV	human immunodeficiency virus
Hp	horsepower
HPC	heterotrophic plate count
hr	hour
HVAC	heating, ventilation, and air conditioning
i	discount rate

I&C	instrumentation and controls
ICR	Information Collection Rule
IESWTR	Interim Enhanced Surface Water Treatment Rule
in	inch
kgal	thousand gallons
kgpd	thousand gallons per day
kW	kilowatt
kWh	kilowatt hour
lb	pound
LOX	liquid oxygen
LP	low pressure
LPHO	low pressure high output
LPUV	low pressure ultraviolet light
LT1ESWTR	Long Term 1 Enhanced Surface Water Treatment Rule
LT2ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
MCL	maximum contaminant level
MCLG	maximum contaminant level goal
M-DBP	microbial-disinfection Byproduct
MF	microfiltration
mg/kg	milligrams per kilogram
µg/L	micrograms per liter
mg/L	milligrams per liter
mgal	million gallons
MGD or mgd	million gallons per day
mJ	milliJoules
mJ/cm^2	milliJoules per square centimeter
µm	micrometer

mm	millimeter
MP	medium pressure
MRDL	maximum residual disinfectant level
MRDLG	maximum residual disinfectant level goal
MWCO	molecular weight cut-off
MWDSC	Metropolitan Water District of Southern California
N	number of years
NDWAC	National Drinking Water Advisory Council
NF	nanofiltration
NIPDWR	National Interim Primary Drinking Water Regulation
nm	nanometers
NOM	natural organic matter
NPDWR	National Primary Drinking Water Regulation
NSF	National Science Foundation
NTNCWS	nontransient noncommunity water system
NTU	nephelometric turbidity units
O&M	operations and maintenance
OGWDW	Office of Ground Water and Drinking Water
OH&P	overhead and profit
OSHA	Occupational Safety and Health Administration
P&V	pipes and valves
PAC	powder activated carbon
PLC	programmable logic controller
POTW	publicly owned treatment works
ppb	parts per billion
ppm	parts per million
PPI	Producer Price Index (for Finished Goods)

PSA	pressure swing absorption
psi	pounds per square inch
psig	pounds per square inch gauge
PUV	pulsed ultraviolet
PVC	polyvinyl chloride
PWS	public water supply
RIA	regulatory impact analysis
RNA	ribonucleic acid
RO	reverse osmosis
SAB	Science Advisory Board
SCADA	Supervisory Control and Data Acquisition
scf	standard cubic feet
SDS	simulated distribution system
SDWA	Safe Drinking Water Act
sf (ft^2 or sq ft)	square feet
SOC	soluble organic carbon
SOC	synthetic organic compound
sq ft (or sf or ft^2)	square feet
SWAT	surface water analytical tool
SWTR	Surface Water Treatment Rule
TDH	total dynamic head
TDP	Technology Design Panel
TDS	total dissolved solids
THM	trihalomethane
THMFP	trihalomethane formation potential
TMP	transmembrane pressure
TNCWS	transient noncommunity water system

TOC	total organic carbon
TOX	total organic halide
TOXFP	total organic halide formation potential
TSS	total suspended solids
TTHM	total trihalomethane
TWG	Technical Work Group
UC	uniformity coefficient
UF	ultrafiltration
UPS	uninterrupted power supply
UV	ultraviolet
UVT	ultraviolet transmittance
UV_{254}	ultraviolet absorbance at 254 nm
VSS	Very Small Systems Best Available Technology Cost Document
wk	week
WTP	water treatment plant
W/W	water and wastewater
yr	year

1. Introduction

1.1 Purpose of Technology and Cost Document

This document provides information on costs and treatment effectiveness of technologies and treatment strategies available to public water systems (PWSs) to remove or inactivate pathogenic microorganisms, specifically *Cryptosporidium*, and/or reduce the formation of disinfection byproducts (DBPs). This information is developed solely for use in conducting Economic Analyses (EAs) for the proposed Stage 2 Disinfectants and Disinfection Byproducts Rule (DBPR) and Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). Please note that the information provided by this document is of a general nature. It is not intended to guide PWSs in selecting or designing technologies for compliance with existing or proposed rules.

The proposed LT2ESWTR will require systems to provide additional *Cryptosporidium* treatment if *Cryptosporidium* concentrations in their source waters exceed specified levels. *Cryptosporidium* is resistant to chlorine but can be inactivated with certain alternative disinfectants or can be physically removed through filtration processes.

The proposed Stage 2 DBPR will require PWSs to reduce the formation of trihalomethanes (THMs) or haloacetic acids (HAAs) if they exceed specified levels. THMs and HAAs form primarily through reactions between chlorine and natural organic matter (NOM). Their formation can be reduced with alternative disinfectants or disinfection practices or through increases in NOM removal prior to chlorine application.

Issues associated with microbial disinfection and the formation of DBPs are interwoven; PWSs should not undercut microbial protection in their efforts to reduce DBP levels. Several of the alternative disinfectants that systems could choose to reduce the formation of THMs and HAAs can provide increased protection against chlorine-resistant pathogens like *Cryptosporidium*. For these reasons, PWSs should have the ability to make decisions regarding compliance strategies for the Stage 2 DBPR and LT2ESWTR at the same time. Consequently, the United States Environmental Protection Agency (EPA) is developing these regulations as a paired rulemaking and is addressing compliance technologies for both rules in a single document.

The EAs for the LT2ESWTR and Stage 2 DBPR evaluate the total impact of a regulation in terms of costs associated with additional treatment requirements and benefits associated with reduced risk. This evaluation requires the following types of information:

- National occurrence of the regulated contaminant(s)
- Existing level of treatment for the contaminant provided by PWSs

- Unit costs and efficacy of treatment strategies available for compliance with the proposed regulation
- Number and sizes of PWSs that will select a particular treatment strategy for regulatory compliance

- Benefits and costs resulting from changes to existing treatment

This document supports the EA by describing the design criteria necessary for a technology to achieve a desired level of treatment and the cost associated with that technology as a function of the design criteria. Information on unit costs and treatment performance is critical to projecting technology usage stemming from a regulation and to evaluating national compliance costs and benefits. No information is given here on the national compliance costs (that information is provided in the EA) or on the numbers of PWSs that will adopt various treatment strategies to comply with the proposed regulations.

Process design criteria for alternative disinfection strategies and DBP precursor removal technologies were developed in large part using water quality data gathered under the Information Collection Rule (ICR) and best engineering judgement. Where appropriate, EPA used ICR data to generate statistics regarding water quality parameters that affect technology performance. These water quality statistics were used to estimate costs for technology options presented in this document. Costs were developed using EPA cost models, manufacturer price data, and recent literature. Unit prices and cost indices for model input were based upon vendor information, prevailing rates, and published values in the trade literature (e.g., *Engineering News Record*, Bureau of Labor Statistics). These costs were reviewed by the Technical Work Group, which was convened by EPA to assist in the Stage 2 DBPR and LT2ESWTR regulatory development process. Subsequent revisions have also been made to respond to comments from outside reviewers, particularly the National Drinking Water Advisory Council (NDWAC) and EPA's Science Advisory Board (SAB).

1.2 Existing Regulations

The following are existing regulations that address risks posed by microorganisms and DBPs in public water systems.

1.2.1 Surface Water Treatment Rule

Under the Surface Water Treatment Rule (SWTR), finalized in 1989, EPA set Maximum Contaminant Level Goals (MCLGs) of zero for *Giardia lamblia*, viruses, and *Legionella*; and promulgated National Primary Drinking Water Regulations (NPDWRs) for all PWSs using surface water or ground water under the direct influence of surface water (GWUDI). Unfiltered systems were required to comply with the SWTR by 1991 and filtered systems by 1993. The SWTR includes treatment technique requirements for filtered and unfiltered systems that are intended to protect against

the adverse health effects of exposure to *Giardia*, viruses, and *Legionella*, as well as other pathogenic microorganisms (63 FR 69478 December 1998b). Briefly, those requirements include the following:

- Maintenance of a disinfectant residual in the distribution system
- Removal/inactivation of 3 log (99.9 percent) for *Giardia* and 4 log (99.99 percent) for viruses
- Combined filter effluent turbidity performance standards
- Watershed protection and raw water quality requirements for unfiltered systems

1.2.2 Information Collection Rule

The ICR is a monitoring and data reporting rule that was promulgated in 1996. The purpose of the ICR was to collect occurrence and treatment information to help evaluate the need for possible changes to the SWTR and microbial treatment practices and to help evaluate the need for future regulation of DBPs. The ICR provided EPA with information on the occurrence of pathogenic microorganisms, including *Cryptosporidium*, *Giardia*, and viruses, as well as the occurrence of DBPs and water quality parameters that impact DBP formation. The ICR also provided engineering data on how PWSs control such contaminants (65 FR 19046 April 2000).

1.2.3 Interim Enhanced Surface Water Treatment Rule

The Interim Enhanced Surface Water Treatment Rule (IESWTR) was finalized in December 1998 and applies only to surface water and GWUDI PWSs serving 10,000 or more people. The purposes of the IESWTR were to improve control of microbial pathogens, specifically *Cryptosporidium* and to address risk trade-offs between pathogens and disinfection byproducts (65 FR 19046 April 2000). Key provisions of the rule include the following:

- MCLG of zero for *Cryptosporidium*
- 2 log (99 percent) *Cryptosporidium* removal requirements for systems that filter
- Strengthened combined filter effluent turbidity standards
- Requirements for individual filter turbidity monitoring
- Disinfection benchmark provisions to ascertain the level of microbial protection provided as systems take steps to comply with new DBP standards

- Inclusion of *Cryptosporidium* in the definition of GWUDI and in the watershed control requirements for unfiltered systems

- Requirements for covers on new finished water reservoirs

- Requirements for sanitary surveys for *all* surface water and GWUDI systems, even those serving fewer than 10,000 people

1.2.4 Stage 1 Disinfectants and Disinfection Byproducts Rule

The Stage 1 Disinfectants and Disinfection Byproducts Rule was promulgated in 1998. The Stage 1 DBPR applies to all PWSs that are community water systems (CWSs) or non-transient non-community water systems (NTNCWSs) and that treat their water with a chemical disinfectant for either primary or secondary disinfection. In addition, certain requirements for chlorine dioxide apply to transient non-community water systems (TNCWSs). Surface water and GWUDI systems serving at least 10,000 people were required to comply with the Stage 1 DBPR by January 2002. All ground water systems, as well as surface water and GWUDI systems serving fewer than 10,000 people, must comply with the Stage 1 DBPR by January 2004.

The Stage 1 DBPR established the following provisions:

- Maximum residual disinfectant level goals (MRDLGs) for chlorine, chloramines, and chlorine dioxide

- MCLGs for three trihalomethanes (bromodichloromethane, dibromochloromethane, and bromoform), two haloacetic acids (dichloroacetic acid and trichloroacetic acid), bromate, and chlorite

- Maximum residual disinfectant levels (MRDL) for chlorine, chloramines, and chlorine dioxide

- MCLs for total trihalomethanes (TTHM), five haloacetic acids (HAA5), bromate, and chlorite

The rule also includes monitoring, reporting, and public notification requirements for the listed compounds. EPA estimates that the rule will provide public health protection for an additional 20 million households not previously covered by drinking water rules for DBPs (65 FR 19046 April 2000).

1.2.5 Long Term 1 Enhanced Surface Water Treatment Rule

The Long Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) (67 FR 1812 January 2002), finalized in January 2002, extends the requirements of the IESWTR to surface water and GWUDI systems serving fewer than 10,000 people.

1.2.6 Filter Backwash Recycling Rule

The Filter Backwash Recycling Rule (FBRR) (66 FR 31086 June 2001) regulates systems in which filter backwash is returned to the treatment process. The rule, promulgated in June 2001, applies to surface water and GWUDI systems that use direct or conventional filtration and recycle spent filter backwash water, sludge thickener supernatant, or liquids from dewatering processes. The rule requires that these recycled liquids be returned to a location such that all steps of a system's conventional or direct filtration process are employed. The rule also requires systems to notify the state that they practice recycling. Finally, systems must collect and maintain information for review by the state.

1.3 Public Health Concerns

1.3.1 Pathogenic Microorganisms

In 1990, EPA's SAB, an independent panel of experts established by Congress, cited drinking water contamination as one of the most important environmental risks and indicated that disease-causing microbial contaminants (e.g., bacteria, protozoa, and viruses) are probably the greatest remaining health risk management challenge for drinking water suppliers (EPA/SAB 1990). Information on the number of waterborne disease outbreaks from the U.S. Centers for Disease Control and Prevention (CDC) underscores this concern. CDC indicates that, between 1991 and 2000, 145 drinking water-related disease outbreaks were reported, with more than 431,000 associated cases of disease (This includes outbreaks in individual water systems, which are not PWSs. About 400,000 cases of illness were from one outbreak.) During this period, a number of agents were implicated as the cause, including protozoa, viruses, and bacteria.

Waterborne diseases are usually acute (i.e., sudden onset and typically lasting a short time in healthy people), and most waterborne pathogens cause gastrointestinal illness, with diarrhea, abdominal discomfort, nausea, vomiting, and/or other symptoms. Some waterborne pathogens cause, or are associated with, more serious disorders such as hepatitis, gastric cancer, peptic ulcers, myocarditis, swollen lymph glands, meningitis, encephalitis, and other diseases.

Cryptosporidium, a protozoan parasite, is of particular concern as a waterborne pathogen because it is highly resistant to inactivation by chlorine and chloramines. In addition, no therapeutic treatment currently exists for cryptosporidiosis, the infection caused by *Cryptosporidium*.

Cryptosporidiosis usually causes 7-14 days of diarrhea, sometimes accompanied by a low-grade fever, nausea, or abdominal cramps in healthy individuals (Juranek 1995). It may, however, cause the death of individuals with compromised immune systems. In 1993, *Cryptosporidium* caused more than 400,000 people in Milwaukee to experience intestinal illness. More than 4,000 were hospitalized, and at least 50 deaths were attributed to the cryptosporidiosis outbreak. Nevada, Oregon, and Georgia have also experienced cryptosporidiosis outbreaks over the past several years.

Despite filtration and disinfection, *Cryptosporidium* oocysts have been found in filtered drinking water (LeChevallier et al. 1991), and many of the individuals affected by waterborne disease outbreaks caused by *Cryptosporidium* were served by filtered surface water supplies (Solo-Gabriele and Neumeister 1996). Surface water systems that filter and disinfect may still be vulnerable to *Cryptosporidium*, depending on the source water quality and treatment effectiveness.

1.3.2 Disinfectants/Disinfection Byproducts

While the use of chemical disinfectants is highly effective in reducing the risk of waterborne disease, disinfectants are known to react with NOM to form byproducts that may pose a public health risk. In addition, the disinfectants themselves may pose a public health risk at high concentrations.

The assessment of public health risks from chlorination of drinking water currently relies on inherently difficult and incomplete empirical analysis. Nevertheless, while recognizing these uncertainties and taking into account the large number of people exposed to DBPs and the different potential health risks that may result from exposure to DBPs (e.g., cancer and adverse reproductive and developmental effects), EPA believes that the weight of evidence represented by the available epidemiology and toxicology studies support a hazard concern and a protective public health approach to regulation.

1.4 Proposed Regulations

1.4.1 Long Term 2 Enhanced Surface Water Treatment Rule

In September 2000, an Agreement in Principle was reached by EPA and members of the Stage 2 Microbial-Disinfection Byproduct (M-DBP) Federal Advisory Committee Act (FACA) Committee regarding the requirements of the proposed LT2ESWTR (65 FR 83015 December 2000). Under the agreement, the LT2ESWTR will require all surface water systems, including GWUDI, that serve at least 10,000 people to conduct two years of source water monitoring for *Cryptosporidium*. Conventional systems whose annual average *Cryptosporidium* concentrations are at least 0.075, 1.0, or 3.0 oocysts per liter would be required to achieve an additional 1, 2, or 2.5 logs, respectively, of *Cryptosporidium* removal or inactivation beyond conventional treatment. Systems could meet these additional treatment requirements through the use of various options including: enhanced filtration

performance, watershed control, alternative disinfectants, membranes, various types of filters, and demonstrations of performance.

1.4.2 Stage 2 Disinfectants/Disinfection Byproducts Rule

The Stage 2 DBPR, which will be proposed along with the LT2ESWTR, will apply to all CWSs and NTNCWSs that add a disinfectant other than ultraviolet (UV) light or deliver disinfected water. Under the Stage 2 M-DBP Agreement in Principle (65 FR 83015 December 2000), the Stage 2 DBPR will retain the MCLs of 80 μg/L for TTHM and 60 μg/L for HAA5 established by the Stage 1 DBPR. However, the Stage 2 DBPR will change the way compliance with these MCLs is determined. Under Stage 1, compliance with the TTHM and HAA5 MCLs is based on a running annual average of all monitoring points within a distribution system. Under the Stage 2 DBPR, compliance would be based on a locational running annual average, which means that the running annual average at each monitoring point within a distribution system would have to be less than the MCL. The Stage 2 DBPR would also require systems to conduct an initial distribution system evaluation which would identify the areas with the highest concentrations of TTHM and HAA5; compliance monitoring will be conducted at those locations.

1.5 Technologies Evaluated for the Control of Pathogens and Disinfection Byproducts

Systems required to provide additional treatment for *Cryptosporidium* under the LT2ESWTR can use two basic mechanisms: inactivation and removal. While chlorine and chloramines are not effective against *Cryptosporidium* at doses used in drinking water treatment, chlorine dioxide, ozone, and UV light have been demonstrated to inactivate this pathogen. Chlorine dioxide and ozone generally require higher doses to inactivate *Cryptosporidium* than those necessary for *Giardia* and viruses; the use of these disinfectants is limited by the formation of regulated byproducts like chlorite and bromate. UV has been shown to achieve high levels of *Cryptosporidium* inactivation at relatively low doses but is currently not widely used in the United States for drinking water treatment. Nevertheless, EPA believes that ozone, chlorine dioxide, and UV are available to PWSs to inactivate *Cryptosporidium*. Consequently, EPA has evaluated these technologies in this document.

PWSs can increase the physical removal of *Cryptosporidium* in their treatment plants by using additional physical barriers like microfiltration (MF) or bag and cartridge filtration. These technologies have been shown to achieve high log reductions of *Cryptosporidium* when properly designed and implemented. This document addresses *Cryptosporidium* removal.

Utilities can also take steps to reduce the concentration of *Cryptosporidium* entering the treatment plant through strategies such as watershed control, pre-sedimentation basins, and bank

filtration. Costs for these technologies were obtained from the M-DBP FACA Committee and are provided in Chapter 4. However, these costs were too uncertain to use in the EA for the LT2ESWTR.

Systems required to reduce the formation of TTHM and HAA5 for compliance with the Stage 2 DBPR can use two approaches. One approach is to reduce the use of free chlorine by switching to disinfectants that do not form, or form only low concentrations of, TTHM and HAA5. Such disinfectants include: chloramines, ozone, chlorine dioxide, and UV. Systems may also reduce free chlorine doses by using physical barriers like microfiltration; microfiltration removes more microorganisms so that less disinfection is needed. This document evaluates chloramines, ozone, chlorine dioxide, UV, and MF as alternative disinfection strategies for reducing TTHM and HAA5 formation. (Note that several of these disinfection strategies were also evaluated for *Cryptosporidium* treatment as described above.)

The second approach for systems to reduce TTHM and HAA5 formation is to increase the removal of DBP precursors (i.e., NOM) prior to disinfection. Systems can remove precursors by increasing coagulation dosages in a process termed enhanced coagulation, or softening, or by installing granular activated carbon (GAC) or nanofiltration (NF). For the purposes of this document, it was assumed that utilities will have already optimized coagulation or softening practices to meet the requirements of the Stage 1 DBPR. As a result, this document evaluates only GAC and NF as precursor removal strategies.

In summary, this document provides an analysis of the following technologies:

<u>Alternative disinfection strategies</u>

- Chloramination
- Chlorine dioxide
- Ultraviolet (UV) light
- Ozone
- Microfiltration and ultrafiltration
- Bag and cartridge filters
- Bank filtration
- Second stage filtration
- Pre-sedimentation basins

- Watershed control
- Combined Filter Performance

Alternative DBP precursor removal strategies

- Granular activated carbon adsorption
- Nanofiltration

1.6 Document Organization

This remainder of this document contains the following sections:

Chapter 2 - Technologies for DBP and Microbial Contaminant Control: Presents comprehensive discussions of all disinfection, *Cryptosporidium* removal, and DBP precursor removal strategies considered in this document. Includes technology descriptions, effectiveness of technologies for DBP precursor and/or microbial control, and factors affecting the performance of each technology.

Chapter 3 - Technology Design Criteria: Discusses the rationale behind development of the design criteria for which costs are presented in Chapter 4. Includes design approach, assumptions and additional factors (e.g., residuals handling) which may impact design.

Chapter 4 - Technology Costs: Presents capital, operations and maintenance, and total annualized costs for each disinfection strategy and DBP precursor removal technology considered. Also includes discussion of estimation methods (e.g., cost models and vendor information).

Chapter 5 - References: Provides a comprehensive bibliography of all literature used in the compilation of this document.

Appendices: Contain capital cost breakdown summaries for technologies for which cost models were used.

2. Technologies for DBP and Microbial Contaminant Control

2.1 Introduction

Public water systems may employ various treatment strategies to reduce chlorinated DBPs and to provide better physical removal or inactivation of *Cryptosporidium* for compliance with the proposed Stage 2 DBPR and LT2ESWTR. EPA considers the following treatment strategies as being available for compliance with these two proposed regulations:

Alternative disinfection strategies

- Chloramination (section 2.2.1)
- Chlorine dioxide (section 2.2.2)
- Ultraviolet light (section 2.2.3)
- Ozone (section 2.2.4)
- Microfiltration and ultrafiltration (section 2.2.5)
- Bag and cartridge filtration (section 2.2.6)
- Bank filtration (section 2.2.7)
- Second stage filtration (section 2.2.8)
- Pre-sedimentation (section 2.2.9)
- Watershed control (section 2.2.10)
- Combined filter performance (section 2.2.11)

DBP precursor removal strategies

- Granular activated carbon adsorption (section 2.3.1)
- Nanofiltration (section 2.3.2)

2.2 Alternative Disinfection Strategies

2.2.1 Chloramination

Chloramines are formed by reactions of ammonia with aqueous chlorine. These reactions may result in the formation of monochloramine (NH_2Cl), dichloramine ($NHCl_2$) and trichloramine (NCl_3). The relative concentrations of these species depend upon the pH of the water and the relative proportion of chlorine and ammonia. At chlorine-to-ammonia mass ratios of 3:1 to 5:1 (Cl_2:NH_3-N) and neutral pHs, conditions common to drinking water treatment, the principal chloramine species formed is monochloramine (USEPA 1999b).

One of the least expensive methods for controlling DBP formation is the use of monochloramine, instead of free chlorine, to maintain a distribution system residual. After the appropriate free chlorine contact time, ammonia is added to quench the residual free chlorine and to retard DBP formation. This reduces the free chlorine contact time and, thus, DBP formation, without compromising microbial protection. The initial free chlorine contact time and chloramine together provide sufficient disinfection. A survey conducted by the American Water Works Association Research Foundation (AWWARF) has shown that most of the utilities that changed disinfection practices to lower distribution system THM levels have done so by switching to chloramine as the secondary disinfectant (McGuire 1989).

Systems that do not use free chlorine for primary disinfection (e.g., that use ozone or UV light) must add chlorine prior to ammonia addition. For most systems, the free chlorine residual needs to be increased prior to the point of ammonia addition to maintain the desired chloramine residual in the distribution system. This can be accomplished by: 1) simultaneous addition of chlorine and ammonia (after primary disinfection with free chlorine or ozone) or 2) the addition of ammonia after chlorine addition.

Further information, including case studies of systems converting from free chlorine to chloramine, is summarized in *Optimizing Chloramine Treatment* (Kirmeyer et al. 1993). This reference supplies additional information on the reason(s) for switching to chloramine and contains information on chloramination changeover and start-up procedures, nitrification, and impact on taste and odor.

2.2.1.1 Efficacy Against Pathogens

Chloramine is less effective than free chlorine for the disinfection of most pathogenic microorganisms. At pH 7 and below, monochloramine is approximately 200 times less effective than free chlorine for coliform inactivation under the same contact time, temperature, and pH conditions. For viruses and cysts, the combined chlorine forms (e.g., monochloramine and dichloramine) are considerably less effective than free chlorine (USEPA 1999b). Historical studies have found time

factors (monochloramine contact time:free chlorine contact time) from 20:1 to 80:1 for the same bacterial inactivation efficiency. For the same conditions of contact time, temperature, and pH, combined chlorine (monochloramine) doses are approximately 25 times higher than free chlorine for the same bacterial inactivation efficiency (White 1999). There is evidence that dichloramine may be twice as effective as monochloramine; however, dichloramine is generally avoided because it contributes to taste and odor problems.

The *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources* (SWTR Guidance Manual–USEPA 1990) presents CT (contact time multiplied by residual disinfectant concentration) values for multiple disinfectants, pathogens, pH and temperature ranges. Exhibit 2.1 compares CT requirements for chloramine with those of free chlorine over a range of temperature and pH values.

Exhibit 2.1: Comparison of CT Values for Free Chlorine and Chloramine

Log Removal	*Giardia*						Viruses					
	<1° C		10° C		20° C		<1° C		10° C		20° C	
	Cl	NH_2Cl	Cl	NH_2Cl	Cl	NH_2Cl	Cl	NH_2Cl	Cl	NH_2Cl	Cl	NH_2Cl
0.5	40	635	21	310	10	185	--	--	--	--	--	--
1	79	1270	42	615	21	370	--	--	--	--	--	--
2	158	2535	83	1230	41	735	6	1243	3	643	1	321
3	237	3800	125	1850	62	1100	9	2063	4	1067	2	534

Note: -- Data not available.
Source: USEPA 1990.

Exhibit 2.1 demonstrates that chloramine is relatively ineffective compared to free chlorine for *Giardia* and virus inactivation. In addition, chloramine is ineffective for inactivation of *Cryptosporidium* (Peeters et al. 1989, Korich et al. 1990). Several studies have evaluated whether disinfection with ozone followed by chloramination (Liyanage et al. 1997a, Driedger et al. 1999) has a synergistic effect on *Cryptosporidium* inactivation (i.e., the inactivation achieved using both disinfectants combined is greater than what is expected for each of the disinfectants separately). The results of these studies are inconclusive but indicate that some synergism may exist for ozone/chloramine applications.

2.2.1.2 DBP Formation

The byproducts formed by chloramination, for the most part, are identical to those produced during chlorination and include THMs, HAAs, haloacetonitriles, and cyanogen chloride. With the possible exception of cyanogen chloride, chloramination does not preferentially form any of the halogenated DBPs compared to free chlorine. In fact, studies have demonstrated that chloramines produce much lower levels of DBPs than free chlorine (Kirmeyer et al. 1993, Symons et al. 1996). This is the primary reason water systems implement chloramines for secondary disinfection rather than free chlorine.

The formation of DBPs resulting from chloramination is influenced by the following treatment variables (Kirmeyer et al. 1993, Carlson and Hardy 1998):

- Contact time and chloramine dosage
- Point of ammonia application
- pH and temperature
- Total organic carbon
- Chlorine-to-ammonia ratio
- Mixing and reaction time for chloramine formation

The point of ammonia application after chlorine addition generally impacts the length of time free chlorine reacts with NOM. For most plants using chlorine as a primary disinfectant, the point of ammonia application depends on disinfection requirements and goals. Once ammonia is added, the rate of DBP formation is significantly reduced (Kirmeyer et al. 1993).

Within the range of chloramine residuals commonly used in the water industry (1 to 5 milligrams per liter (mg/L)), chloramine dose does not appear to be a significant factor in DBP formation; the chlorine-to-ammonia ratio appears to be more significant. TTHM concentrations remain quite low at chlorine-to-ammonia weight ratios less than 5:1, then increase dramatically above the 5:1 ratio (Kirmeyer et al. 1993). Most utilities use chlorine-to-ammonia ratios of 3:1 to 5:1 because dichloramine and trichloramine form at higher ratios. These species are unstable and cause taste and odor problems.

2.2.1.3 Factors Affecting Performance

When chlorine and ammonia are added simultaneously, good mixing can reduce the time free chlorine has to react with NOM. With complete mixing at neutral pHs (7 to 9) and temperatures of 20 to 25 degrees Celsius (˚C), the reaction of ammonia and chlorine to form monochloramine takes less than 3 seconds. This eliminates the free chlorine almost immediately and reduces the potential for DBP formation (Kirmeyer et al. 1993). Efficient mixing and dispersion of chemicals (chlorine and ammonia) at the point of addition determines the extent of free chlorine contact and, thus, substantially impacts the formation of DBPs.

As noted above, pH is important for rapid formation of chloramine. Symons et al. (1996) showed that DBP formation decreased with increasing pH. Exceptions to the trend are noted in some instances at pH 8, where Symons et al. noted that the complexity of chloramine chemistry may cause water-specific responses.

Carlson and Hardy (1998) evaluated the effects of various water quality variables, such as pH, temperature, chlorine dosage, and total organic carbon on THM and HAA formation for waters from five utilities. Of the variables studied, the free chlorine contact time was found to be the most important in forming chlorinated DBPs. Chlorine contact time must be balanced to provide disinfection and to control byproduct formation. The type of DBP precursor was also found to be important. Based on this study, the authors proposed the concept of two sets of precursors: those that form DBPs quickly and those that form DBPs slowly. The precursor material that rapidly reacts with chlorine to form DBPs (i.e. the quick formers) are of greater importance when chloramine is used to maintain a residual. These quick formers are less affected by reaction conditions than are the slow formers. Relatively consistent THM and HAA concentrations formed quickly after the addition of chlorine. Temperature, chlorine dosage, and pH had a greater effect on precursor materials that formed DBPs slowly.

White (1999) summarizes the effect of contact time and dose on the disinfection properties of chloramines. Generally, chloramines require much longer contact times than other chemical disinfectants (e.g., free chlorine and ozone). This is one reason they are more suitable for secondary disinfection in the distribution system, where residence times can be several days. Chloramines are a less powerful oxidant than many other chemical disinfectants and can require substantially higher doses to achieve the same level of disinfection (White 1999). Because longer contact times and higher doses are required for effective chloramine disinfection, residual stability is of major importance. Monochloramine, the preferred chloramine form, is the dominant species at pH levels greater than 5.5 and is essentially the only species present at pH levels around 7.5 (Kirmeyer et al. 1993). Systems using chloramines for secondary disinfection should try to maintain a distribution system pH of approximately 7.5.

A primary concern for systems using chloramines is nitrification in the distribution system. Nitrification is a microbiological process by which free ammonia is converted to nitrite and nitrate. Ammonia oxidizing bacteria and *nitrobacter*, which are naturally present in distribution system biofilms

and may infiltrate leaking or corroding pipes, convert free ammonia to nitrite and (in the presence of sufficient dissolved oxygen) nitrate, respectively. Among the effects of nitrification are a depletion of the chloramine residual and an increase in heterotrophic plate counts (HPC) (Kirmeyer et al. 1995). To prevent nitrification, it is important to optimize the chlorine:ammonia ratio and minimize free ammonia in the distribution system. Nitrification is most likely to occur in distribution system dead ends, areas of low demand, and storage tanks. As a result, the potential for nitrification can also be minimized by improving distribution system piping configurations (e.g., looping to eliminate dead ends and increasing flow in low demand areas) and by increasing storage tank turnover.

2.2.2 Chlorine Dioxide

Chlorine dioxide has been used for drinking water treatment in the United States for more than 50 years, primarily to control taste and odor problems. However, chlorine dioxide has received attention lately because of its potential application for *Cryptosporidium* inactivation (Finch et al.1995, Li et al. 1998) and for reduced formation of THMs or HAAs during disinfection (White 1999). However, chlorine dioxide degrades to form chlorite and chlorate. Chlorite is considered to have public health implications and is a regulated DBP.

Chlorine dioxide cannot be transported because of its instability and explosiveness. Therefore, it is generated on-site. The five common methods for producing chlorine dioxide are as follows: 1) sodium chlorite reaction with acid, 2) chorine solution reaction with chlorite solution, 3) chlorine gas reaction with chlorite solution, 4) reduction of sodium chlorate using hydrogen peroxide and concentrated sulfuric acid, and 5) chlorine gas reaction with solid chlorite (White 1999). The yield, purity, and production capacities of chlorine dioxide vary for the five types of methods. The most common chlorine dioxide generation technique is chlorine solution reaction with chlorite solution. Chlorine dioxide dosages that can be used in drinking water treatment are constrained by regulatory limits on the production of chlorite and chlorine dioxide residual.

2.2.2.1 Efficacy Against Pathogens

The SWTR Guidance Manual presents CT values for inactivation of *Giardia* and viruses for both free chlorine and chlorine dioxide. The values indicate that chlorine dioxide is approximately four times more effective that chlorine for the inactivation of *Giardia* at most conditions. Chlorine, however, is more effective for the inactivation of viruses. Exhibit 2.2 summarizes CT values contained in the guidance manual.

Exhibit 2.2: Comparison of CT Values for Free Chlorine and Chlorine Dioxide

Log Removal	*Giardia*						Viruses					
	<1° C		10° C		20° C		<1° C		10° C		20° C	
	Cl	ClO_2	Cl	ClO_2	Cl	ClO_2	Cl	ClO_2	Cl	ClO_2	Cl	ClO_2
0.5	40	10	21	4	10	2.5	--	--	--	--	--	--
1	79	21	42	7.7	21	5	--	--	--	--	--	--
2	158	42	83	15	41	10	6	8.4	3	4.2	1	2.1
3	237	63	125	23	62	15	9	25.6	4	12.8	2	6.4

Note: -- Data not available.
Source: USEPA 1990.

Chlorine dioxide has been compared to other oxidants for inactivating *Cryptosporidium* (Korich et al. 1990); chlorine dioxide and ozone are found to be more effective in inactivating *Cryptosporidium* compared to chlorine and monochloramine. However, unlike ozone, the degradation byproducts of chlorine dioxide do not contribute to the inactivation of *Cryptosporidium* (Liyanage et al. 1997b). The American Water Works Service Company (AWWSC) evaluated the effectiveness of chlorine dioxide for the inactivation of *Cryptosporidium* (AWWSC 1998). AWWSC found that chlorine dioxide is effective for warm, high pH waters (pH of approximately 8 and temperature around 20 degrees Celsius). Finch et al. (1995) summarized the chlorine dioxide research regarding the inactivation of *Cryptosporidium*. A summary of CT values for *Cryptosporidium* is presented in Exhibit 2.3.

Exhibit 2.3: Summary of Chlorine Dioxide CT Values for *Cryptosporidium* Inactivation

Log Inactivation	AWWSC		From Summary by Finch et al. (1995)
	10° C	20° C	
1	99	48	60
2	257	115	80
3	--	--	140

Note: -- Data not available.
All values are for pH 8.
Temperature for Finch et al. is unknown.

Chlorine dioxide has also been proven effective for the inactivation of selected bacteria over a pH range of 3.0 to 8.0 (Junli et al. 1997, White 1999) and is a stronger disinfectant than chlorine for bacteria, requiring lower CT values. Some of the bacteria evaluated in Junli et al. (1997) are *E. coli* (A and B), *Staphylococcus aureus*, *Sarcina*, *Chloropseudomonas*, *Bacillus subtilis*, and *Shigella dysenteriae*.

2.2.2.2 DBP Formation

Studies have demonstrated that chlorine dioxide does not produce THMs (White 1999); under proper generation conditions (i.e., no excess chlorine), halogen-substituted DBPs are not formed. The application of chlorine dioxide produces only a small amount of total organic halide (TOX) (Werdehoff and Singer, 1987). The use of chlorine dioxide aids in reducing the formation of TTHMs and HAAs by oxidizing precursors. By moving the point of chlorination downstream in the plant after coagulation, sedimentation, and filtration, the quantity of NOM is reduced. This results in a lower chlorine dosage during post-chlorination of the water which, in turn, results in fewer THMs.

In normal pH ranges (6 to 9), chlorine dioxide undergoes a variety of oxidation reactions with NOM to form oxidized organic species, such as chlorinated, brominated, or polysubstituted organic byproducts and chlorite (ClO_2^-). Chlorite concentrations can account for up to 70 percent of the chlorine dioxide consumed (American Water Works Association (AWWA) 1999; Werdehoff and Singer 1987). Chlorite, and chlorate (ClO_3^-) are formed when chlorine dioxide is added to water. All three oxidized chlorine species (chlorine dioxide, chlorite, and chlorate) are considered to have adverse health effects and are of concern in finished water (AWWA 1999).

Chlorine dioxide may also facilitate a number of chlorine substitution reactions. Studies evaluating drinking water and NOM have shown that TOX concentration increases upon application of chlorine dioxide at normal treatment dosages (AWWA 1999).

2.2.2.3 Factors Affecting Performance

Temperature dramatically affects *Cryptosporidium* inactivation by chlorine dioxide (Li et al. 1998). At 1 ^oC, a 0.5 log inactivation is observed at a CT of 150 milligrams * minutes / liter (mg-min/L), compared to a 2.0 log inactivation for the same CT at 22"C. Chlorine dioxide can effectively inactivate bacteria over a pH range of 3.0 to 8.0. Because it is a more effective disinfectant for bacteria than free chlorine, lower CT values are required. Caution must be taken, however, when selecting the appropriate dose, as excessive dosages can lead to chlorite formation above permissible levels. Purity and generator yields are two of the most critical factors that effect chlorine dioxide use. Chlorine and the oxychlorine species (i.e., chlorite and chlorate) are typically present in the impurities of chlorine dioxide (White 1999). Therefore, the purity of the chlorine dioxide generated should be considered to avoid a violation of the chlorite maximum contaminant level (MCL).

2.2.3 Ultraviolet Light

The use of UV light for disinfection of drinking water has recently received much attention because of new developments regarding *Cryptosporidium* inactivation at low UV light doses (Bukhari et al. 1999) and because it creates very few DBPs. Disinfection is accomplished by irradiating water with UV light. UV light is electromagnetic radiation between wavelengths of 100 and 400 nanometers (nm). The specific range of UV wavelengths and the level of irradiance depend on the type of UV lamp system used. The effective germicidal wavelength range for most microorganisms is generally considered to be between 200 and 300 nm (Malley 1998).

UV systems consist of UV reactors with an associated control panel. Commercial UV reactors used for drinking water applications are closed reactors containing UV lamps, quartz sleeves, UV intensity sensors, quartz sleeve wipers, and temperature sensors. UV lamps are housed within the quartz sleeves, which protect and insulate the lamps. Some reactors include automatic cleaning mechanisms to keep the quartz sleeves free of deposits that may form due to contact with the water. UV intensity sensors, flow meters, and in some cases, UV transmittance monitors are used to monitor dose delivery by the reactor.

UV lamps can be divided into two categories: continuous wave and pulsed wave. Currently, continuous wave UV lamps are most widely used for drinking water treatment. The types of continuous wave lamps are low pressure mercury vapor (LP), low pressure high output (LPHO), and medium pressure mercury vapor (MP). "Pressure" refers to the pressure of mercury vapor within the lamp casing. A comparison of the LP, LPHO, and MP lamps is shown in Exhibit 2.4.

Exhibit 2.4: Comparison of UV Lamps

Parameter	LP	LPHO	MP
Germicidal UV light	Monochromatic at 254 nm	Monochromatic at 254 nm	Polychromatic, including germicidal range (200 - 300nm)
Mercury Vapor Pressure (torr)	Optimal at 0.007	Optimal at 0.007	100 - 10,000
Operating Temperature (^{o}C)	Optimal at 40	130 - 200	600 - 900
Electrical Input (W/centimeter (cm))	0.5	1.5 - 10	50 - 150
Germicidal UV Output (W/cm)	0.2	0.5 - 3.5	5 - 30
Electrical to Germicidal UV Conversion Efficiency (%)	35 - 38	30 - 40	10 - 20
Arc Length (cm)	10 - 150	10 - 150	5 - 75
Relative Number of Lamps Required for a Given Dose	High	Intermediate	Low
Lifetime (hours(hrs))	8,000 - 10,000	8,000 - 12,000	3,000 - 5,000

Source: EPA UV Disinfection Guidance Manual (USEPA 2003).

The light emitted by LP and LPHO lamps is essentially monochromatic at 253.7 nm, which is in the range of the most germicidal wavelengths for microorganisms. MP lamps emit at a higher intensity than LP lamps but at a wide range of wavelengths. Therefore, LP and LPHO lamps convert power to germicidal light more efficiently than MP lamps. Theoretically, LPHO lamps have the same efficiency as LP lamps because they operate at similar vapor pressures. However in practice, LPHO lamp efficiency can be significantly lower when operating at different power settings. The main differences between LP and MP lamps, as shown in Exhibit 2.4, are the vapor pressure, operating temperatures, electrical input, and germicidal UV output.

Pulsed ultraviolet (PUV) systems irradiate a high intensity UV light in flashes at approximately 50 flashes per second. PUV systems have limited operating experience on the full-scale and are not costed in this document.

The UV lamp ballast controls the amount of electricity supplied to the lamp and should ensure a consistent and constant power delivery. Power supplies and ballasts can be supplied in many different configurations and are tailored to a unique lamp type and application. UV systems may use electronic ballasts, magnetic ballasts, or transformers.

UV intensity sensors are photosensitive detectors that measure the UV intensity at a point within the UV reactor. This intensity information is used to indicate dose delivery. Intensity sensors can be classified as wet or dry. Dry sensors monitor UV light through a monitoring window whereas wet UV intensity sensors are in direct contact with the water flowing through the reactor. Monitoring

windows and the wetted ends of the wet sensors can become fouled over time and require cleaning, similar to quartz sleeves.

The lamp cleaning mechanism used for a UV system depends on the lamp type, system size, and fouling potential of the water. Both manual and automatic cleaning regimes have been developed. Manual cleaning is primarily used for smaller systems with relatively few sleeves and lower fouling potential. Automatic cleaning approaches may be classified as flush and rinse systems, mechanical wipers, or physical-chemical wipers. LPHO systems typically use flush and rinse systems, and MP systems typically use wipers because the higher lamp temperatures accelerate fouling under certain water qualities. The cleaning frequency of the lamps is a function of the lamp temperature and the concentration of dissolved organic and inorganic species that can cause lamp fouling.

2.2.3.1 Efficacy Against Pathogens

When UV light is applied to a microorganism, deoxyribonucleic acid (DNA) and ribonucleic acid (RNA) absorb the light energy and their structure is altered, thereby interfering with replication of the microbe. The UV dose necessary for inactivation of microorganisms varies from species to species and across microorganism classifications. Inactivation of microorganisms increases with increasing UV dose, although it does not always follow the typical log-linear relationship.

Of the pathogens of interest in drinking water, viruses are most resistant to UV disinfection, followed by bacteria and protozoa. Exhibit 2.5 presents UV dose requirements for inactivation of *Cryptosporidium*, *Giardia*, and viruses (as derived in the USEPA UV Disinfection Guidance Manual, Appendix B). The UV dose requirements presented in Exhibit 2.5 are the minimum required; operational UV doses will likely be two to four times higher after application of a safety factor.

Exhibit 2.5: UV Dose Requirements for Inactivation of *Cryptosporidium*, *Giardia*, and Viruses During Validation Testing

	Log Inactivation							
	0.5	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**
Cryptosporidium	1.6	2.5	3.9	5.8	8.5	11.7	-	-
Giardia	1.5	2.1	3.0	5.2	7.7	10.8	-	-
Virus	39.4	58.1	79.1	100.1	120.7	142.6	163.1	186.0

Note: All values presented in mJ / cm^2
Source: USEPA UV Disinfection Guidance Manual, Appendix B.

Based on the analysis presented in Appendix B of the EPA UV Disinfection Guidance Manual, the sensitivities of *Giardia* and *Cryptosporidium* to UV disinfection are very similar; viruses, however, are more difficult to inactivate. Battigelli et al. (1993) performed bench scale UV collimated beam experiments to determine the relationship between UV dose and inactivation of Hepatitis-A virus (strain HM-175), coxsackievirus type B-5, rotavirus strain SA-11, and bacteriophages MS-2 and fX174. MS-2 bacteriophage required the highest dose of 25 milliJoules per square centimeter (mJ/cm^2) for less than 1 log inactivation. With the other viruses, 4 log inactivation is achieved at doses ranging between 16 and 42 mJ/cm^2. The most UV-resistant viruses of concern in drinking water are adenovirus Type 40 and Type 41. Meng and Gerba (1996) report a dose of 23.6 to 30 mJ/cm^2 for a 1 log inactivation in adenovirus and a dose of 111.8 to 124 mJ/cm^2 for 4 log inactivation.

Because microbes that have been exposed to UV light still retain metabolic functions, some are able to repair the damage done by UV light and regain infectivity. Repair of UV light-induced DNA damage includes photoreactivation and dark repair (Knudson 1985). Photoreactivation (or photorepair) is an enzymatic DNA repair mechanism wherein the DNA damage is repaired when exposed to light between 310 and 490 nm. Dark repair is an enzymatic repair mechanism that does not require light. Not all microorganisms contain the necessary cellular mechanisms to repair UV-damaged DNA. One study has shown that *Cryptosporidium* contains the capability to undergo some DNA repair. However, even though the DNA was repaired, infectivity was not restored (Oguma et al. 2001). Another study, by Shin et al. (2001), did not observe photorepair with *Cryptosporidium parvum*. Linden et al. (2002a) did not observe photoreactivation or dark repair of *Giardia* at UV doses typical for UV disinfection applications (16 and 40 mJ/cm^2). However, unpublished data from the same study showed *Giardia* reactivation in light and dark conditions at very low UV doses (0.5 mJ/cm2; Linden 2002a). Shaban et al. (1997) found that viruses also lack the repair enzymes necessary for photoreactivation. However, photorepair of viral DNA can occur using the enzyme systems of their host cells. Knudson (1985) found that bacteria were able to repair in light and dark conditions, suggesting that bacteria may have the enzymes necessary for photorepair and dark repair. As such, photoreactivation is generally limited to bacteria.

E. coli and HPC inactivation by UV light are well documented, particularly with respect to wastewater disinfection (Chang et al.1985, Wilson et al. 1992). Photoreactivation of bacteria has been documented with *E. coli*, *S. aureus*, and coliphage, while dark repair has been documented with *S. aureus* and coliphage (Shaban et al. 1997). One study (Knudsen, 1985) examined two different strains of *E. coli*: one that had the enzymes necessary for repair (B/R strain) and one that lacked the necessary repair enzymes ($recA^-$ uvr^- strain). The difference in UV dose needed for 1-log inactivation of the strain capable of repair was over two orders of magnitude higher than the dose needed for 1-log inactivation of the repair deficient strain, indicating that dark repair impacts the UV dose-response of microorganisms. Unlike bacteria, viruses do not have the enzymes necessary for dark repair. However, viruses can repair in the host cell using the host cells' enzymes (Rauth 1965).

2.2.3.2 DBP Formation

Several studies have been conducted to determine if DBPs are formed as a result of UV light irradiation. Zheng et al. (1999) found that TTHM and HAA9 formation did not increase when UV light was applied to chlorinated water at a dose of 100 mJ/cm^2. Linden et al. (1998) investigated DBP formation in wastewater secondary effluent that is irradiated with LP and MP UV lamps and found no evidence of photochemical reactions or DBP formation. Malley et al. (1996) examined the effects of post-UV disinfection (chlorination and chloramination) on DBP formation and found no significant impact by UV on DBP levels formed by chemical disinfection. Malley et al. (1995) also observed no significant change in THM, HAA, bromate, or other halogenated DBP concentrations following disinfection with UV light. A study performed with filtered drinking water indicated no significant change in aldehydes, carboxylic acids, or TOX (Kashinkunti et al., 2003). However, a low conversion rate (about one percent) of nitrate to nitrite by UV light has been observed (von Sonntag and Schuchman, 1992). Conversion of nitrate to nitrite is lower with LP lamps than with MP lamps because the UV absorbance of nitrate is higher below 240 nm than it is at 254 nm. Due to the low conversion rate of nitrate to nitrite by UV light, it is of minimal concern in drinking water applications. Several studies have shown low-level formation of non-regulated DBPs (e.g., aldehydes) as a result of applying UV light to wastewater and raw drinking water sources. The difference in results can be attributed to the difference in water quality, most notably the higher concentration of organic material in raw waters and wastewaters.

2.2.3.3 Factors Affecting Performance

Particle content can impact UV disinfection performance. Particles may absorb and scatter light, thereby reducing the UV intensity delivered to the microorganisms. Particle-associated microbes also may be shielded from UV light, effectively reducing disinfection performance. Particles in source waters are diverse in composition and size and include large molecules, microbes, clay particles, algae, and flocs.

Recent research by Linden et al. (2002b) indicates that the UV dose-response of microorganisms added to filtered drinking waters was not altered by variation in turbidity that met regulatory requirements. For unfiltered raw waters, Passantino and Malley (2001) found that source water turbidity up to 10 nephelometric turbidity units (NTU) did not impact the UV dose-response of separately added (seeded) organisms. In these experiments, however, organisms were added to waters containing various levels of treated or natural turbidity. Therefore, it was not possible to examine microbes associated with particles in their natural or treated states. Consequently, these drinking water studies can only suggest the impact of turbidity on dose-response as it relates to the impact of UV light scattering by particles. The studies cannot predict the effect on UV disinfection of microbes attaching to particles.

UV absorbance, often exerted by dissolved organic matter in drinking water applications, affects the design of the UV system. Water that absorbs a significant amount of UV light (i.e., high UV absorbance and low transmittance) will need a higher UV irradiance or longer exposure to achieve the same level of inactivation as water with lower UV absorbance. As UV absorbance increases, the intensity throughout the reactor decreases for a given lamp configuration. This results in a reduction in delivered dose and measured UV intensity for a given lamp output. In a situation with a fixed UV output, lower UV absorbance values result in more UV energy being available in the water column, causing a higher log-inactivation of microorganisms than a water with a higher UV absorbance. For systems with high levels of dissolved organic matter (high UV absorbance), it is more efficient to apply UV light after unit processes that remove organic matter.

Several chemicals used in water treatment processes can increase the UV absorbance of water (e.g., Iron (Fe^{+3})). However, some oxidants (including ozone) can reduce the UV absorbance (APHA et al. 1998). Water treatment processes upstream of the UV reactors can be operated to control and reduce UV absorbance, thereby optimizing the design and costs of the UV system.

Depending on the water quality (e.g., dissolved ions, hardness, alkalinity, and pH levels) and lamp temperature, scale can form on the UV lamps. MP lamps tend to scale more easily than LP and LPHO lamps because the operating temperature of MP lamps is considerably higher. Scale can reduce the UV energy being transmitted through the lamp sleeve into the water and potentially compromise disinfection. Lamp cleaning is an important consideration for the design of UV systems to control lamp scaling and to ensure consistent disinfection performance. Water pH may also affect lamp scale formation, but inactivation of microorganisms with UV light is not pH dependent (Malley 1998).

UV inactivation of microorganisms is not directly affected by water temperature. However, the performance of UV lamps is dependent on the lamp temperature. Most UV lamps have sleeves (usually made of quartz) that insulate the lamps, maintain optimal temperature, and provide maximum irradiance. If the lamp temperature deviates from optimal, the lamp irradiance will be reduced. This is especially true with LP UV lamps in cold waters (Mackey et al. 2000). Therefore, the water temperature variation should be considered when designing a low pressure system. However, MP lamps have a significantly higher operating temperature compared to the water temperature. Thus, as long as an insulating quartz sleeve is in place, the water temperature has little effect on the operating temperature of the MP lamp and MP lamp performance.

Hydraulics are an important part of the UV equipment. Ideally, the UV reactor should exhibit plug-flow characteristics. In plug flow, water that enters the reactor is completely mixed axially and moves through the reactor as a single plug with no dispersion in the direction of flow. However, "real world" hydraulics in a full-scale reactor are never plug flow. UV reactors are typically equipped with baffles to reduce the amount of short-circuiting through the reactor and to encourage plug flow, although these baffles can increase head loss through the reactor. Staggered lamp arrays also promote mixing within the reactor and minimize short-circuiting of flow. Alternatively, vortex mixers can be used to increase lamp spacing, thereby reducing head loss through the reactor.

Inlet and outlet conditions can have a significant impact on reactor hydrodynamics. Straight inlet conditions with gradual changes in cross sectional area can be used to develop flow for optimal dose delivery. Straight inlets with no sharp bends or sudden changes in cross sectional area optimize dose deliveries.

It may be necessary to characterize the reactor flow regime in order to determine the level of disinfection provided. Tracer tests are typically not feasible because the hydraulic residence time in the reactor is too short (i.e., on the order of seconds or fractions of a second). However, hydraulic models are available to understand the behavior of the UV reactor.

2.2.4 Ozone

In recent years, the use of ozone technology in water treatment has dramatically increased. In 1991, approximately 40 water treatment plants in the United States, each serving more than 10,000 people, utilized ozone (Langlais et al. 1991). As of April 1998, this number had grown to 264 operating plants (Rice et al. 1999). The main reasons for the escalating use of ozonation are the strong oxidizing properties of ozone and the absence of the formation of chlorinated DBPs during disinfection (however, bromated DPBs are formed).

In water, ozone reacts with hydroxide ions (OH^-) to form hydroxyl free radicals (HO^1); therefore, pH is a very important parameter in determining the extent and rate of contaminant oxidation. Oxidation with ozone is also influenced by other water quality characteristics, such as temperature, alkalinity, and the concentration of reduced chemical species (i.e., iron and manganese). Other important considerations include ozone dose and contact time.

Ozone is commonly added to raw water (pre-ozonation) or settled water. To take advantage of ozone's ability to improve flocculation and NOM removal, ozone must be applied to raw water. Application of ozone to raw or settled water is considered to be equally effective for primary disinfection. However, larger doses may be necessary for raw water application due to the higher NOM and particulate matter concentrations.

There are two basic types of ozone generation equipment: liquid oxygen-based systems and air-based systems. Liquid oxygen feed systems are relatively simple (e.g., there is no air pretreatment equipment), less capital intensive, and yield a higher ozone concentration than air-based systems. The liquid oxygen feed system components include a storage tank, an evaporator to convert the liquid to a gas, filters to remove impurities, and pressure regulators to limit the gas pressure to the ozone generators.

Air-fed systems require air pretreatment equipment to prevent damage to the ozone generator. Air needs to be dry, free of contaminants, and with a dew point between -50" and -60"C. Air

pretreatment equipment consists of compressors, after coolers (optional), refrigerant dryers, desiccant dryers, air filters, and pressure regulators. Power consumption is higher for air feed systems (8-12 kWh/lb O_3) than for oxygen feed systems (4-8 kWh/lb O_3). Exhibit 2.6 presents a comparison of the advantages and disadvantages of the two types of ozonation systems (USEPA 1999b).

Exhibit 2.6: Comparison of Air and Liquid Oxygen Systems

System	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 1-5 % Higher power consumption Fairly complicated technology
Liquid Oxygen	Less equipment required Simple to operate and maintain Suitable for small and large systems Can store excess oxygen to meet peak demands Higher ozone concentration (14-18%) Approximately doubles ozone production for same generator Lower power consumption	Variable liquid oxygen costs Storage of oxygen onsite (i.e., safety concerns) Loss of liquid oxygen in storage when not in use Oxygen-resistant materials required

Ozone is usually applied in one of three flow configurations: 1) co-current (ozone and water flowing in the same direction), 2) counter-current (ozone and water flowing in the opposite direction), or 3) alternating co-current/counter-current. Ozone application systems include fine bubble diffusers, injectors/static mixers, and turbine mixers (Langlais et al. 1991). The fine bubble diffuser system is the most common and offers high ozone transfer rates, process flexibility, operational simplicity, and no moving parts. The injector/static mixer system applies ozone in an in-line or a sidestream configuration. Ozone is injected under negative pressure, created by a venturi section, and then mixed to enhance dispersion of ozone in the water stream. The turbine mixer systems feed ozone in the contactor and mix ozone with the water in the contactor. The turbine mixer can either project outside of the ozone contactor or be submerged.

Hoigne and Bader (1976) described ozone decomposition in water. Once ozone enters solution, it follows one of two reaction pathways: 1) direct oxidation, which is slow and selective in its oxidation of organic compounds, and 2) autodecomposition to the hydroxyl free radical ($HO^{\bullet}$), which is extremely fast and nonselective. The hydroxyl free radical is scavenged by carbonate and bicarbonate ions, commonly measured as alkalinity, to form carbonate and bicarbonate free radicals. These radicals do not affect the organic reactions. The hydroxyl radicals produced by the autodecomposition react with organics and other radicals to reform hydroxyl radical in an autocatalytic process.

The stability of dissolved ozone is affected by pH, ultraviolet light, ozone concentration, and the concentration of radical scavengers (Langlais et al. 1991). Conditions of low pH favor the direct oxidation pathway, and high pH conditions favor the autodecomposition pathway described earlier. At pH levels between 3 and 6, the ozone is present primarily in its molecular form (O_3), and direct oxidation dominates. However, as the pH rises, the autodecomposition of ozone to produce the hydroxyl free radical (HO•) becomes increasingly rapid. At pH levels greater than 10, the conversion of molecular O_3 to HO• is virtually instantaneous. In general, better disinfection would be expected at lower pHs, since free hydroxyl radicals are short-lived compared to molecular ozone. Studies have shown that increasing the temperature from 0" to 30" C reduces the solubility of ozone and increases its decomposition rate (Kinman 1975).

2.2.4.1 Efficacy Against Pathogens

Ozone is one of the most potent biocides used in water treatment. It is effective against a wide range of pathogenic microorganisms including bacteria, viruses, and protozoa. Ozone shows greater efficiency inactivating most types of pathogenic microorganisms than chlorine, chloramine, and chlorine dioxide (Clark et al. 1994). This is demonstrated by the CT values found in the SWTR Guidance Manual presented in Exhibit 2.7. The resistance of pathogenic microorganisms to ozone increases in the following order: bacteria, viruses, protozoa (Camel and Bermond 1999).

Exhibit 2.7: Comparison of CT Values for Free Chlorine and Ozone

Log Removal	*Giardia*						Viruses					
	<1° C		10° C		20° C		<1° C		10° C		20° C	
	Cl	O_3	Cl	O_3	Cl	O_3	Cl	O_3	Cl	O_3	Cl	O_3
0.5	40	0.48	21	0.23	10	0.12	--	--	--	--	--	--
1	79	0.97	42	0.48	21	0.24	--	--	--	--	--	--
2	158	1.9	83	0.95	41	0.48	6	0.9	3	0.5	1	0.25
3	237	2.9	125	1.43	62	0.72	9	1.4	4	0.8	2	0.4

Note: -- Data not available Source: USEPA (1990)

Small concentrations of ozone are usually effective against bacteria. *E. Coli* levels were reduced by 4 log (99.99 percent removal) in less than one minute at an initial ozone concentration of 9 micrograms per liter (µg/L) (Wuhrmann and Meyrath 1955). *Legionella pneumophila* levels were reduced by 2 log (99 percent removal) in less than five minutes at an initial ozone concentration of 0.21 milligrams per liter (mg/L) (Domingue et al. 1988).

Typically, viruses are more resistant to ozone than bacteria, although ozone is still effective against viruses. Ozone dosages of 0.2 to 1.5 mg/L consistently achieved 2 log inactivation of poliomyelitis viruses with a contact time of 40 seconds (Katzenelson et al. 1974). Katzenelson et al. (1974) also observed that poliomyelitis inactivation increased to nearly 5 log at a dose of 1.5 mg/L and a contact time of approximately 100 seconds. Coxsackie virus inactivation is more than 5 log with an initial ozone dosage of 1.45 mg/L (Keller et al. 1974). The sensitivity of human rotavirus to ozone was found to be similar to that of coxsackie virus (Vaughn et al. 1987).

Protozoan cysts are more resistant to ozone than bacteria and viruses. Data available for inactivation of *Cryptosporidium* oocysts suggest that, among protozoans, this pathogen is the most resistant to ozone (Peeters et al. 1989; Langlais et al. 1990).

Ozone inactivation kinetics of *Cryptosporidium* are evaluated by Gyurek et al. (1999). The observed inactivation behavior of *Cryptosporidium* by ozone is characterized by a "tailing-off" effect. At 22"C and a 5 minute contact time, an initial ozone residual of 1.2 mg/L was required to provide 2 log inactivation. For contact times less than 5 minutes, a relatively small increase in the applied contact time significantly decreases the required initial ozone residual; however, for contact times greater that 10 minutes an increase in the applied contact time provides a negligible decrease in the required initial ozone residual. Hence, the influence of contact time on the inactivation kinetics decreases as *Cryptosporidium* is progressively exposed to ozone.

CT values for ozone inactivation of *Cryptosporidium* are still under development. Initial studies have demonstrated that CT values may be as much as 25 times higher than those required for *Giardia* (Rennecker et al. 1999). These preliminary studies also demonstrate that CT requirements for *Cryptosporidium* inactivation increase by an average factor of approximately three for every 10" C decrease in temperature. A summary of reported ozonation requirements for 2 log inactivation of *Cryptosporidium* oocysts is presented in Exhibit 2.8.

Exhibit 2.8: Reported Ozonation Requirements for 2 log Inactivation of *Cryptosporidium* Oocysts

Experimental Protocol	Initial Ozone Residual (mg/L)	Temperature (^{0}C)	Contact Time (min)	CT (mg-min/L)	Reference
Batch liquid, batch ozone	0.77 0.51	Ambient	6 8	4.6 4.0	Peeters et al. 1989
Batch liquid, continuous gas	1.0	25	5-10	5-10	Korich et al. 1990
Batch liquid, batch ozone	0.50 0.50	7 22	18 7.8	9.0 3.9	Finch et al. 1993
Flow through contactor, continuous gas		22-25	7.4	5.5	Owens et al. 1994
Batch liquid, batch ozone	0.7	22	3.2	3.2	Gyurek et al. 1999

Note: Owens et al. do not report residual dose.

2.2.4.2 DBP Formation

Ozone does not produce chlorinated DBPs; however, through the oxidation of natural organic precursor materials, ozone can alter the reactions between chlorine and NOM and affect the formation of chlorinated DBPs when chlorine is added downstream. Ozonation of natural waters produces aldehydes, haloketones, ketoacids, carboxylic acids, and other types of biodegradable organic material which must be adequately controlled (often with a granular media biofilter).

Ozonation often increases the biodegradability of NOM in the treated water. Increasing biodegradability could be beneficial if a biological filtration process follows the ozonation step. A biological filtration step can remove the biodegradable fraction of NOM, increasing organic precursor removal. Biological filters remove NOM by using it as a substrate. Biological filtration can be employed on adsorptive media, such as GAC, and/or non-adsorptive media, such as sand and anthracite. Conversely, if the biodegradable fraction is not removed, it can increase the regrowth of microorganisms in the distribution system.

Haag and Hoigne (1983) have shown that ozone oxidizes bromide to form hypobromous acid and hypobromite (HOBr and OBr^-) under water treatment conditions. Hypobromite was found to be further oxidized to bromate or to a species that regenerates bromide, whereas HOBr reacts with NOM to form brominated organic byproducts in waters containing bromide.

Changes in pH can have a dramatic effect on the concentrations of HOBr and OBr^- and, therefore, the species of byproducts formed. An increase in pH increases the relative concentration of Br^-, which, in turn, leads to increased bromate formation. Reduced pH levels are often accompanied by a reduction in bromate concentrations; the lower pH enhances formation of bromoform and other organic brominated DBPs.

Krasner et al. (1989) found that an ozone residual is necessary to produce detectable levels of bromate. Siddiqui and Amy (1993) found that the bromoform concentration first increased then diminished at higher dosages. Song et al. (1995) demonstrated that lower ozone dosage and longer contact time should produce less bromate than higher dosages and shorter contact times.

Halogenated organic compounds are formed when NOM reacts with free chlorine or free bromine. Free bromine can be formed in ozone disinfection whenever bromide is present in the raw water source. The level of brominated byproducts formed during oxidation is dependent on the concentration of bromide in the raw water source and/or the relative amount of bromide present compared to organic precursors.

Ozonation followed by chlorination has been observed to produce higher levels of haloketones than chlorination alone (Jacangelo et al. 1989b). Chloral hydrate occurs primarily as a result of chlorination, although ozonation followed by chlorination has been observed to increase levels beyond those observed with chlorination only. Ozonation followed by chlorination or chloramination can increase chloropicrin levels above those observed with chlorination or chloramination alone. Ozonation followed by chloramination has been observed to increase cyanogen chloride levels beyond those observed with chloramination only. Cyanogen bromide, the brominated analog of cyanogen chloride, has been detected after ozonation of water containing high bromide levels (McGuire et al. 1990).

Much less is known about non-halogenated disinfection byproducts than the halogenated organic compounds. Among the major ozonation byproducts, aldehydes and carboxylic acids have the highest concentrations (Glaze et al. 1993). Ozonation followed by chlorination has been found to yield the highest levels of acetaldehyde and formaldehyde. In addition, ozonation prior to chloramination is shown to produce more of these aldehydes than chloramination alone. Najm and Krasner (1995) report that the formation of ketoacids is proportional to the amount of dissolved organic carbon (DOC) in the water. Ketoacid concentrations are largely unaffected by bromide concentration.

Ammonia addition has been used to limit the formation of some ozonation byproducts. In one study (Siddiqui and Amy 1993), bromoform concentrations decrease by approximately 30 percent when ammonia is added at a NH_3-to-ozone ratio of 0.25 mg/mg. The reason for this reduction is because HOBr reacts with ammonia to form bromamines, presumably making HOBr unavailable for reaction with NOM.

Conflicting results of ammonia addition on bromate formation have been observed (Glaze et al. 1993, Krasner et al. 1993). Siddiqui et al. (1995) explained the percentage of bromate reduction upon

adding ammonia is more dependent upon pH and bromide concentration than on ammonia concentration (Siddiqui et al. 1995). High bromide levels trap more oxidizing equivalents to give higher bromine yields and scavenge more radicals, thus reducing the radical processes that may cause bromate formation. Siddiqui et al. (1995) demonstrated that (at similar ammonia concentrations) bromate formation decreased by more than 80 percent upon increasing the bromide concentration from 0.1 to 1.0 mg/L.

2.2.4.3 Factors Affecting Performance

Ozone decays rapidly at high pH and warm temperatures. Krasner et al. (1993) noted that as the ozonation pH decreases, the required dose to meet CT requirements of the IESWTR drops and less bromate is formed. For one of the waters evaluated during bromide spiking experiments, bromate concentrations ranged from 24 to 68 µg/L at pH 8. For the same water, bromate concentrations ranged from less than 5 to 7 µg/L when the pH was decreased to 6. Better disinfection is expected at pH levels between 6 and 8 where molecular ozone dominates.

Temperature and alkalinity also affect formation of byproducts during ozonation. Increased temperature will increase the levels of bromate, bromoform, and total organic bromide. It also increases the decomposition of ozone. Conversely, increasing alkalinity has been shown to reduce the formation of bromoform and total organic bromide and increase the formation of bromate. Bicarbonate scavenges OH radicals, suggesting that the OH radical may play a role in the formation of brominated species by affecting the level of HOBr, which is presumed to be an active species for total organic bromide formation (Glaze et al. 1993).

Total organic carbon (TOC) concentration can have significant impacts on *Cryptosporidium* CT requirements. It has been demonstrated that ozone-to-TOC ratios greater than 1 are required for *Cryptosporidium* inactivation; whereas ozone-to-TOC ratios are typically less than 0.5 for *Giardia* inactivation. As previously discussed, temperature can also drastically affect the solubility, decomposition rate and biocidal effectiveness of ozone. Exhibit 2.9 presents CT requirements for *Cryptosporidium* inactivation at multiple temperatures and for inactivation ranging from 0.5 to 3 log. Exhibit 2.9 also compares the *Cryptosporidium* CT requirements with those of *Giardia* and presents the ratio of the *Cryptosporidium* requirement to the *Giardia* requirement.

Exhibit 2.9: CT Considerations for *Cryptosporidium* Inactivation

Log Inactivation	*Crypto* CT at Temperature (C)[1]			*Giardia* CT at Temperature (C)[2]			Multiplier at Temperature (C)[3]		
	1°	13°	22°	1°	13°	22°	1°	13°	22°
0.5	6.00	2.00	0.60	0.48	0.19	0.10	12.5	10.6	5.8
1.0	12.00	4.00	1.50	0.97	0.38	0.21	12.4	10.4	7.2
1.5	24.00	8.00	3.00	1.50	0.58	0.31	16.0	13.9	9.6
2.0	40.00	11.00	4.40	1.90	0.76	0.42	21.1	14.5	10.6
2.5	45.00	15.00	6.00	2.40	0.95	0.52	18.8	15.7	11.5
3.0	62.00	22.00	8.00	2.90	1.14	0.62	21.4	19.3	12.8

[1] Values reported to be acceptable for a pH range of 6 to 9.
[2] *Giardia* CT required numbers are based upon the CT table included in the SWTR Guidance Manual.
[3] Multiplier = *Crypto* CT at a given temperature / *Giardia* CT at the same temperature.

Source: Summary from Finch 1999.

2.2.5 Microfiltration and Ultrafiltration

Membranes act as selective barriers, allowing some constituents to pass through while blocking the passage of others. The movement of these constituents across a membrane requires a driving force (i.e., to overcome the potential difference across the membrane). Only pressure-driven processes are discussed in this document due to their feasibility for DBP precursor and microbial control and their popularity in the drinking water field.

There are four categories of pressure-driven membrane processes: microfiltration, ultrafiltration (UF), nanofiltration, and reverse osmosis (RO). Low-pressure membrane processes, MF and UF, are typically applied for the removal of particulate and microbial contaminants and can be operated under positive or negative (i.e., vacuum) pressure. Positive pressure systems typically operate between 3 and 40 pounds per square inch (psi), whereas vacuum systems operate between -3 and -12 psi. RO and NF are typically applied for the removal of dissolved contaminants, including both inorganic and organic compounds. These processes operate at pressures significantly greater than the applied pressure in MF and UF processes, between 100 and 150 psi. Desalination applications can operate at pressures as high as 1,200 to 1,500 psi.

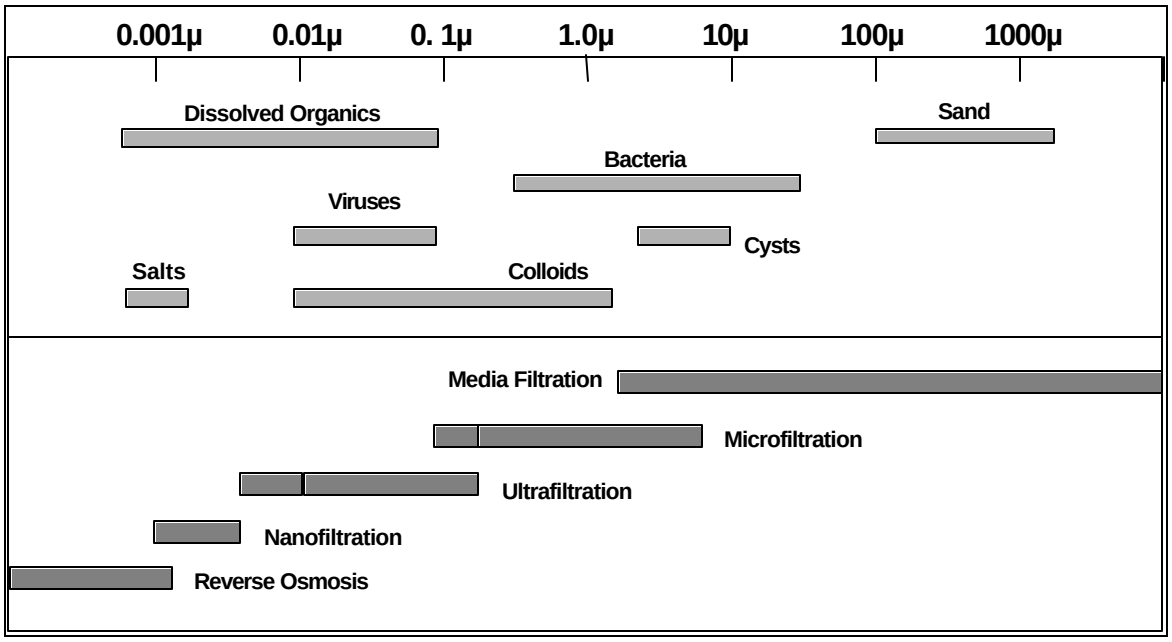
The ability of a membrane to remove a particular contaminant is influenced by its molecular weight cut-off (MWCO) or pore size. MWCO is a manufacturer specification that refers to the molecular mass of a macrosolute (e.g., glycol or protein) for which a membrane has a retention capacity greater than 90 percent. The pore size refers to the diameter of the micropores on the membrane surface. The true pore size is difficult to measure, and, as a result, membrane manufacturers typically use some measure of performance to categorize the pore size of a membrane. The nominal pore size is typically based upon a given percent removal of a marker (e.g., microsphere) of a known diameter. The absolute pore size is typically characterized as the minimum diameter above which 100 percent of a

marker of a specific size is removed by the membrane. Exhibit 2.10 presents the MWCO/pore size ranges for membrane processes, as well as the relative size of common drinking water contaminants.

MF and UF are primarily used for particle and microbial removal, either following granular media filtration or as a replacement for media filters. Chemical disinfection may be required, depending upon the approach of the State regulatory agency and the class of membrane used (i.e., MF or UF). MF pore sizes are generally too large for virus removal and many States require a minimum 0.5 log chemical inactivation as part of a multiple barrier approach to disinfection.

The major components of a typical MF or UF membrane system include cartridge filters, low pressure feed pumps, membrane modules, high-pressure backwash pumps, a chemical cleaning system, a chlorination feed system, and a concentrate handling and disposal system.

Exhibit 2.10: Pressure-Driven Membrane Separation Spectrum



Note: µ = Microns.

2.2.5.1 Efficacy Against Pathogens

MF and UF have shown excellent capabilities in turbidity, particulate matter, and microbial removal. MF and UF processes remove contaminants through physical straining of the feed water as it passes through the membrane. In this respect, microbial contaminants that are larger than a given membrane pore will be retained and prevented from entering the treated water. Since the size and

shape of microorganisms varies among species and since the size and shape of membrane pores varies among membrane types, the removal of a particular microorganism by MF and UF may vary. Many States have adopted disinfection log removal credits for MF and UF processes. States grant removal credits on a case-by-case basis for up to 3 log *Giardia* removal and 4 log virus removal. However, virus removal credits are typically 0.5 log or less due to the smaller size of viruses relative to MF/UF pores.

MF and UF offer disinfection capabilities that are much improved over conventional media filtration. Exhibits 2.11 through 2.14 summarize observed removals of bacteria, *Giardia*, *Cryptosporidium*, and viruses, respectively.

Exhibit 2.11: MF and UF Studies Documenting Bacteria Removal

Reference	Process	Membrane Pore Size	Bacteria Type	Log Removal
Hofmann et al. (1998)	MF	150,000 to 200,000 Daltons	HPC, coliforms, thermotolerant coliforms, SSRC	2.5 to 3.5
Jacangelo et al. (1997)	MF	100,000 Daltons	P. Aeruginosa	>8.7*
Jacangelo et al. (1997)	MF	0.2 µm	P. Aeruginosa	>8.2*
Jacangelo et al. (1997)	MF	0.2 µm	Coliforms	>1.8*
Jacangelo et al. (1997)	MF	0.2 µm	E. Coli	>7.8*
Jacangelo et al. (1997)	MF	0.2 µm	HPC	>1.8*
Clair et al. (1997)	MF	0.2 µm	HPC	2.4
Clair et al. (1997)	MF	0.2 µm	Total Coliforms	>3
Glucina et al. (1997)	MF	0.2 µm	HPC and total Coliforms	>3
Glucina et al. (1997)	UF	100,000 Daltons	Total Coliforms	>3
Jacangelo et al. (1997)	UF	100,000 Daltons	Coliforms	>2.1*
Jacangelo et al. (1997)	UF	100,000 Daltons	E. Coli	>7.8*
Luitweiler (1991)	MF	--	HPC	1.7
Jacangelo et al. (1991)	UF	--	Total Coliforms	>3
Heneghan and Clark (1991)	UF	--	HPC	>3.4
Jacangelo et al. (1989a)	UF	--	HPC	2.8

Note: *Indicates removal to detection limit.

-- Data not available.

Exhibit 2.12: MF and UF Studies Documenting *Giardia* Removal

Reference	Process	Membrane Pore Size	Log Removal
Scheider et al. (1999)	MF	0.2 µm	>4.8
Scheider et al. (1999)	MF	0.1 µm	>4.8*
Scheider et al. (1999)	MF	0.1 µm	>4.8*
Trussel et al. (1998)	MF	0.2 µm	>5.1*
Jacangelo et al. (1997)	MF	0.2 µm	>5.2*
Jacangelo et al. (1997)	MF	0.2 µm	>6.8*
Hagen (1998)	UF	100,000 Daltons	>8*
Trussel et al. (1998)	UF	100,000 Daltons	>5.1*
Jacangelo et al. (1997)	UF	100,000 Daltons	>5.2*
Jacangelo et al. (1997)	UF	100,000 Daltons	>6.8*
Jacangelo et al. (1991)	UF	–	>4*
Jacangelo et al. (1989a)	UF	100,000 Daltons	>5*

Note: *Indicates removal to detection limit.
--Data not available.

Exhibit 2.13: MF and UF Studies Documenting *Cryptosporidium* Removal

Reference	Process	Membrane Pore Size	Log Removal
Scheider et al. (1999)	MF	0.2 µm	4.2
Scheider et al. (1999)	MF	0.1 µm	>4.2
Scheider et al. (1999)	MF	0.1 µm	>4.2
Trussel et al. (1998)	MF	0.2 µm	>4.7*
Jacangelo et al. (1997)	MF	0.2 µm	>4.9*
Jacangelo et al. (1997)	MF	0.2 µm	>6.4*
Trussel et al. (1998)	UF	100,000 Daltons	>5.1*
Hagen (1998)	UF	100,000 Daltons	>8*
Jacangelo et al. (1997)	UF	100,000 Daltons	>4.9*
Jacangelo et al. (1997)	UF	100,000 Daltons	>6.4*
Jacangelo et al. (1989a)	UF	100,000 Daltons	>5*
Jacangelo et al. (1997)	UF	100,000 Daltons	>6.4*
Jacangelo et al. (1997)	UF	100,000 Daltons	>6.4*

Note: *Indicates removal to detection limit.

Exhibit 2.14: MF and UF Studies Documenting Virus Removal

Reference	Process	Membrane Pore Size	Log Removal
Scheider et al. (1999)	MF	0.2 µm	0.5
Scheider et al. (1999)	MF	0.1 µm	1.1
Scheider et al. (1999)	MF	0.1 µm	2.3
Trussel et al. (1998)	MF	0.2 µm	0.4 to 3.1
Jacangelo et al. (1997)	MF	0.2 µm	>1
Jacangelo et al. (1997)	MF	0.2 µm	>1.5
Kruithof et al. (1997)	MF	--	0.7 to 2.3
Trussel et al. (1998)	UF	100,000 Daltons	>6.9*
Jacangelo et al. (1997)	UF	100,000 Daltons	>6
Kruithof et al. (1997)	UF	--	>5.4
Jacangelo et al. (1989a)	UF	100,000 Daltons	>8*
Jacangelo et al. (1989a)	UF	--	>6

Note: *Indicates removal to detection limit.

-- Data not available.

As shown in Exhibits 2.11 through 2.14, both MF and UF systems are capable of significant log removal of bacteria, *Giardia cysts,* and *Cryptosporidium* oocysts. The data presented indicate that MF/UF are capable of bacteria removals of nearly 9 log and *Giardia* and *Cryptosporidium* removals in excess of 8 log. In fact, in nearly all cases, the log removal demonstrated is simply a function of the influent microbe concentration, since bacteria and cysts are typically removed to detection limits. As shown in Exhibit 2.14, however, MF and UF are differentiated by virus removal. The maximum virus removal reported for MF membranes is approximately 3 log, but the average reported removal is nearer to 1 log. UF membranes typically remove viruses to detection limits.

Note that the studies summarized in Exhibits 2.11 through 2.14 are conducted with intact membranes (i.e., the membranes are not compromised). Had a fiber from one of these membranes been broken, either deliberately or accidentally, the results could be significantly different, since the potential would exist for microorganisms to pass into the treated water. For this reason, it is important to include membrane integrity testing when assessing the ability of a membrane to act as a barrier against microorganisms. Many types of membrane integrity tests exist. These tests fall into two categories: 1) direct methods and 2) indirect methods. Indirect methods include monitoring the treated water for parameters such as particle counts or turbidity. Direct methods include tests, such as air pressure decay and diffusive airflow, that directly assess the integrity of the membrane itself. Integrity testing represents an important aspect of a membrane system from a regulatory perspective, since it provides a measurement of the integrity of the filter. Commercial manufacturers have recognized this, and most systems are now provided with automatic integrity testing that can be conducted frequently (e.g., hourly).

2.2.5.2 DBP Formation

Disinfection by MF/UF is achieved through physical removal. Because of this, no DBPs are formed during disinfection by MF/UF. Chlorine or chloramines must be added subsequent to MF/UF to maintain a disinfectant residual. Chlorination and chloramination can produce DBPs as discussed in section 2.1.2.

MF and UF alone are generally not effective for DBP precursor removal. The pore sizes are typically large enough to allow most NOM to pass through these membranes, thus removing little NOM. Some tight UF membranes with MWCOs on the order of 10,000 Daltons may be capable of removing some NOM, but significant NOM removal cannot be achieved by MF or UF alone. MF/UF systems may be combined with other processes to aid in removing DBP precursors. By associating the NOM with a filterable particulate matter (e.g., powder activated carbon (PAC) or coagulant floc), the membranes can, in effect, reject some NOM. Adsorption of organics onto PAC depends on the type and dose of PAC, the contact time available, and the type of NOM. Similarly, the efficiency of incorporating NOM into coagulant flocs depends on the type and dose of coagulant, the operating conditions, and the type of NOM.

2.2.5.3 Factors Affecting Performance

Membrane pore size greatly affects microorganism removal. To illustrate this, Exhibit 2.10 shows the size of several microbes of concern against different membrane filtration options. As shown in Exhibit 2.10, cysts (including *Giardia* and *Cryptosporidium*) are larger than essentially all MF and UF pore sizes. Consequently, these processes are capable of large log removal of cysts. On the other hand, as shown in Exhibit 2.10, viruses are larger than most UF pore sizes, but smaller than most MF pore sizes. For this reason, UF is capable of large virus removal, but MF typically is not.

Membrane pores are typically a distribution of sizes (Mallevialle et al. 1996), only as accurate as the manufacturing process allows. At the present time, no precise techniques for membrane pore size determination are available. For these reasons, a membrane of a given MWCO may have pores that are larger and smaller than the given MWCO. Imperfections in the membrane module or membrane system may result in passage of microorganisms into the treated water.

Imperfections can arise through manufacturing imprecision, allowing microbes to penetrate o-rings, end seals, or spacers. Conversely, microbial contaminant removal may be increased by the cake layer, which forms on the membrane surface during a filtration cycle. This cake layer consists of contaminants rejected by the membrane, including particles, organic matter, and microorganisms. As this layer builds, it can aid filtration of suspended particulates, such as microorganisms, as water passes across the membrane. In this way, microorganisms that might normally pass through a membrane pore can be filtered from the feed water stream.

One of the critical design parameters for a membrane process is flux, which is typically expressed in gallons of filtrate per day per square foot of membrane area (gfd). The design flux determines the membrane area required for a specific plant capacity. Thus, flux has a significant impact on capital cost and results in a competitive motivation for design engineers to use a higher membrane flux, thereby reducing the area requirements. Although increasing the membrane flux can reduce the capital cost, it will increase operational costs due to higher operating pressure, more frequent chemical cleaning, and a potential increase in membrane replacement costs.

Another important design parameter is recovery, the ratio of feed water to product water. Recovery for MF and UF systems is typically 85 to 97 percent, and a function of the backwash method and frequency. Recovery can play a significant role in the design of membrane facilities, particularly in water-scarce regions.

Feed water quality can also have a significant impact on membrane system design, operation, and performance. Suspended solids and other contaminants (e.g., iron, calcium, barium, or silica) can result in more rapid fouling of the membrane, decreases in flux, and increases in transmembrane pressure (TMP). TMP is the pressure applied to drive water through the membrane. As a result, most membrane systems include some level of pretreatment to reduce the concentration of these foulants, with the level of pretreatment dependent upon raw water quality.

2.2.6 Bag and Cartridge Filtration

Like MF and UF, bag and cartridge filters act as selective barriers and are used to remove particles, including pathogens, in water treatment. As water passes through the bag or porous cartridge, particulate matter and organisms whose size exceeds the largest pore size are retained on the filter. The nature of the filter material and the direction of flow are two features that differentiate bag from cartridge filtration (AWWA 1999).

Bag filters can be either woven or felt and made of materials such as polypropylene, polyester, nylon, or teflon. Typically, only felt filters will display nominal pore size ratings as low as 0.5 to 1 μm, which are values likely to be associated with high removal of pathogens. Bag filters can also comprise a sealing system on their open end in order to ensure flow integrity between the water inlet and the bag filter.

The bag is housed in a pressure vessel and supported by a mesh basket. The pressure vessel is made of carbon steel or stainless steel. The water flow is from inside the bag filter to outside. As filtered material (i.e., suspended solids) accumulates on the filter surface, head loss increases, and a pressure differential develops between both sides of the filter.

A number of bag filter configurations are commercially available. Pressure vessels exist in single, duplex or four-plex, series or parallel modules, or as multi-filter vessels. Manufacturers claim that a single vessel can filter flow rates from 10 to approximately 2,000 gallons per minute (gpm), depending on its configuration. The standard pressure-rating for vessels has been observed to be 150 psi.

Cartridge filters are typically composed either of fiberglass or ceramic membranes supported by a rigid core or are made from strings of polypropylene, acrylics, nylon, or cotton wrapped around a filter element. Nominal pore size ratings generally range from 0.3 to 200 microns. With regard to membranes, the number of pleats in a cartridge filter is typically larger relative to a bag filter, thus providing greater surface area. The cartridge is housed in a pressure vessel made of carbon steel or stainless steel, similar to the bag filter, but the direction of the flow is from the outside to the inside of the cartridge. Accumulation of particulate matter on the surface and in the depth of the cartridge element leads to increased pressure loss across the cartridge. Operation of the cartridge filter beyond the recommended maximum pressure drop would result in the structural failure of the cartridge and potential damage to the cartridge filter vessel.

Commercially available cartridge filter single vessels allow for housing of 1 to approximately 200 cartridges. It is possible to connect these vessels in series (for multiple-stage filtration) or parallel (for treatment capacity increase and/or continuous operation).

2.2.6.1 Efficacy Against Pathogens

Because their mode of operation is based on a size-exclusion mechanism, bag and cartridge filters with the proper pore size rating can remove *Cryptosporidium*, *Giardia*, and other pathogens, depending on their size. Available studies assessing the efficacy of bag and cartridge filters against pathogens have frequently utilized polystyrene beads as surrogates for the *Cryptosporidium* oocysts and *Giardia* cysts (Li et al. 1997, Goodrich et al. 1995, Long 1983). Cysts and oocysts are suspected to fold and deform, eventually passing through filtration pores that are smaller than their nominal diameter. In an effort to account for this flexibility, investigators have used polystyrene beads smaller than the pathogens they represent.

In a study by Li et al. (1997), log removals of *Cryptosporidium* oocysts and 4-6 µm polystyrene microspheres by bag filters were determined and compared. The investigators concluded a linear correlation: 1 log removal of 4-6 µm polystyrene microspheres is equivalent to 1.040 log removal of *Cryptosporidium*. This is attributed to similar size distributions.

The EPA Risk Reduction Engineering Laboratory assessed the ability of bag filtration to remove *Cryptosporidium* and surrogates under various flow (12.5 and 25 gpm) and pressure drop (0, 7, 15, and 25 psi) conditions (Li et al. 1997). The study evaluated three polypropylene bag filters. The surrogates tested were turbidity, 1-25 µm particle counts, 4-6 µm particle counts, and 4-6 µm

polystyrene microspheres. The study found the polystyrene microspheres to be "accurate and precise" indicators of filter performance with respect to *Cryptosporidium*. The results of this study are summarized in Exhibit 2.15.

Exhibit 2.15: Summary of Bag Filter Performance

Filter Type	Nominal Pore Size	Contaminant	Log Removal (Average)
Multi-layer polypropylene	1-µm	4.5-µm microspheres	1.14 - 1.88 (1.39)
		Cryptosporidium	1.35 - 1.48 (1.41)
Single-layer polypropylene	1-µm	4.5-µm microspheres	0.14 - 0.72 (0.46)
		Cryptosporidium	0.26 - 0.64 (0.42)
Multi-layer polypropylene	99% removal of 2.5 µm particles, 95% removal of 1.5 µm particles	4.5-µm microspheres	0.93 - 3.42 (2.08)
		Cryptosporidium	3.00 - 3.63 (3.29)

Source: Li et al. (1997).

The results presented in Exhibit 2.15 may indicate a benefit in removal efficiency associated with multi-layering of the filter fabric. Based on this study, a multi-layer fabric bag filter can achieve 1 to 2 log *Cryptosporidium* removal under proper operation conditions. One interesting result of these tests is that experimental controls performed with *Cryptosporidium* showed that 0.1 to 0.2 log removal can be attributed to the pressure vessels alone without bag filters. This is assumed to reflect the ability of *Cryptosporidium* oocysts to adhere to the surface walls of the vessel.

Another study by the Risk Reduction Engineering Laboratory (Goodrich et al. 1995) evaluated cartridge filters for the removal of 4-6 µm polystyrene spheres. The results of this study indicate that a single cartridge filter, with a 2 µm rating, achieved an average microsphere removal of 3.6 log.

A study conducted by Long (1983) evaluated the log removal of 17 different cartridge filters for *Giardia* surrogates. These cartridge filters were tested using the same pressure vessel at a pressure of 45 psi and a flow rate of 0.5 gpm. The microspheres used as surrogates for *Giardia* cysts had an average diameter of 5.7 µm, with a standard deviation of 1.5 µm. The filters were made of a variety of materials (cotton, cellulose, glass fiber, polypropylene, polyester) and configurations (majority pleated or spirally wound). The pore ratings ranged from 0.2 to 10.0 µm.

According to a scanning electron microscopy analysis that allowed visual counting of the microspheres passing through the filter, ten cartridge filters out of seventeen had a microsphere removal of 99.99 percent (4 log reduction). The lower performances seemed to be associated with the absence of end seals on the cartridges and the use of cotton or polyester as the main filtering material (Long

1983). Note that the tests were conducted at a flow rate of 0.5 gpm, which is significantly lower than the expected operation flow rate (typically 20 gpm per unit). The impact of this reduced flow rate on removal performance is unclear.

2.2.6.2 Factors Affecting Performance

Feed water quality is the primary factor affecting the performance of both bag and cartridge filters. Although these filters can operate at turbidity levels from 0.1 to 10 NTU, it is recommended that turbidity be minimized to extend the filter lifetime. If turbidity of the feed water is above 1 NTU, bag filters may operate properly for only a few hours (USEPA 1998). Thus, use as a secondary barrier following conventional treatment is a preferred mode of operation. Granular media filters can reduce feed water turbidity to less than 0.1 NTU and provide a feed water stream of appropriate quality for bag and cartridge filters.

Feed water should also contain very low levels of sand, silt, or algae to prevent clogging of the filters. If raw water quality is such that the concentrations of these parameters are high, pretreatment, such as sand, multimedia filters, or preliminary bag or cartridge filters with larger pore size (e.g., 10 μm), is encouraged.

The appropriate choice of the pore size rating is an important issue. *Giardia* cysts and *Cryptosporidium* oocysts are suspected to deform and fold, enabling them to pass through pores that are nominally smaller than the pathogen. The selected pore size should be sufficient to achieve significant removal of microorganisms while maximizing the expected filter lifetime, based upon raw water quality and filter loading. Likewise, the quality of the system's seals will greatly impact the level of performance. The most critical seals appear to be between the filter and the pressure vessel and within the structure of the filter itself. A faulty seal is a way for pathogens to partially or completely bypass filtration.

Pilot testing (i.e., challenge studies) is frequently recommended to assess the performance of bag and cartridge filters. However, the costs associated with pilot testing, particularly for small systems, can represent a significant portion of the installation costs. As a result, pilot testing may not be affordable for small systems and may limit the use of these technologies where pilot testing is necessary. Some States (e.g., Oregon) accept manufacturer data regarding removal efficiency and permit systems to operate in a demonstration mode, with additional monitoring requirements.

The skill level required to operate bag or cartridge filters is typically described as basic (AWWA 1999, Campbell et al. 1995a). Turbidity, head loss, and total number of gallons filtered should be monitored daily to evaluate the need to replace the bag or cartridge (AWWA 1999). For example, cartridges are generally replaced when the pressure differential reaches 35 psi, after one to six months of operation (Malcolm Pirnie 1993). The maximum allowable pressure differential is typically recommended by the manufacturer.

Cartridges and bags are easily damaged at the time of installation. Bags should be replaced with caution to prevent tearing of the material. Likewise, the operator should carefully install new cartridges, as the filter seal can be damaged and induce leakage.

Because of their rigid structure and multi-layer design, cartridge filters are generally more sturdy and offer more operational flexibility than bag filters. However, this higher performance is typically associated with higher cost. As mentioned previously, cysts and oocysts can adhere to and accumulate on the surface walls of the system. As a consequence, the inward flow of water in the cartridge filter requires that the housing be cleaned entirely when replacing the cartridge, which is not the case with bag filters.

2.2.7 Bank Filtration

Bank filtration is a water treatment process that uses a river bed or the bank of a river or lake as a natural filter. Water from a river or stream flows through the bank and draws from one or more wells. Microorganisms and other particles are removed by contact with the aquifer materials as the water travels through the subsurface, either horizontally or vertically. High removal occurs when ground water velocity is slow and the aquifer consists of granular materials with open pore space, allowing water flow around the grains. In these granular porous aquifers, the flow path is very tortuous, thereby providing ample opportunity for the microorganism to contact and attach to a grain surface. Although detachment from the grains can occur, it typically occurs at a very slow rate. When ground water velocity is exceptionally slow, or when little or no detachment occurs, most microorganisms become inactivated before they can enter a well. Thus, bank filtration provides physical removal and, in some cases, inactivation to protect wells from pathogen contamination.

2.2.7.1 Efficacy Against Pathogens

Due to the low recovery rate of *Cryptosporidium* oocysts in influent and effluent samples, full scale treatment data are of limited utility for assessing removal of *Cryptosporidium* via bank filtration. However, measurement of other parameters indicate the potential for pathogen removal. Exhibit 2.16 summarizes bank filtration studies that measured coliform and spore removal. *Cryptosporidium* removal is site-specific and highly dependent on the aquifer characteristics; therefore, these data are only an indication of contaminant removal that can be achieved by bank filtration.

Exhibit 2.16: Bank Filtration Studies Measuring Coliform and Spore Removal

			Log Removal		
Reference	**Travel Distance (m)**	**Travel Time (days)**	**Total Coliform**	**Thermotolerant Coliform**	**Spores[1]**
Havelaar et al. (1995)	30	15	≥ 5.0	≥ 4.1	≥ 3.1
Havelaar et al. (1995)	25	63	≥ 5.0	≥ 4.1	≥ 3.6
Medema et al. (2000)	13	7	N/A	4.1	3.3
	25	18	N/A	4.5	3.9
	150	43	N/A	6.2	5.0
Wang et al. (2000)	0.6	N/A	N/A	N/A	2.0
	1.6				2.0
	3.0				2.0
	16				3.0

[1] Spore data are sulphite-reducing clostridium for all references except Wang et al. (2000), where spore data are aerobic endospores.

2.2.7.2 Factors Affecting Performance

The main factor affecting the performance of bank filtration is the type of aquifer material through which the water is filtered. Granular media is the most effective, while fractured rock or gravel with large pore sizes may be the least effective and allow *Cryptosporidium* to pass through without contacting a grain surface. The flow rate is also an important factor in determining performance. Too high a flow rate can cause oocysts to detach from the aquifer material. Low flow rates, however, may make it difficult to meet volume demands.

2.2.8 Second Stage Filtration

Second stage, or secondary, filtration requires the use of rapid sand, dual media, GAC, or other fine grain media in a separate stage following rapid sand or dual media filtration. A cap, such as GAC, on a single stage of filtration is not considered second stage filtration.

Filtration processes are standard in the water treatment process, and much design and operational information is available. However, the use of a second filtration stage is not as common, and little information is available.

2.2.8.1 Efficacy Against Pathogens

There is relatively little published data on the removal of *Cryptosporidium* by second stage filtration. Results based on a number of single stage filtration studies demonstrate that rapid sand filtration, when preceded by coagulation, can achieve significant removal of *Cryptosporidium*. While these studies evaluated only a single stage of filtration, the same mechanisms of removal would occur with a second filtration stage. Studies have also shown that *Cryptosporidium* breakthrough occurs after the first stage of filtration; therefore, a second stage of filtration is likely to provide a barrier to these oocysts.

Many studies (Dugan et al. 2001 and Emelko et al. 1999) have demonstrated that aerobic spores are a conservative indicator of *Cryptosporidium* removal by granular media filtration when preceded by coagulation. Consequently, EPA believes that data on spore removal by a second stage filtration process are indicative of the capacity of this process to remove *Cryptosporidium*.

Between 1999 and 2000, the Cincinnati Water Works collected spore and turbidity removal data from their GAC system. The specifics of their system are provided below.

- 11-foot deep GAC filter following dual media filter
- Loading Rate = 3.4 - 3.9 gpm/ft^2 (average); 7.1 gpm/ft^2 (design)
- 12*40 mesh
- d_{10} = 0.5 - 0.75 millimeters (mm); d10 is the diameter through which 10 percent of the media will pass
- Uniformity Coefficient (UC) $\leq$ 2; UC is the uniformity coefficient of the media
- Media age -- new to 7 years old; carbon reactivation two times per year
- Empty Bed Contact Time (EBCT) = 22 minutes at 120 million gallons per day (mgd) (average flow); 12 minutes at 220 mgd (design flow)

A median incremental spore removal of 0.92 log was observed in their GAC filter. Additionally, the secondary GAC filters were observed to dampen or eliminate turbidity spikes from preceding dual media filters that occurred during ripening, breakthrough, etc. These data indicate that 0.5 log or greater removal of *Cryptosporidium* can be achieved by a secondary filtration process like GAC contractors.

Based on information presented by Hall et al. (1994), up to a 50 percent improvement in turbidity removal was observed when using a second stage filter. However, no improvement in *Cryptosporidium* removal was observed due to the second stage filter. This information was collected after spiking 500 oocysts/L into the raw water of a conventional filter followed by a secondary filter consisting of GAC.

2.2.8.2 Factors Affecting Performance

Filter Type

There are several types of filters. Fine sand filters, dual media filters, and multimedia filters are the main types of filters used in conventional filtration plants. In order to encourage penetration of solids into the depth of the bed, the dual media filter, consisting of a layer of coarser anthracite coal on top of a layer of finer silica sand, was developed. Studies conducted by many researchers (Conley and Pitman 1960a, Conley 1961, Tuepker and Buescher 1968) showed the benefits of dual media filters in reducing the rate of head loss development, which lengthened the filter run. Although dual media is presumed to improve the quality of the filtrate, this benefit has not been well demonstrated (Water Quality and Treatment 1999). Research conducted by Robeck, Dostal, and Woodward (1964) demonstrated that the head losses in dual media filters were lower than the head losses in traditional fine sand filters. When a typical dual media filter and a fine sand filter are operated at the same filtration rate on the same influent water, the head loss development rate for the typical dual media filter should be about half the rate of the fine sand filter (Water Quality and Treatment 1999). Multimedia filters add a layer of garnet to the media which allows for a finer layer of media at the bottom of the filter.

Filter Media

As with all filters (first or second stage), various properties of a filter medium, such as size, shape, density, and hardness, affect filtration performance. Filter media are defined by their uniformity coefficient (UC) and effective size (ES). The porosity of the filter bed formed by the grains is also important (Water Quality and Treatment 1999). Filter media should be coarse enough to retain large quantities of floc, yet fine enough to prevent passage of suspended solids. The filter bed should also be deep enough to allow long filter runs and graded to permit backwash cleaning. In order to obtain high rates of filtration, coarse sands and dual media beds of anthracite overlying sand have been used in the recent past (Water Supply and Pollution Control 1993).

The bed porosity and the ratio of the bed depth to media grain diameter affect the filter efficiency. The larger the depth of the filter bed (L), the more opportunities exist for particle capture; the larger the average diameter of the media (d), the more of the media is available to capture particles over the depth of the filter bed. The two most commonly used methods in selecting the optimal filter bed depth and media size are pilot plant studies and existing data from filtration facilities treating similar waters.

Filter Hydraulics

Hydraulic surges occur when the flow through a filter changes rapidly (e.g., during either filter backwashing or servicing). Hydraulic shifts can lead to significant particle detachment. To ensure that the second stage filtration unit is unaffected by any hydraulic surges caused by the backwashing of the first stage filtration unit, the first stage filters should be hydraulically isolated during backwashing and servicing.

2.2.9 Pre-Sedimentation

Pre-sedimentation is a preliminary treatment process used to remove particulate material from the source water before the water enters the main treatment plant. Because pre-sedimentation reduces particle concentrations, it is also expected to reduce *Cryptosporidium* levels. In addition, by reducing variability in water quality of the source water, pre-sedimentation may improve the performance of subsequent processes in the treatment plant. To remove pathogens through floculation and sedimentation, it is necessary to add coagulant.

Sedimentation processes are standard in the water treatment process, and much design and operational information is available. However, the use of a pre-sedimentation basin is not as common, and little information is available.

2.2.9.1 Efficacy Against Pathogens

There is relatively little published data on the removal of *Cryptosporidium* by pre-sedimentation. Consequently, EPA analyzed studies that investigated *Cryptosporidium* removal by conventional sedimentation basins. The removal efficiency in conventional sedimentation basins may be greater than in pre-sedimentation due to differences in surface loading rates, coagulant doses, and other factors. To supplement these studies, EPA reviewed data provided by utilities on removal of other types of particles, primarily aerobic spores, in the pre-sedimentation processes of full-scale plants. Studies have shown that, in the presence of a coagulant, spore removal is a conservative indicator of *Cryptosporidium* removal (Dugan et al. 2001).

The literature studies reviewed by EPA show *Cryptosporidium* log removals of 0.6 to 3.8 (Dugan et al. 2001, Payment and Franco 1993) and mean *Bacillus subtilis* and total aerobic spores log removals of 0.6 to 1.1 (data collected independently by the Cincinnati, OH, and St. Louis, MO, water utilities) by sedimentation processes. The removal of aerobic spores through sedimentation basins in full-scale plants demonstrate that pre-sedimentation is likely to achieve mean reductions of greater than 0.5 log *Cryptosporidium* removal under routine operating conditions and over an extended time period.

2.2.9.2 Factors Affecting Performance

Short Circuiting

Short circuiting in the sedimentation tank occurs when a portion of the influent flow reaches the outlet of the sedimentation basin much faster than the designed detention time of the basin. Short circuiting increases the operational surface loading rate since the true settling area available for a portion of the flow is reduced. If short circuiting causes the basin to operate at an effective loading rate greater than 1.6 gpm/ft^2, the basin would not receive *Cryptosporidium* removal credit. High wind velocities and density and temperature differentials between the influent water and the water in the sedimentation basin cause short circuiting. Additionally, the design or configuration of both the inlet and outlet are important factors that can affect short-circuiting and turbulence. Systems can minimize short circuiting by adding baffles or making other modifications to the flow pattern.

Coagulant Dose

The principle goal of coagulation is to destabilize the particles so that they can be more easily aggregated into flocs. The commonly used coagulants are alum, ferric chloride, polyaluminum chloride (PACl), activated charcoal, and activated silica. The coagulant dose required to treat an influent stream depends on the chemical composition of the influent, the characteristics of the colloids and suspended matter in the influent, the water temperature, and mixing conditions. The use of a coagulant improves the pathogen removal capabilities of the pre-sedimentation process, although some pathogen removal is expected without coagulant addition. Optimizing a coagulation scheme for a two-stage sedimentation process is site-specific and not simple. It is therefore not possible to prescribe the type of coagulant and appropriate dose for an aggregate of source waters. To account for an additional sedimentation process, the standard jar test can be modified to a two-stage process reflecting the two stages of sedimentation.

2.2.10 Watershed Control

A well-designed watershed control program can reduce overall microbial risk. The risk reduction would be associated with the implementation of practices that reduce *Cryptosporidium,* as well as other pathogens. Knowledge of the watershed and factors affecting microbial risk, including sources of pathogens, fate and transport of pathogens, and hydrology can also help a system reduce microbial risk.

2.2.10.1 Efficacy Against Pathogens

No data are available on the ability of watershed control programs to reduce *Cryptosporidium* loading to surface water. This is partly because, until recently, most watershed programs have focused

on improving water quality for recreational and ecological uses rather than for drinking water protection. Thus, studies of the success of such programs frequently monitor parameters such as phosphorus and sediment levels. Watershed programs that do have drinking water protection as a goal frequently track fecal coliform bacteria levels but do not regularly monitor *Cryptosporidium*. Fecal coliform concentrations do not always correlate with *Cryptosporidium*, but better indicator data are not usually available. *E. coli* may be a better indicator of fecal contamination than fecal coliform bacteria, but monitoring for *E. coli* is not common practice.

Most water systems that do monitor *Cryptosporidium* have been doing so for only a few years and would not have enough data to show a change in water quality resulting from watershed management. In addition, because *Cryptosporidium* occurs in such low concentrations and is often undetected, reductions in microbiological contamination are difficult to demonstrate.

Regardless of the constituents monitored, it is difficult to show that a watershed control program in its entirety has improved water quality. Often, reductions in contamination from one source can be overshadowed by increases from other sources, especially in urban areas. However, various components of a watershed control program have been shown to have a positive effect on microbiological water quality at a local level, at least for fecal coliform. Combined, these components should theoretically contribute to an overall decrease in microbiological contamination.

For instance, Thurston et al. (2001) showed that a constructed wetland could reduce fecal coliform levels in wastewater treatment plant effluent by 98 percent (where effluent had previously received secondary treatment). *Cryptosporidium* reductions of 64 percent were also achieved through this study. A similar pilot-scale study with untreated wastewater indicated an overall removal of microorganisms of 90 percent by constructed wetlands (Quinonez-Diaz et al. 2001). Preliminary results of a watershed restoration program in Vermont showed that streambank stabilization, fencing of riparian zones to prevent grazing, and protected stream crossings reduced bacterial levels (Meals 2001). A fencing program in Virginia suggested some reduction in fecal coliform levels, and the proportion of fecal streptococci strains traced to livestock was reduced (Hagedorn et al. 1999).

Another way to reduce microbiological contamination of an urban watershed is to upgrade wastewater collection systems. The Fairfax County, Virginia, Wastewater Collection Division decreased inflow and infiltration into its sewers and increased the sewers' capacity through a rehabilitation and maintenance program. Between 1995 and 2001, the utility reduced the number of sanitary sewer overflows throughout the county by 67 percent and reduced the peak flow to one of its wastewater treatment plants by 35 mgd (USEPA 2001). Similar programs throughout the United States are contributing to reduced effluent volumes from sanitary sewer overflows and combined sewer overflows.

2.2.10.2 Factors Affecting Performance

A combination of interventions such as those described above is expected to result in an overall decrease of *Cryptosporidium* in source water. However, many factors can negatively affect the success of a watershed control program. The interventions a system implements depend on the types of contamination sources in the watershed. Control of point source discharge (e.g., waste water treatment plants and industrial discharges) can be straightforward. Agricultural and urban nonpoint sources are the most difficult to control. Reduction of *Cryptosporidium* from these sources generally depends on the voluntary cooperation of urban residents and farmers.

Urban watersheds are subject to increasing development, which increases surface imperviousness and the amount of runoff entering surface waters, along with the pollutant load. Acquisition of undeveloped land, particularly that closest to the source water and its tributaries, is one of the best ways to prevent degradation of the water quality, but it may not be feasible in some watersheds. Other restrictions on development, such as zoning requirements, can also control urban runoff to some extent, but, again, these may not be feasible or may not have the support of the public or other government agencies.

Another problem facing PWSs is that the watershed may extend beyond the municipal boundaries into other jurisdictions. A higher authority (e.g., State or county government) may be needed to regulate activities outside a PWS's jurisdiction that could affect water quality.

2.2.11 Combined Filter Performance

Combined filter performance reduces *Cryptosporidium* levels by enhancing filter performance to produce very low turbidity water. It is defined specifically as producing 0.15 NTU turbidity water in the combined filter effluent (CFE) 95 percent of the time. Methods that systems may use to improve filter performance and lower turbidity include adding polymer, optimizing the filtration process by adding media or installing filter-to-waste capabilities, and improving staff ability to optimize the process by additional training, hiring new operators, and buying new laboratory equipment.

Systems likely to use this technology are those which operate conventional filtration or softening plants and which are already operating well below the current turbidity limits of 0.3 NTU. These systems more than likely target their effluent under 0.15 NTU already but are not currently hitting that target more than 95 percent of the time. These plants are assumed to be able to reach the target 95 percent of the time with relatively minor adjustments to their process.

2.2.11.1 Efficacy Against Pathogens

There have been a number of studies examining the removal of pathogens by conventional filtration. Several of these studies have examined the relationship between finished water turbidity and protozoa removal. Studies by Dugan et al. (2001) and Patania et al. (1995) showed that turbidity is an adequate indicator of pathogen removal. Although the correlation between turbidity removal and pathogen removal is not one to one, removal of turbidity is a conservative indicator of pathogen removal.

Under the IESWTR and LT1ESWTR, conventional and direct filtration plants may claim 2.0 log *Cryptosporidium* removal credit if their CFE turbidity never exceeds 1 NTU and is less than or equal to 0.3 NTU in 95 percent of samples taken. Under the LT2ESWTR, systems using conventional filtration treatment or direct filtration treatment may claim an additional 0.5 log *Cryptosporidium* removal credit for any month that a plant demonstrates CFE turbidity levels less than or equal to 0.15 NTU in at least 95 percent of the measurements taken each month, based on sample measurements collected under §§141.73,141.173(a) and 141.551.

EPA expects plants that rely on complying with a 0.15 NTU standard to consistently operate below 0.1 NTU. Results from studies conducted by Patania et al. (1995), Emelko et al. (1999), and Dugan et al. (2001) show that plants consistently operating below 0.1 NTU can achieve an additional 0.5 log or greater removal of *Cryptosporidium* than when operating between 0.1 and 0.2 NTU.

2.2.11.2 Factors Affecting Performance

Many factors can affect removal of pathogens through sedimentation and filtration and hinder a plant's ability to achieve 0.15 NTU in its CFE. In order to achieve 0.15 NTU 95 percent of the time, plants will need to have tight control of their process. The areas which require specific attention include: control of coagulant dosing and mixing, control of dosing of other chemical additions, filter hydraulics and media, and backwashing procedures.

Coagulant Dose

Insufficient coagulant can lead to colloidal particles remaining in suspension, while too much coagulant can lead to inefficient settling. Therefore, coagulant must be optimized for the entire plant. It must also be adjusted as influent water quality varies or if there are other major changes in plant operation.

Filter Ripening

During the period immediately after a backwash, the lack of particles on the filter media can make capture of the particles by the media more difficult and lead to breakthrough of particles and

turbidity. Hall and Croll (1996) studied removal in a pilot plant and saw peaks in both turbidity and oocysts in the filtered water for an hour after backwashing. West et al. (1994) found that removal increased from 2 log to of 3 log once the filters had ripened, and the turbidity had dropped from an initial value of 0.2 NTU to a value less than 0.1 NTU.

Filter Breakthrough

During filter runs, particles can collect in the filter and, if not backwashed, will reach the point where an increased amount of particles pass through (referred to as breakthrough). Emelko et al. (2000) studied the performance of filters throughout a typical run cycle. They found that removal was 5.5 log when the filters were operating at 0.04 NTU. When the turbidity began to climb, removal dropped to 2.1 log even while turbidities were still less than 0.1 NTU. By the time turbidity had reached 0.3 NTU, the removal had dropped to 1.4 log.

Filtration Rate

If the filtration rate is too high, filtration effluent water quality can suffer. McTigue et al. (1998) found that removal dropped by 2 log when the filtration rate was doubled. West et al. (1994), however found no difference in removal between filtration rates of 6 and 14 gpm/ft^2.

Backwashing

The flow rate used for backwashing is important in maintaining effluent quality. Too low a rate can leave the media dirty and lead to mudballs and eventual particle breakthrough. Too high a rate can cause loss of filter media and also lengthen filter ripening times. A surface wash can also help detach particles from the media and improve backwash performance.

2.3 DBP Precursor Removal Strategies

2.3.1 Granular Activated Carbon Adsorption

Removal of undesired compounds from water supplies can be achieved through adsorption onto solids. GAC is used in water treatment to adsorb a variety of organic and inorganic compounds. Important properties of GAC that determine its effectiveness include particle size, specific surface area, pore size distribution, and chemical nature of the surface. GAC adsorption, as practiced in water treatment, is an non-steady state process, with the effluent concentration of the contaminant increasing with time. Once the effluent concentration meets the maximum allowable concentration for a contaminant, the GAC column must be taken off-line, and the GAC must be replaced with reactivated or fresh GAC. The operation time to this maximum effluent concentration is termed the reactivation interval.

The EBCT is defined as the volume of media divided by the flow rate. GAC contactors should be used when longer EBCTs are required, while sand filters with a GAC cap, where the top portion of the sand is replaced by GAC, can be used when shorter EBCTs are feasible. These GAC-capped filters are often called filter-adsorbers. Filter-adsorbers can also be filtration units which contain GAC alone. Because of their shorter EBCTs, filter-adsorbers meet desired water quality goals for a much shorter period of time than GAC contactors. For the purposed of treating seasonal changes in water quality or contaminant shock loads, filter-adsorbers may have an economic advantage over post-filter GAC contactors. One disadvantage of filter-adsorbers is that GAC losses are high during backwashing, and reactivation and equipment separating GAC from sand may be required before reactivation.

GAC contactors operate in either downflow or upflow configurations. Downflow fixed-bed contactors offer the simplest and most common contactor configuration for drinking water treatment. Upflow beds are typically used in situations where very long contact times (greater than 120 minutes) are required and/or where the level of suspended solids is high. Flow through GAC contactors can be either gravity or pressure driven.

The hydraulic constraints of a given system govern the selection between pressure or gravity contactors. Pressure contactors may be more applicable for ground water systems, since these systems already are pumping their water. Gravity contactors are generally found in surface water systems, if sufficient head is available. Downflow contactors are typically placed downstream of the plant filters to minimize the solids loading to the contactor.

The GAC in a contactor is usually replaced when the effluent concentrations exceed the treatment objective. At this point, however, only a portion of the GAC is fully utilized, and replacement of the media will result in unnecessarily high carbon usage rates. Operating multiple GAC contactors in either series or parallel configurations are the two common methods to reduce GAC usage rates.

For contactors configured in series, the GAC in the first contactor is reactivated when the effluent from it no longer meets the treatment objective. The first contactor is taken offline while the second contactor continues operation. After the GAC in the first contactor is replaced, it is brought back online downstream from the operating contactor. That is, the position of the two contactors is reversed, with what was originally the second contactor becoming the first contactor and vice versa. For efficient operation, the mass transfer zone should be contained within the bed length of one contactor. This can be achieved using reasonable bed lengths for adsorption of micropollutants, but the mass transfer zone for TOC removal and, therefore, DBP precursor removal is usually too long. The use of two contactors in series does not result in significantly longer run times over single contactor operation (USEPA 1999a).

For contactors configured in parallel, multiple GAC beds are operated with a staggered reactivation pattern. The effluent from individual contactors may contain contaminants at concentrations higher than the treatment objective, since they may be blended with effluent from other contactors with

little or no breakthrough. The combined effluent concentration, from all the GAC beds, can thus be maintained below the specified treatment objective, further reducing carbon usage rates. For DBP precursor removal, contactor effluents should be blended prior to disinfection.

The choice between a single contactor and contactors in series or parallel is site specific and depends on the type and concentration of the contaminant to be removed and its rate of adsorption. This choice also depends on the type, concentration, and adsorption rate of competing contaminants.

2.3.1.1 DBP Precursor Removal

In many circumstances, GAC is an effective process for the removal of NOM from drinking water sources. With an EBCT of 15 minutes and a reactivation interval of 180 days, GAC can remove 35 to 70 percent of the influent TOC on a running average basis. Running average TOC removals of 55 to 85 percent can be achieved with an EBCT of 30 minutes and a reactivation interval of 180 days.

2.3.1.2 Factors Affecting Performance

The removal of NOM by GAC adsorption depends on a large number of factors including the following:

- Molecular size, polarity, and concentration of NOM entering the GAC process
- Water quality characteristics such as pH and ionic strength
- GAC characteristics such as pore size distribution and surface chemistry
- Operational characteristics such as EBCT and GAC usage rate
- Treatment processes used prior to the GAC process
- Configuration of GAC contactors

This section briefly describes the impacts of these factors as seen in several GAC studies.

Constituents of NOM are adsorbed within the GAC bed in a manner proportional to their adsorption potential. Weakly adsorbing components of NOM may irreversibly preload the GAC at the downstream end of the bed and may, therefore, reduce the capacity of the bed for stronger adsorbing components at the end of the bed.

The impacts of pH on adsorption of NOM and humic extracts have been well documented in equilibrium studies using powdered activated carbon (Weber et al. 1983, Randtke and Jepsen 1982, McCreary and Snoeyink 1980, Summers 1986). All of these studies showed increased removal of TOC with decreased pH levels. Unfortunately, some of the work has been done with different initial TOC concentrations, and the increased performance attributed to low pH may be because of the lower initial TOC. A relationship between the relative adsorption capacity for TOC at the same initial TOC and pH has been established for 13 different source waters and a bituminous coal-based GAC (Hooper et al. 1996b). Within the pH range of 5 to 10, a decrease in the pH of one unit yielded a six percent increase in adsorption capacity. However, the number of continuous flow evaluations of pH impacts is limited.

The relationship between GAC pore size distribution and NOM molecular size distribution has been shown to be important (Summers and Roberts 1988, Lee et al. 1983, Semmens and Staples 1986, El-Rehaili and Weber 1987, Chadik and Amy 1987). In general, investigators have found the GAC process to favor removal of NOM molecules of low to moderate size even though the adsorption process was expected to favor removal of large molecules. This phenomenon occurs because small GAC pores physically exclude large NOM molecules from adsorbing. Thus, GAC with a greater quantity of large pores can be expected to remove more NOM than GAC with a smaller quantity of large pores.

The impacts of EBCT on GAC usage rate for NOM removal have been studied in numerous continuous flow evaluations. The trend observed in all studies is that increasing EBCT can reduce the carbon usage rate. One study (Miller and Hartman 1982) saw significant reduction in usage rates as the EBCT is increased from 2.8 to 15.2 minutes. Summers et al. (1997) evaluated EBCTs of 10 and 20 minutes for a number of water sources and concluded that EBCT had a definite effect in prolonging the bed life of a GAC contactor. However, the carbon usage rate is relatively unaffected by EBCTs at the ranges evaluated. They also noted that the balance between EBCT and the frequency of GAC replacement or reactivation is primarily a choice between larger capital investment (i.e., longer EBCTs) and greater operational complexities (i.e., more frequent reactivation). Another study indicated that GAC usage rate decreased with an increase in EBCT from 7.5 to 30 minutes. However, a further increase in EBCT from 30 to 60 minutes did not influence the GAC usage rate (McGuire et al. 1989).

GAC systems may require some kind of pretreatment to prevent clogging of the GAC bed, to minimize the organic loading on the GAC, and to improve cost effectiveness. Clogging of the GAC bed could be caused by suspended solids in the raw water or by precipitation of calcium carbonate, iron, and manganese on the GAC. Suspended solids typically cause problems in surface water systems, while carbonate scaling, iron, and manganese precipitation may occur in both surface and ground waters. When the GAC bed life is long, clogging may also be caused by biological growths. Pretreatment methods include coagulation, filtration, or softening ahead of the GAC system. Conventional coagulation, clarification, and filtration processes may be optimized for the removal of organic material to reduce natural organic loading to the GAC bed.

The impacts of coagulation on NOM adsorption have also been well documented in batch experiments studying adsorption equilibria (Weber et al. 1983, Randtke and Jepsen 1981, Lee et al. 1981, El-Rehaili and Weber 1987, Harrington and DiGiano 1989). Coagulation processes, as a pretreatment to GAC, can both reduce influent TOC concentration and decrease the influent pH to the adsorber, thus leading to improved GAC performance.

Several investigators have reported better GAC performance for TOC control after coagulation or after increasing the coagulant dose (i.e., enhanced coagulation). Hooper et al. (1996a, 1996b, 1996c) have shown that the increase in GAC run time after enhanced coagulation can be attributed to the lower pH and lower initial TOC concentration associated with the enhanced coagulated water. This improvement is most often attributed to a decrease in solubility of NOM at lower pH (Symons et al. 1998).

In most GAC applications of any significant size, multiple contactors are operated in a parallel configuration. Parallel GAC contactors are operated in a staggered mode wherein each contactor has been in operation for different lengths of time. In this mode of operation, one contactor at a time is taken off-line when the blended effluent exceeds the target effluent concentration, and a column with fresh or reactivated GAC is then placed on-line. The effluent from the contactor in operation the longest can be higher than the target breakthrough concentration, as it is blended with water from the contactors that have effluent concentrations much lower than the target concentrations. Consequently, the effluent of parallel contactors are blended prior to disinfection. Thus, parallel operation in a multiple contactor configuration will result in longer GAC bed-life and the time between reactivation will be longer. Under ideal conditions, staged blending with multiple parallel contactors leads to near steady-state effluent concentration (Roberts and Summers 1982).

Experimental and modeling methods for predicting the blended effluent concentration from GAC contactors were developed by Summers et al. (1997). The authors observed during this study that the time to GAC performance goals can be significantly extended by blending the effluent from multiple contactors. For the three waters examined, blending increased the run time by an average of 150 percent for both TOC and TTHM.

The research described above demonstrates how the performance of GAC systems can be influenced by many process variables. In general, the process can be modified to provide the same level of NOM removal at lower GAC usage rates by the following:

- Maintaining low pH conditions through the process
- Increasing NOM removal in processes that precede GAC adsorption
- Using an EBCT greater than or equal to 10 minutes

Ozonation prior to GAC does not guarantee improved NOM removals because it can either decrease or increase the ability to adsorb and increase the biodegradability of NOM. The overall impact of preozonation on NOM removal in GAC contactors depends on the efficiency of biotreatment to remove the weakly adsorbing hydrophilic fraction.

2.3.2 Nanofiltration

Nanofiltration is a high-pressure membrane process that has been traditionally used as a softening process to remove hardness ions. Generally, NF membranes reject divalent ions (e.g., Mg^{2+}, Ca^{2+}), but pass monovalent ions (e.g., Na^{+}, Cl^{-}). Recently, NF has been used more extensively for removal of DBP precursors and color, particularly in brackish waters, as well as other surface waters. Although NF processes remove nearly all turbidity in feed water, they cannot be used for turbidity removal in the same manner as MF and UF due to smaller pore sizes (Mallevialle et al. 1996). Smaller pore size makes NF membranes more prone to fouling. The application of NF for surface waters is generally not accomplished without extensive pretreatment for particle removal and possibly pretreatment for dissolved constituents.

The percentage of treated water that can be produced from the feed water is known as the recovery. Recovery is an important factor for cost of membrane processes and is one measure of the efficiency of a system. Recovery for NF systems is typically 75 to 90 percent and is impacted by feed water characteristics, membrane properties, and operating conditions, such as TMP. Since treatment and disposal of the reject stream (i.e., waste stream) can be a significant portion of the overall cost of a system, recovery can greatly affect cost efficiency.

2.3.2.1 Efficacy Against Pathogens

As would be expected based on MF and UF microbial removal efficiencies, NF processes are capable of excellent disinfection by removing nearly all microbial contaminants in feed water, including *Giardia*, *Cryptosporidium*, and viruses. Historically, NF processes have not been used as a primary means of disinfection, since, in large part, they have been used to treat ground water or have been coupled with pretreatment processes such as MF or UF. When only disinfection is required, MF and UF processes are typically used instead of NF, since they are less costly and can achieve the required level of pathogenic rejection (Mallevialle et al. 1996). Because of this, relatively few studies documenting microbial removal with NF membranes have been conducted in comparison to MF and UF processes. Because NF and RO processes represent systems that are very similar in terms of disinfection capabilities, available studies documenting microbial removal with RO as well as NF membranes are presented in Exhibit 2.17.

Exhibit 2.17: NF Studies Documenting Microbial Removal

Reference	Process	Membrane	Giardia Log Removal	Crypto Log Removal	MS2 Virus Log Removal
Gagliardo et al. (1999)	RO	HR	--	--	3.0
Gagliardo et al. (1999)	RO	DOW	--	--	5.4
Gagliardo et al. (1999)	RO	ESPA	--	--	4.7
Gagliardo et al. (1998)	RO	ULP	--	--	3.4
Seyde et al. (1999)	NF (Pilot)	Acumem-4040	>5[1]	>6[1]	4.2 to 5.0
Colvin et al. (1999)	RO (bench)	FilmTec BW30	--	--	>4[2]
Colvin et al. (1999)	RO (bench)	FilmTec BW30	--	--	>7[1]
Trussel et al. (1998)	RO (MF pretreat)	FilmTec BW30	--	--	4.1 to 5.9
Trussel et al. (1998)	RO (MF pretreat)	Hydranautics 4040 UHA	–	--	3.7 to 5.7
Trussel et al. (1998)	RO (MF pretreat)	Fluid Systems TFLC/M48 20HR	--	--	2.1 to 3.3
Trussel et al. (1998)	RO (MF pretreat)	Fluid Systems TFCL/ULP	–	--	2.9 to 4.3
Gagliardo et al. (1997)	RO (pilot)	TFC	>5.7	>5.7	3.0 to 4.0
Gagliardo et al. (1997)	RO (pilot)	CA	>5.7	>5.7	3.3 to 5.1

Note: [1] Indicates removal to detection limit.
[2] 0.02 µm Fluospheres
– Data not available

As shown in Exhibit 2.17, NF and RO processes are capable of significant log removals of cysts and viruses, which is to be expected since these microbes are much larger than the pore size of the membranes. However, the data in Exhibit 2.17 show that NF and RO systems are not an absolute barrier; they can allow microorganisms to pass through the membrane into the treated water. For this reason, it is important to consider membrane integrity testing when assessing the ability of a membrane to act as a barrier to microorganisms. Although no standard NF integrity testing method exists, some tests that have been proposed include vacuum testing and monitoring effluent water quality parameters such as chloride, UV-254 absorbance, microorganisms, and particle counts (Spangenberg et al. 1999). Vacuum testing entails taking the membrane off-line. This has the disadvantage of being unable to provide on-line integrity monitoring. Should a system become compromised, it would not be realized until the module is taken off-line and tested. Effluent water quality monitoring does provide real-time results. However, the parameters being monitored must be sensitive enough to provide an alert if the

system is compromised. Sensitivity of various parameters will depend on the influent level of that particular parameter along with the amount of removal accomplished by the membrane. The parameter acting as a surrogate for membrane integrity must be removed to a significant degree such that a noticeable increase in effluent concentration would be seen if the membrane system were compromised.

NF processes are also capable of reducing biodegradable organic carbon (BDOC) (Escobar and Randall 1999). Since BDOC serves as substrate for microorganisms in the distribution system, reducing BDOC can reduce the potential for regrowth in a distribution system, disinfectant doses, and DBPs. A recent full-scale study was performed to document the microbiological and disinfection benefits derived from implementing NF where conventional treatment had previously been practiced (Laurent et al. 1999). The results of this study showed significant decreases in chlorine residual fluctuations, microbiological counts, DOC, and BDOC in treated water and in the distribution system. In effect, this created greater water quality stability in all areas of the distribution system, particularly in areas with high residence times. In addition, the finished water chlorine dose required was lowered from about 1 mg/L to 0.2 mg/L by the use of NF.

2.3.2.2 DBP Precursor Removal

Membrane processes can remove DBP precursors through filtration and adsorption of organics. Membranes remove NOM through filtration (i.e., sieving) when NOM molecules are larger than a given membrane pore size, causing them to be rejected. Size, however, is only one factor that influences NOM rejection. Shape of the NOM molecules and membrane pores, along with chemical characteristics of the NOM molecules and membrane also play important roles in the permeation of NOM across a membrane (Mallevialle et al. 1996). Membranes may also remove NOM through adsorption of organics directly on the membrane surface. The level of adsorption to the membrane surface depends on the chemical characteristics, particularly charge and hydrophobicity, of both the membrane material and the NOM. Unfortunately, organic adsorption is generally undesirable since it has proven to be a primary cause of irreversible fouling of membranes, thereby shortening membrane life.

Without pretreatment, NF processes remove NOM to varying degrees. NOM removals for NF and RO processes are typically on the order of 50 to 99 percent. NOM removal depends on many factors, including membrane MWCO and hydrophobicity, characteristics of the NOM, and membrane system operating parameters such as recovery and operating pressure. Results from several studies on NOM removal by NF processes are provided in Exhibit 2.18.

Exhibit 2.18: NOM Removal Through NF Processes

Reference	Design Criteria	Conclusions of Study
Taylor et al. (1987 and 1989)	Operating pressure: 98-141 psi Flux: 8.9-16.4 gpd/sf Recovery: 50-79%	• MWCO of 100 to 500 are needed for DOC removal up to 90%. • MWCOs of 1000 to 3000 may achieve 50% DOC removal. • Trihalomethane formation potential (THMFP) and total organic halide formation potential (TOXFP) reductions up to 95% could be achieved with 300 MWCO. • Operating pressure had a negligible impact on NOM removal. • TDS[1] and hardness rejection are increased by increased operating pressure.
Conlon and McClellan (1989)	Operating pressure: 90-100 psi Recovery: 75%	• NOM removal greater than 90% for 200 MWCO.
Allgeier and Summers (1995)	Operating pressure: 95 psi Flux: 15-24 gpd/sf Recovery: 30-87%	• 66-94% TOC removal for 200 MWCO. • TOC removal decreased by up to 15% as recovery approached 90%.
Lozier et al. (1997)	Operating pressure: 70 psi Flux: 10 gpd/sf Recovery: 85%	• 69-98% TOC removal using MF pre-treated water.
Chellam et al. (1997)	Operating pressure: 70 psi Flux: 10 gpd/sf Recovery: 85%	• 90-95% TOC removal with 200 MWCO on MF and UF pretreated water. • 95-99% SDS THM precursor removal. • 96-99% SDS HAA6 precursor removal.
Mulford et al. (1999)	Operating pressure: 100 psi Flux: 15 gpd/sf Recovery: 82%	• 96% DOC removal with 200 MWCO.
Fu et al. (1995)	Operating pressure: 80 psi Flux: 15-20 gpd/sf Recovery: 75-90%	• 85-97% TOC removal with 100 to 500 MWCO.
Yoon et al. (1999)	Not reported	• 60-90% TOC removal with 200 to 8,000 MWCO. • Slightly higher NOM removal is achieved at pilot-scale than at bench-scale.
Legube et al. (1995)	Not reported	• 79-91% DOC removal. • 91-95% TOXFP reduction. • 93-94% THMFP reduction.

[1]TDS = total dissolved solids

In addition to NOM removal, NF processes are capable of some DBP removal, although little work has been performed in the area. Bromide removal is also important for the reduction of brominated DBPs. NF membranes are capable of significant bromide removal. Several studies documenting the use of NF processes for bromide removal are summarized in Exhibit 2.19.

Exhibit 2.19: Bromide Removal Through NF Processes

Reference	Conclusions of Study
Amy and Siddiqui (1999)	38-41% bromide removal with 150 to 300 MWCO.
Mulford et al. (1999)	50-63% bromide removal with 200 MWCO.
Allgeier and Summers (1995)	40-61% bromide removal with 200 MWCO.
Fu et al. (1995)	24-38% bromide removal with 100 to 500 MWCO.
Prados-Ramirez et al. (1993)	63% bromide removal.
Conlon and McClellan; Taylor et al. (1989)	60-70% chloride removal, with bromide removal expected to be nearly identical.

As shown by the data in Exhibit 2.19, NF is capable of high percentage bromide removal. Overall, however, bromide removal using NF would probably not be cost effective if used only for that purpose. If the process were incorporated into a treatment train and used for other contaminant removal, membrane removal of bromide may become cost effective (Amy and Siddiqui 1999). It is important to note that, if bromide is not removed sufficiently but TOC levels are reduced, the bromide-to-TOC ratio will increase considerably and will cause a net shift in speciation of DBPs to the more brominated compounds. In the worst case, such a scenario could cause a net increase in the absolute level of brominated DBPs (i.e., bromoform) after chlorination (Amy and Siddiqui 1999).

2.3.2.3 Factors Affecting Performance

NF is gaining popularity as a DBP precursor removal process, since production costs are comparable with competing processes (Mallevialle et al.1996). Due to the small pore size associated with NF, other feed water constituents will also be removed. For example, divalent salts, some metals, and some soluble organic carbon (SOCs) may be rejected by these membranes and, therefore, be concentrated in the waste stream. This may increase the cost associated with disposing of the waste stream compared to disposal costs associated with MF, UF, and conventional treatment processes. If regulatory limits prohibit sending the waste stream to a receiving body, costs for waste handling and disposal can be a substantial portion of the overall treatment cost.

MWCO is a key characteristic affecting membrane performance. Membranes with MWCOs in the 100 to 500 range appear to be very effective as a means of DBP precursor removal. TOC, THMFP, and TOXFP reductions of 70 to 95 percent are commonly achieved in systems using such membranes. These processes can effectively remove bromide as well, with reductions up to 95 percent. Larger MWCO membranes (i.e., MWCO near and above 10,000), however, will not be as effective for NOM reduction.

Commercial NF (as well as MF and UF) membranes are available in many types of material (e.g., cellulose acetate and polysulphone) and in various configurations (e.g., spiral wound and hollow fiber). The chemistry of the membrane material, particularly surface charge and hydrophobicity, can play an important role in rejection properties, since membranes can remove contaminants through adsorption on the membrane surface as well as through sieving across the membrane pores. These factors must be taken into consideration to accommodate source water characteristics and removal requirements.

Source water quality can also dictate pretreatment requirements. The small pore size of NF and RO membranes makes them more prone to fouling than UF or MF membranes, necessitating higher quality feed water. The application of NF and RO for surface water treatment is generally not accomplished without extensive pretreatment for particle removal and possibly pretreatment for dissolved constituents. For example, the rejection of scale-forming ions, such as calcium and silica, can lead to precipitation on the membrane surface since these ions are concentrated on the feed side of NF and RO membranes. Organic constituents and metal compounds, such as iron and manganese, can promote fouling through precipitation and adsorption as well. Precipitation and adsorption can result in irreversible fouling and must be avoided through appropriate pretreatment, including anti-scaling chemical and/or acid pretreatment and possibly pretreatment for organics removal.

In terms of contaminant removal, membrane performance can also be influenced by the operating pressure and percent recovery, depending on the mechanism of rejection. (This is true for NF and RO systems, but generally not true for MF and UF systems.) Contaminant rejection by NF and RO systems generally increases with decreasing operating pressure and with decreasing recovery. Thus, rejection can be enhanced by changing operating parameters, but not without corresponding increases in operating costs. To increase recovery, membranes are often staged (i.e., the concentrate of one stage of membranes is treated by another stage of membranes). Two to three stages are common for NF and RO systems. (Staging, however, is generally not used for MF and UF.) Staging is also used to keep the fluid velocity across the membranes at a specified rate. The maximum attainable percent recovery is usually governed by the degree to which the water can be concentrated without the occurrence of precipitation for NF and RO.

3. Technology Design and Criteria

3.1 Introduction

This Chapter provides assumptions related to the overall design for each technology addressed in this document. Types of information provided in this Chapter include:

- Assumed water quality conditions (e.g., median filter water quality assumptions for UV design)
- Chemical doses (e.g., ozone dose for *Cryptosporidium* inactivation)
- Equipment type (e.g., types of UV lamps for various system sizes)

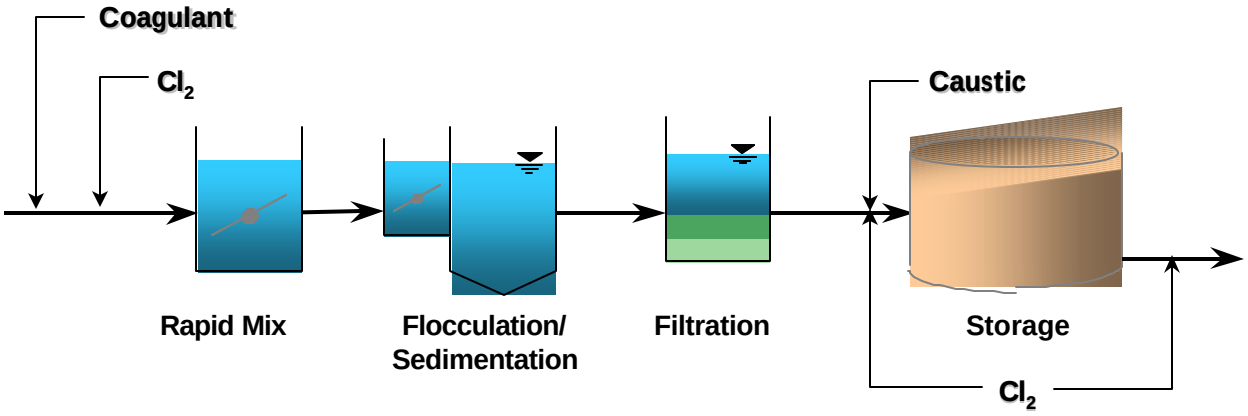
Chapter 4 builds on this Chapter by providing more detailed design assumptions for technology components and presents the costs for each technology.

Section 3.2 describes the assumed based treatment plant used for all technology modifications. Sections 3.3 and 3.4 describe the design approach for alternative disinfectant and DBP precursor removal technologies, respectively.

3.2 Base Treatment Plant

The base treatment plant is assumed to represent the existing treatment configuration. All modifications with alternative disinfection strategies and removal of DBP precursors are assumed to be retrofitted from this base treatment plant. The base plant is represented by a conventional treatment plant, employing the basic processes of coagulant addition and mixing, flocculation, clarification, granular media filtration, and chlorination for both primary disinfection and maintenance of a distribution system residual. A schematic of the base plant is shown in Exhibit 3.1.

Exhibit 3.1: Base Plant



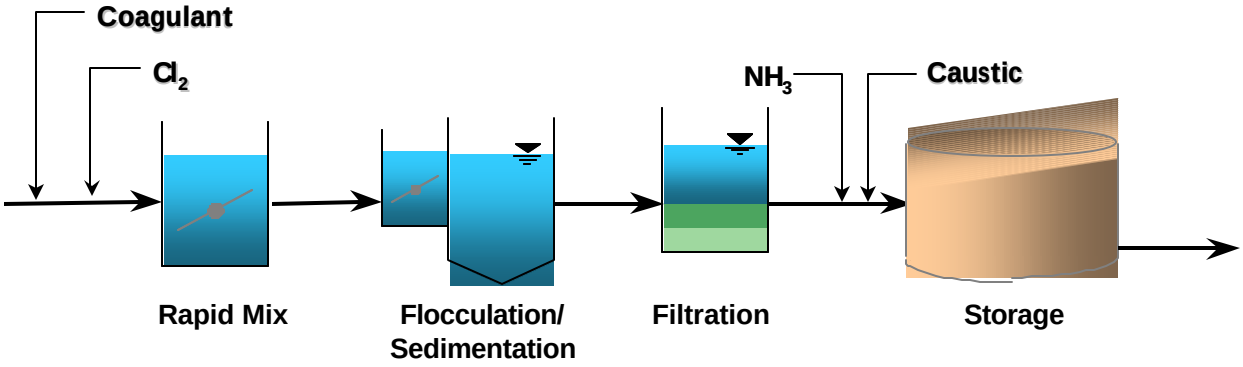
3.3 Alternative Disinfection Strategies

Pertinent to compliance with the Stage 2 DBPR and the LT2ESWTR, alternative disinfection strategies may be selected to provide additional treatment for *Cryptosporidium* and/or to limit the formation of DBPs. This section describes the overall design approach used for costing a number of alternative disinfection strategies capable of achieving these goals.

3.3.1 Chloramination

Chloramines can be used for secondary disinfection to limit DBP formation in the distribution system. Chloramines are less effective for microbial inactivation than chlorine and are typically ineffective as a primary disinfectant; however, they may be used in combination with other technologies discussed in this section (e.g., ozone for primary disinfection) to reduce DBP formation in the distribution system. Typically, ammonia is added after filtration (or possibly after storage) to quench the chlorine residual and form chloramines. A schematic of a chloramine system is shown in Exhibit 3.2

Exhibit 3.2: Chloramines for Secondary Disinfection



Description of Process: Pre-chlorination for primary disinfection; add ammonia after filtration at a residual chlorine to ammonia ratio of 4:1.

A range of finished water chlorine residuals were derived using the ICR database. The 10th and 90th percentile finished water chlorine residuals from the ICR database are approximately 0.6 and 2.2 mg/L, respectively. From these residuals, the ammonia dosages of 0.15 and 0.55 mg/L were derived assuming a 4:1 chlorine to ammonia ratio (typical chlorine to ammonia ratios are between 3:1 and 5:1 to ensure monochloramine formation). Upgrade costs were generated only for ammonia storage and feed systems (the base plant is assumed to provide the necessary chlorine). It is assumed that all chloramination can be accomplished at the plant and that no distribution system booster stations are required.

Aqueous ammonia is assumed for small systems (<1 mgd), and anhydrous ammonia is assumed for large systems (>1 mgd). Anhydrous ammonia is generally more cost effective for larger utilities; however, safety and handling issues with anhydrous ammonia also need to be considered. The aqueous ammonia system consists of a chemical storage container, metering pumps, an on-line process analyzer, piping, and valves. The anhydrous ammonia system consists of bulk storage pressure vessels, a vacuum feed system, an on-line process analyzer, piping, and valves: The larger systems may also include a vaporizer and an emergency scrubber system.

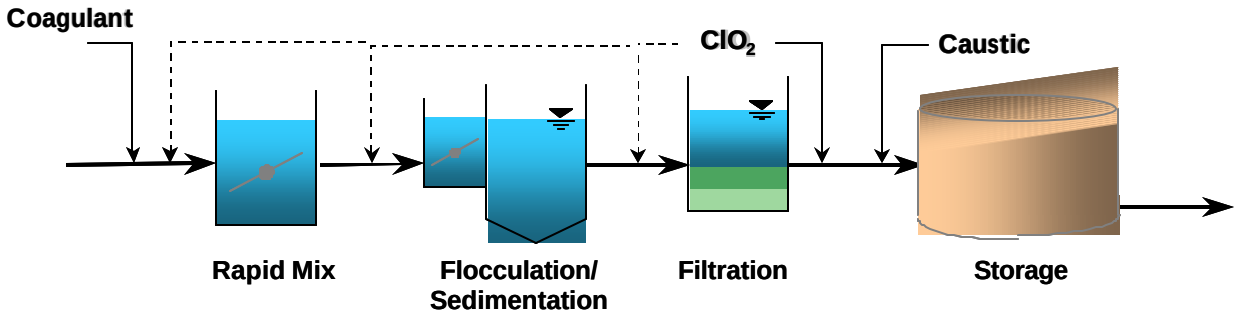
3.3.2 Chlorine Dioxide

Chlorine dioxide is an effective oxidant/disinfectant that is frequently used to control THM formation. It has also been shown to inactivate *Cryptosporidium*, as described in Chapter 2. Thus, chlorine dioxide can replace chlorine (or other oxidants) as the primary disinfectant and potentially achieve a greater level of pathogen inactivation while decreasing THM and HAA formation. However, controlling the formation of chlorite ions can be a considerable challenge in chlorine dioxide treatment implementation.

Because of the significant operator attention required to monitor and control chlorite formation as well as to address safety concerns, it is assumed that systems serving fewer than 500 people will not have the expertise necessary to use this technology. Therefore, costs are only developed for systems with a design flow of 0.091 mgd or greater.

Many plants add chlorine dioxide as a pre-oxidant, but it can also be added after filtration. For the analysis presented here, it is assumed that chlorine dioxide can be added at any point in the process train. (A schematic of the chlorine dioxide system is shown in Exhibit 3.3.) Chlorine dioxide costs do not include construction of a basin for additional chlorine dioxide contact time. It is assumed that plants can achieve adequate contact time with their existing configuration.

Exhibit 3.3: Disinfection with Chlorine Dioxide



Description of Process: Replace chlorination with chlorine dioxide addition.
Point of addition may be
1) prior to rapid mix, or
2) prior to flocculation, or
3) prior to filtration, or
4) post filtration

All chlorine dioxide cost analyses presented in this document are based on an applied dose of 1.25 mg/L. This is close to the maximum dosage of chlorine dioxide that can be added while remaining in compliance with a 1.0 mg/L MCL for chlorite, conservatively assuming a 70 percent conversion of chlorine dioxide to chlorite and a safety factor to account for impurities, such as unreacted chlorine, in the chlorine dioxide feed. This analysis evaluated chlorine dioxide costs at the maximum dosage because chlorine dioxide is being considered here for inactivation of *Giardia* and *Cryptosporidium*. Protozoa inactivation by chlorine dioxide typically requires high CT values as described in Chapter 2. Additionally, evaluating the maximum chlorine dioxide dose provides a degree of conservatism to these cost estimates. The level of *Cryptosporidium* inactivation that would be achieved by this dose depends on water quality and contact time and is not assessed in this cost analysis. Higher doses that would necessitate the removal of chlorite are not evaluated at this time due to uncertainty about the applicability and efficacy of chlorite removal processes.

For all systems, the use of an automatic generator is assumed. Key design assumptions for large systems are presented below.

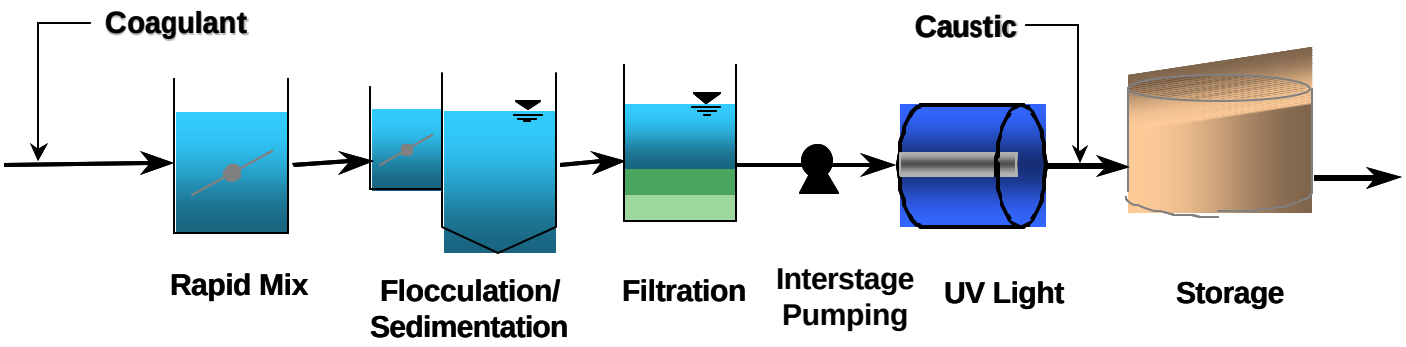
- Chlorine dioxide generation is accomplished through addition of sodium chlorite to a chlorine solution created by dissolution of chlorine gas in water.
- A sodium chlorite metering and mixing system is provided.
- A chlorine dioxide generator (detention time = 0.2 minutes) is provided.
- A polyethylene day tank and mixer are provided to store chlorine dioxide prior to its addition to the process.
- A dual head metering pump is provided to add chlorine dioxide to the process.
- A 1:1 mass ratio of chlorine gas to sodium chlorite is assumed to ensure that the sodium chlorite is completely utilized. (The additional chlorine serves to lower the pH for reaction through creation of hypochlorous acid.)

It is assumed that small systems (<2 mgd) will rent the ClO_2 generation equipment and only incur capital costs for instrumentation and piping and valves.

3.3.3 Ultraviolet Light

UV light is an effective disinfectant for bacteria, viruses, *Giardia*, and *Cryptosporidium* and does not form THMs or HAAs (see Chapter 2). For cost estimates in this document, a conceptual design for retrofitting the base plant with a UV disinfection system was developed based on plant flow (i.e., system size category) and water quality. Because particulate matter may affect the performance of UV systems, the cost estimates assume that the UV system is installed downstream from the filter. Exhibit 3.4 presents a schematic of a conventional water treatment plant (WTP) with UV disinfection. As shown in the schematic, interstage pumping is assumed because many utilities will not have sufficient hydraulic head to support the addition of UV disinfection facilities without significantly affecting plant operation.

Exhibit 3.4: UV Disinfection



Description of Process: Replace chlorination with UV light for disinfection.

The filtered water quality conditions assumed for all UV costs are based on median values reported in the ICR, as indicated in Exhibit 3.5.

Exhibit 3.5: Water Quality Assumptions for UV Disinfection

Parameter	Value
UV 254 absorbance[1] (cm^{-1})	0.051
UVT (%)[1]	89
Turbidity (	0.1
Alkalinity (mg/L as $CaCO_3$)[2]	60
Hardness (mg/L as $CaCO_3$)[2]	100

[1] Median of maximum filtered water UVT (minimum UV absorbance) from the ICR data
[2] Median of all ICR filtered water data
Source: ICR Data

Cost estimates for UV are provided for two UV doses: 40 and 200 mJ/cm^2. As discussed in Chapter 2, a UV dose of 40 mJ/cm^2 has been shown to be sufficient for 3 log inactivation of *Cryptosporidium* and *Giardia* and 1 to 2 log inactivation of viruses. Studies have shown that a UV dose of 200 mJ/cm^2 is adequate for 4 log inactivation of viruses.

Low pressure UV lamp based systems have been used for small treatment plants but are not typically installed at larger facilities due to the high number of lamps that would be required. Medium pressure lamp systems are not typically used for smaller utilities due to higher capital costs in comparison to LP systems at low flow rates. Therefore, UV reactors utilizing LP lamps are assumed for the small system (<1 mgd) designs. Depending upon the manufacturer, LPHO and/or MP reactors are provided in the large system (>1 mgd) cost estimates.

All UV systems are designed with an equipment redundancy of one extra UV reactor (n+1) or 15 percent capacity above design flow, whichever is greater. The number of reactors costed for each system size is shown in Exhibit 3.6 below. The number of reactors for each design flow is based on currently available UV reactor sizes and flows.

Exhibit 3.6 Number of Assumed UV Reactors

Design Flow (mgd)	Duty UV Reactors	Standby UV Reactors	Total Number of UV Reactors
0.022 - 3.5	1	1	2
17	3	1	4
76	5	1	6
210	11	2	13
430	22	4	26

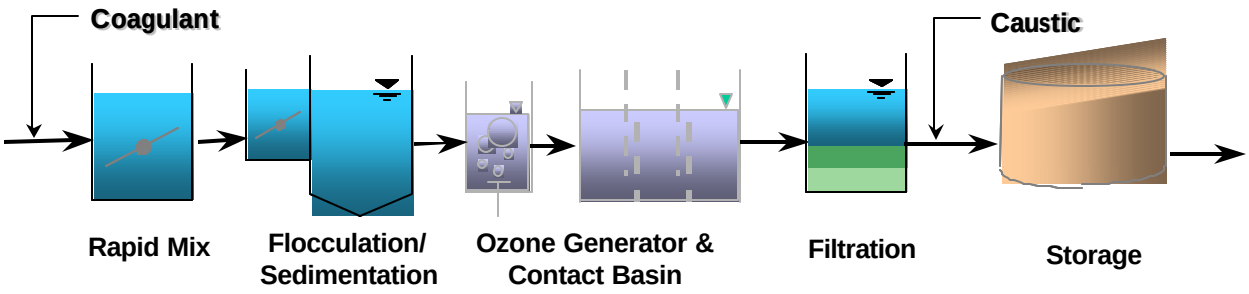
UV disinfection systems are sensitive to power interruptions and fluctuations. When a UV reactor goes down, it can take from four to ten minutes for the UV lamps to regain full power. A utility with poor power quality might have problems with their UV systems going down too frequently. One way to prevent this problem is to install a uninterruptible power supply (UPS), which is essentially a battery that smooths out the power interruptions and fluctuations. Because some systems may need UPS systems, cost estimates in Chapter 4 are completed at UV doses of 40 and 200 mJ/cm^2, with and without UPS systems.

3.3.4 Ozone

Ozone can be used to replace chlorine for primary disinfection and can provide a higher level of inactivation of certain pathogens, such as *Cryptosporidium*, while reducing formation of THMs and HAAs. Ozone is one of the most powerful oxidants available for water treatment (second only to the hydroxyl free radical). Disinfection with ozone is influenced by water quality characteristics such as pH, temperature, alkalinity, TOC, and certain inorganic compounds like iron and manganese. The use of ozone can be limited by raw water bromide levels and consequent bromate formation. These factors, in conjunction with the CT necessary for the desired level of pathogen inactivation, impact the design and operation of the ozone system.

A schematic of the ozone configuration is shown in Exhibit 3.7. The costing process allows for ozone application to either raw or settled water (settled water application is depicted in Exhibit 3.7). To control bromate formation during ozonation, it may be necessary to lower the pH in certain waters. Separate costs are estimated for pH adjustment so that this cost may be added to the costs of ozonation, where appropriate. The pH adjustment costs include addition of a chemical feed system. To reduce the pH, sulfuric acid is used and caustic (after ozonation) is used to raise pH.

Exhibit 3.7: Ozone Disinfection



Description of Process: Replace chlorination with ozonation.

Costs for ozone treatment systems are directly related to the dose applied. For the purposes of the LT2ESWTR and the Stage 2 DBPR, three ozone doses are costed based on the three levels of *Cryptosporidium* inactivation: 0.5, 1.0, and 2.0 log. The Surface Water Analytical Tool (SWAT) model is used to calculate the ozone dose required for each inactivation level, based on CT tables in Chapter 2 (Exhibit 2.13) and assuming an ozone CT of 12 minutes. For each plant in the ICR survey, and for each month with data, the SWAT model was used for raw water characteristics and existing plant configurations to determine the dose required to achieve the desired *Cryptosporidium* inactivation. Mean and maximum doses were then determined for each ICR plant.

For costing purposes, two doses were established for each of the three *Cryptosporidium* inactivation levels (0.5, 1.0, and 2.0 logs). The median of all plant-mean ozone doses (1.78, 2.75, and 3.91 mg/L, respectively) were used to calculate operation and maintenance costs. This is the dose which will be the most common for all plants achieving the given inactivation and the dose most representative of daily plant flows. To determine capital costs, the median of the plant-maximum doses (3.19, 5.0, and 7.0 mg/L, respectively) are used, as systems will be designed to meet the maximum dose that could be required under typical conditions.

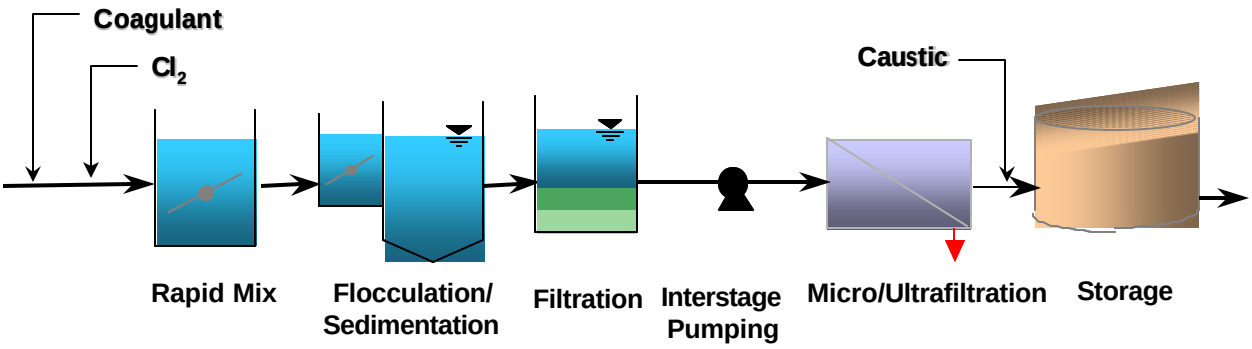
The primary components of the ozone process include in-plant pumping, ozone generation system, ozone contactor, off-gas destruction facilities, effluent ozone quench, stainless steel piping (including valves and ductwork), electrical and instrumentation (E&I), and chemical storage facilities. Components not related directly to the process (e.g., for which indirect costs are calculated) include piloting, permitting, land, operator training, and housing.

3.3.5 Microfiltration and Ultrafiltration

Microfiltration or ultrafiltration can be added to the base plant process train to enhance particle and microbial removal, including removal of *Cryptosporidium*. MF/UF may also allow treatment plants to reduce DBP formation by decreasing the disinfectant dose required to meet plant CT requirements. MF/UF can be added to the treatment process following conventional media filtration, or, in some cases, may be added as a replacement for media filtration. In certain applications (e.g., low total suspended solids (TSS) surface waters or groundwaters), MF/UF can replace the entire conventional treatment process. However, the design assumptions and costs presented in this document assume addition of MF/UF to an existing conventional treatment plant for enhanced removal of *Cryptosporidium* and/or DBP control. Consequently, the costs presented in Chapter 4 do not include all of the components that would be required to replace a conventional treatment train. A schematic of the MF/UF treatment process is shown in Exhibit 3.8

As discussed in section 2.2.5, flux is a critical design parameter for membrane applications and is often used in membrane procurements as a specification. However, the configuration of one membrane is often very dissimilar to that of another. Membrane fiber diameter, pore size, flow configuration (i.e., cross-flow vs. dead-end, pressure vessels vs. submersible membranes), and other membrane-specific factors can impact flux and other design and operating parameters. As a result, membrane feed water quality is used as the basis of design for the membrane portion of the costs presented.

Exhibit 3.8: Microfiltration and Ultrafiltration



Description of Process: Addition of microfiltration or ultrafiltration following granular media filtration. It may be necessary to move the point of chlorination to after MF/UF, as some membranes can be damaged by chlorine.

Cost estimates are based upon a design feed water temperature of 10°C. As previously discussed, temperature can have a significant impact on membrane system design. As the water temperature decreases, water viscosity increases. This, coupled with temperature effects on the membranes themselves, can result in the need for increased membrane area and/or increased operating

pressures to maintain the desired level of production. It is important to note that this effect can vary from membrane to membrane, and many manufacturers have developed membrane-specific correction factors.

Membrane system costs were approximated using estimates provided by four manufacturers (all pressure vessel systems). The only criteria given to the manufacturers was the feed water temperature of 10°C. Since the design assumes a post-filtration retrofit, the effect of solids loading on the membrane is considered minimal and was not specified for manufacturer estimates. Each manufacturer then used its own flux specifications and temperature correction factor to provide cost estimates for design flows ranging from 0.01 to 430 mgd. Estimates for design flows of 0.007 and 520 mgd were extrapolated from these estimates.

The membrane costs from the manufacturers include skid-mounted membrane modules with associated piping, feed pumps, backwash and recirculation pumps (where appropriate), chemical cleaning feed tanks and pumps, and instrumentation and control for proper operation. Additional instrumentation and control and pipes and valves were included in process costs for interconnection with existing plant control systems and processes. Interstage pumping was also added based on the assumption that the existing plant may not have sufficient hydraulic head to accommodate the membrane process. O&M costs include replacement membranes (membrane life is 5 years), process power, chemicals for cleaning, and labor.

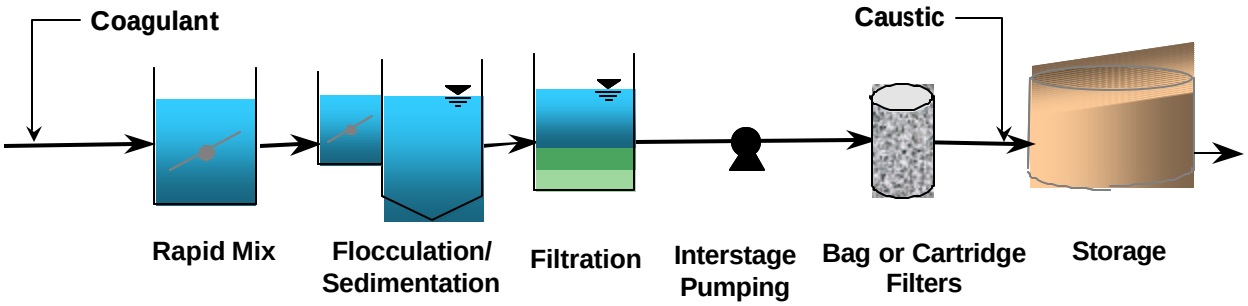
For the purposes of design, it was assumed backwash and reject water could be discharged to a sanitary sewer for treatment at a publicly owned treatment works (POTW). This assumes the sanitary sewer has sufficient capacity to accommodate the increase in flow, and the POTW is able to handle the increase in daily flow. However, in many cases, the reject and backwash water can be recycled to the head of the treatment plant. In some instances, recycle may be a lower cost option than discharge to a POTW. In other cases, recycle may require additional pumping and site piping, modification or addition of chemical feed systems, installation of equalization basins, or expansion of other process components. Therefore, the costs associated with POTW discharge represent a conservative estimate in some cases (i.e., where recycle requires few process improvements) and may underestimate costs in others (i.e., where extensive improvements are necessary). However, for the purposes of approximating treatment costs, POTW discharge represents an approximate average cost per utility.

3.3.6 Bag and Cartridge Filtration

Bag and cartridge filters may be an attractive, low cost option for small systems to improve microbial removal. Filter bags and cartridges are available in a number of different materials and a wide range of pore sizes. The removal efficiency can be affected by the filter material, pore size distribution, and filter durability. Filter durability affects how a filter stands up to routine cleaning and affects replacement frequency.

It is assumed, for the purposes of this document, that bag or cartridge filters are installed downstream of existing granular media filters. Exhibit 3.9 presents a schematic of bag and cartridge filtration.

Exhibit 3.9: Bag and Cartridge Filtration



Description of Process: Addition of bag filters OR cartridge filters following granular media filtration.

Costs for different bag and cartridge filter construction materials were used to develop a range of costs. The frequency of replacement depends upon the durability of construction and water quality and can vary from a few weeks to as long as a year. This can have a significant impact on O&M costs. Filter housings are available in carbon steel for approximately half the cost of a stainless steel unit. However, for drinking water application, stainless steel is more likely to be the material of choice. As a result, only stainless steel housing was considered in development of costs.

3.3.7 Bank Filtration

Bank filtration may be advantageous for systems that currently have surface intake from a stream which is underlain by a granular media. Such a system would essentially drill a well below the water table created by the surface water source. The well would replace the existing surface water intake. Particles and other contaminants would be trapped in the pores of the river bed material or adsorb onto the river bed material. The river bed material thus acts as a pre-filtration step for the treatment process.

3.3.8 Second Stage Filtration

Second stage filtration may be a desirable option for systems with frequent fluctuations in hydraulics and turbidity. Second stage filtration, like single stage filtration, operates by depth removal.

Depth filtration is when the solids are removed within the granular media. The surface area of the media provides attachment sites for the particles suspended in the influent water.

To meet EPA's proposed 0.5 log credit for *Cryptosporidium* removal, second stage filtration must have the following characteristics:

- First stage of filtration must be preceded by a coagulation step.
- Both filtration stages must treat 100 percent of plant flow.

3.3.9 Pre-Sedimentation

Pre-sedimentation basins will be useful for systems with high influent turbidities and high particle counts. EPA is proposing to give pre-sedimentation basins with coagulant addition 0.5 log credit if the following criteria are met:

- All flow must pass through basin.
- Continuous flow through basin and coagulant addition near the influent of the pre-sedimentation basin while plant is in operation.
- Maximum day settling surface loading rate of 1.6 gpm/ft^2.
- Annual mean influent turbidity $\geq$ 10 NTU or maximum daily influent turbidity $\geq$ 100 NTU.

Systems with existing pre-sedimentation basins may monitor after the pre-sedimentation basin and prior to the main treatment plant for the purpose of determining LT2ESWTR bin assignment. Costs in Chapter 4 were determined assuming that the basin met all the above specifications.

3.3.10 Watershed Control

Each PWS's watershed control program plan is expected to be site-specific and will depend on the hydrology and land use in the watershed, the location and type of *Cryptosporidium* sources in the watershed, the population served, size of the watershed, funding, and other issues. Watershed programs may include the following:

- Monitoring for *Cryptosporidium* or indicator organisms throughout the watershed
- Fencing or otherwise restricting access to the source water

- Land acquisition
- Managing land owned by the PWS
- Working with sewer or stormwater utilities to develop plans to upgrade treatment or otherwise reduce pollutant loads
- Working with municipal governments to regulate land use and development,
- Conducting outreach to other stakeholders

To receive credit for removal of *Cryptosporidium*, a watershed control program must have the following elements:

- It must be reviewed and approved by the primacy agency.
- It must include an analysis of the system's source water vulnerability to the different sources of *Cryptosporidium* identified in the watershed. The vulnerability assessment must include a characterization of the watershed hydrology and identification of an "area of influence on source water quality" (i.e., the area to be considered in future watershed surveys). The assessment must also address sources of *Cryptosporidium*, seasonal variability, and the relative impact of the sources of *Cryptosporidium* on the system's source water quality. It is likely that water systems will obtain much of the information to be provided in the vulnerability assessment from the source water assessment performed as part of the State source water assessment program.
- It must present an analysis of sustainable interventions and an evaluation of their relative effectiveness in reducing *Cryptosporidium* in source water. Interventions may include anything from outreach to point source elimination.
- It must address goals and define and prioritize specific actions to reduce source water *Cryptosporidium* levels. The plan must 1) explain how actions are expected to contribute to specified goals, 2) identify partners and their roles, resource requirements and commitments, and 3) include a schedule for plan implementation.
- It must include submission of an annual report performance of a watershed survey, and submission of a request for review and reapproval.

A watershed control program could include interventions such as 1) the elimination, reduction, or treatment of discharges of contaminated wastewater or storm water, 2) treatment of *Cryptosporidium* contamination at the site of generation or storage, and 3) prevention of *Cryptosporidium* migration from the source (e.g., farms or wastewater treatment plants). The

feasibility and sustainability of various interventions may depend on the cooperation of other stakeholders in the watershed. Stakeholders that have some level of control over activities that could contribute to *Cryptosporidium* contamination include municipal government, private operators of wastewater treatment plants, livestock farmers, and other government and commercial organizations.

The LT2ESWTR does not specifically mandate any interventions that must be included in a watershed control program plan. The only required elements are those submitted with an application for approval of the watershed control program plan. These are the delineation of an "area of influence on water quality" and a vulnerability assessment. Watershed delineation and susceptibility analyses are already required under the Source Water Assessment Program; data gathered under this program can, in many cases, be used in preparing information required for the application.

3.3.11 Combined Filter Performance

Combined filter performance is not a single technology but many different activities that can improve existing filtration processes to enhance performance. Plants, which can operate their filters in such a way to produce 0.15 NTU or lower turbidity water 95 percent of the time, will receive a 0.5 log *Cryptosporidium* reduction credit under the LT2ESWTR.

The Regulatory Impact Analyses (RIAs) for the IESWTR and LT1SWTR identified 35 actions that facilities could take to lower the finished water turbidity from the SWTR standard of 0.5 NTU to the IESWTR standard of 0.3 NTU. These tasks were examined and professional judgement was applied to determine which of these actions would be helpful in further lowering turbidity from 0.3 to 0.15 NTU.

In determining processes that could further reduce filtered water turbidity, systems that would select this *Cryptosporidium* removal option were assumed to be conventional filtration or softening plants which were already operating well within the 0.3 NTU standard currently. These plants would likely have to make only minor modifications to the treatment process to meet the 0.15 NTU standard. These plants were also assumed to be operating under 0.15 NTU less than 95 percent of the time or to be capable of achieving 0.15 NTU.

Based on these assumptions, the filter improvements listed in the IESWTR were reviewed for applicability to this treatment option. The following were considered as possible actions that systems may take to implement this option:

- Installing backwash polymer feed capability
- Installing coagulant feed points
- Adding filter media

- Adding filter to waste capabilities
- Replacing the filter rate-of-flow controller
- Increasing plant staffing
- Increasing staff qualifications
- Purchasing or replacing bench-top turbidimeters
- Purchasing or replacing jar test apparatus
- Purchasing or replacing a particle counter or streaming potential meter
- Staff training

It is not assumed that each system using this technology will use all eleven tasks. Instead, it is assumed that each system would have to use at least one of these tasks and, most likely, two or more to meet the turbidity targets (successfully). To develop costs for this technology, the percentage of the plants choosing each action was determined. The percentage of systems choosing a particular task was then multiplied by the unit cost for that task to arrive at an average unit cost for all plants. Further details of the percentages and costs are given in Chapter 4 of this document.

The assumptions for each filter improvement action is discussed below.

Installing Backwash Water Polymer/Coagulant Feed Capability

Adding coagulant to backwash water aids in filter ripening and helps to reduce post backwash turbidity spikes. Systems choosing backwash polymer to lower turbidity were assumed to not have this capability currently. Costs were for a dry polymer feed system that can be loaded with a seven-day polymer supply.

Installing Additional Coagulant Feed Points

Installing additional coagulant feed points can help to improve coagulation of particles and their removal by settling. Capital costs were based on feeding an additional 5 parts per million (ppm) dose of primary coagulant. The primary coagulant is assumed to be ferric chloride, ferric sulfate, or alum. Thirty days of bulk storage are assumed for ferric chloride or ferric sulfate (equivalent to approximately fifteen days of storage for alum).

Adding Filter Media

Often during routine operation of filters, media is lost either through attrition and passage out the underdrains or through the backwash. If too much media is lost, filter performance will suffer. Therefore, adding additional media can often improve turbidity in the effluent.

Adding Filter to Waste Capabilities

Filter turbidity often spikes immediately after backwashing. Installing filter to waste capabilities allows water to be wasted after a backwash instead of sending the high turbidity water to the CFE. Costs included piping, valves, and fittings.

Installing or Replacing Filter Rate-of-Flow Controllers

Flow surges can cause spikes in filter turbidity. Installing a rate-of-flow controller or replacing a faulty one can improve performance. Costs were for replacing a unit and were based on assumed 24-hour operation.

Increasing Plant Staffing

Systems which only have part time staff or are understaffed may have trouble controlling filter conditions closely enough to meet the 0.15 NTU turbidity target. Hiring additional staff or extending current staff's hours may help systems to more finely control filter operations.

Increasing Staff Qualifications

Better trained staff may be able to recognize conditions which lead to filter turbidity breakthrough and to prevent it. Costs for this option were based on the cost of sending an operator to a training class. Costs include class registration fees to attend an operator certification class.

Purchasing or Replacing Bench-Top Turbidimeters

Typically, every plant has at least one bench-top or on-line turbidimeter. However, some of these units may be obsolete to meet the monitoring requirements of the LT2ESWTR for combined and individual filter effluents. Bench-top turbidimeters do not appear to be suited to fulfill these monitoring tasks. Therefore the purchase of up-to-date on-line turbidimeters with electronic data acquisition interface was costed.

Purchasing or Replacing Jar Testing Apparatus

A jar testing apparatus is necessary for optimizing coagulant and polymer dosing. Old units will need to be replaced, and new units purchased if a facility does not have one. Systems serving greater than 100,000 people were assumed to buy two units, and those serving more than 1,000,000 people were assumed to purchase three units.

Purchasing or Replacing a Particle Counter

Instruments such as particle counters, zetameters, and streaming current monitors can be used to optimize filter processes. The cost for this option assumes the purchase of one of these instruments. The cost of a particle monitor was used as a surrogate for any one of these three instruments.

Staff Training

Better trained staff will be better able to spot and fix problems in filter performance. The costs for this option were based on hiring a consultant to provide on-the-job training for 10 to 140 hours.

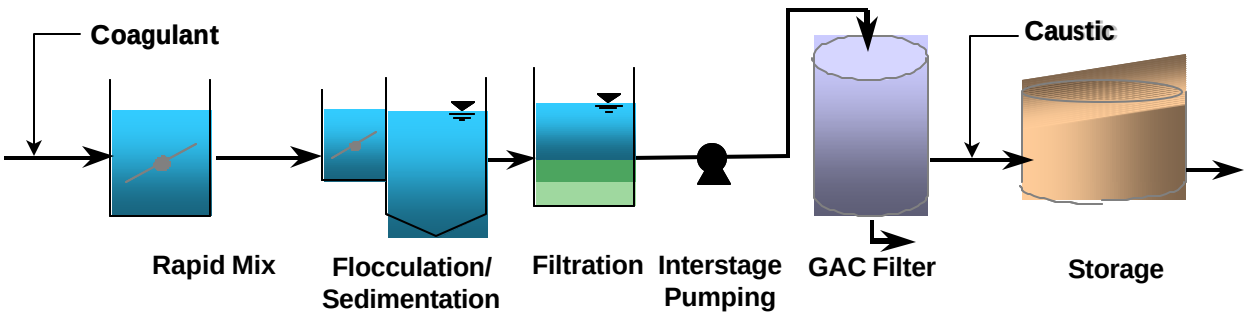
3.4 DBP Precursor Removal Technologies

A strategy for reducing DBP formation is removal of DBP precursors (e.g., natural organic matter). The technologies discussed in this section may not be applicable for all systems. Each technology section presents the approach and assumptions used to develop the costs presented in Chapter 4.

3.4.1 Granular Activated Carbon Adsorption

GAC filters reduce DBP formation by removing organic carbon. For the purposes of this document, installation was assumed after the existing filters. A schematic of the GAC process is shown in Exhibit 3.10.

Exhibit 3.10: GAC Filtration



Description of Process: Install GAC filter following granular media filter.

The application of GAC adsorption involves the following process design considerations:

- Empty bed contact time, volume of empty contactor divided by flow rate

- Reactivation interval or frequency, which affects the GAC usage rate (pounds of GAC used per gallon of water treated)
- Pre-treatment
- Contactor configuration (e.g., downflow versus upflow, pressure versus gravity, single-stage versus multi-stage or parallel, filter adsorber versus post-filter GAC contactor)
- Method of GAC reactivation (e.g., on-site versus off-site)
- Interstage pumping
- Performance monitoring (for TOC)

EBCTs of ten and twenty minutes were chosen for the cost evaluation based upon an analysis of EBCTs and NOM removal. This analysis indicated that EBCTs lower than 10 minutes do not remove sufficient NOM to warrant installation as a control for DBP precursors. Similarly, EBCTs in excess of 20 minutes do not provide significant improvements in NOM removal. Accordingly, 10 minutes and 20 minutes were selected to represent the upper and lower bounds of appropriate EBCTs for NOM removal.

Reactivation/replacement frequencies vary based on water quality and the number of contactors in parallel. For the purposes of this document frequencies of 90, 240, and 360 days were evaluated. Ninety days was selected as a minimum value based upon best professional judgement that reactivating at intervals lower than 90 days is impractical from an operational standpoint. Three hundred and sixty days was selected as the maximum reactivation frequency since the cost of GAC technology increases insignificantly for reactivation frequencies of greater than 1 year. High operating costs were captured by considering 90-day regeneration frequency for the GAC facility with EBCT of 20 minutes. Low operating costs were captured by considering 360-day regeneration frequency for the GAC facility with EBCT of 10 minutes. An intermediate operating cost was also captured by considering 240-day regeneration frequency for the GAC facility with EBCT of 20 minutes.

Based upon best professional judgement, it was decided that small systems are unlikely to regenerate on-site, since it requires more substantial capital investment and operator attention. As a result, small systems (less than 1 mgd) were assumed to operate on a replacement basis (i.e., when the carbon is spent, it is discarded and replaced with new carbon). Large systems (greater than 1 mgd) were assumed to regenerate on-site using multiple hearth furnaces.

Very small system GAC installations (< 0.1 mgd) include: pressure GAC contactors, virgin GAC, pressure booster pumps, pipes and valves, and instrumentation and controls. O&M is a function of regeneration frequency.

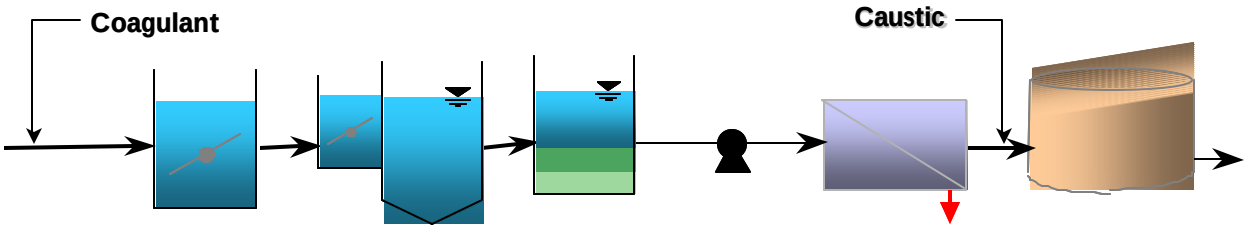
Small system GAC installations (>0.1 mgd and <1 mgd) include: pressure vessels designed for working pressure of 50 psi; factory assembled units mounted on steel skid 12 feet high and varying diameter depending on the EBCT; access for filling and removing carbon; pressure booster pump, valves, piping and pressure gauges, initial charge of activated carbon, supply and backwash pump, and electrical control panels.

Large system GAC installations (> 10 mgd) include: concrete gravity contactors 8.3 feet high; loading rate 5 gpm/ft^2; troughs and pipes for carbon removal as a slurry; other pipe gallery; pressure booster pump; flow measurement and instrumentation; master operations control panel; building; initial virgin carbon; single multiple-hearth furnace for carbon regeneration-loading rate of 50 pounds per square foot per day; and two TOC analyzers.

3.4.2 Nanofiltration

Nanofilters remove NOM, thereby reducing DBP formation. NF is an advanced treatment process which typically requires higher levels of pre- and post-treatment than traditional water treatment processes. For this cost analysis, nanofilters were assumed to be located downstream of existing filters. A schematic of the NF technology is shown in Exhibit 3.12.

Exhibit 3.11: Nanofiltration



Description of Process: Addition of nanofiltration following granular media filtration, OR replacement of granular media filters with nanofiltration

Typically, NF requires both physical and chemical pre-treatment. Pre-treatment is usually required for NF treatment of all surface waters and some ground waters. Physical pre-treatment often includes a component to remove particles, typically multi-media filtration, microfiltration, or cartridge filtration. Chemical pre-treatment often includes acid or anti-scalant addition to reduce the fouling potential of the feed water. Particle removal and softening with chemical addition are also used as pre-treatments. Attention should be paid to the compatibility of coagulant and the membrane for such situations.

Post-treatment may also be required, depending on the characteristics of the product water. NF product waters usually have low pH and total dissolved solids levels. This creates the potential for an unstable and corrosive finished water. Chemical post-treatment may be required to create a more stable and non-corrosive water. Commonly used post-treatments include addition of caustic (to raise the pH), soda ash (to raise pH and alkalinity), and poly/ortho phosphates for stabilizing the water. Blending a portion of raw water with finished water can also be used to stabilize the finished water.

The design criteria in this document assume that the NF system is an "add-on" process to an existing treatment plant which is generating a water that can be fed directly to the NF process without further pre-treatment. It is assumed that 100 percent of the design flow is passing through the NF membranes. Recoveries of 85 percent and operating pressures of 90-110 psi were assumed. Costs were developed assuming a design feed water temperature of 10 degrees Celsius. Like MF, the cost of a NF system can vary significantly with temperature because the membrane productivity, or flux (gallons/ft^2-day), is strongly dependent on feed water temperature. Empirical relations are available to estimate the flux at a design temperature using the flux at a reference temperature (i.e., 10 degrees Celsius). These relations are available both in published literature and with membrane manufacturers.

NF system cost quotations were obtained from manufacturers for all NF equipment items, including membrane elements, online instruments, booster pumps, clean-in-place systems and acid/anti-scalant addition systems. Unlike other treatment processes, membrane systems are typically supplied by the equipment vendor as package, skid-mounted units; therefore, smaller multipliers are assumed. Capital cost multipliers of 1.67 and 2.0 were used respectively for small and large systems to estimate total capital cost. It was assumed that a unit NF skid can produce up to 2 mgd of product water. NF systems smaller than 2 mgd were assumed to have fewer membrane modules and membranes.

The O&M costs include chemical usage, membrane replacement (assumed membrane life of five years), process/building power, additional labor hours, and process monitoring. Efforts were made to capture the drop in prices of the membranes, modules, and associated equipment over the past few years due to increasing use of the NF systems. Where necessary, the costs for retrofitting and operating an NF plant were verified with data from various surveys, including Florida's softening plants (Bergman 1996) and the Bureau of Reclamations (BOR 1997) surveys. The cost curves presented in Chapter 4 were verified with real-plant data for different flow levels.

NF design criteria developed here include handling of the brine stream generated by the NF process. This handling assumes direct discharge of the brine to a receiving body, ocean outfall, sanitary sewer, storm drain, or a salinity interceptor. The costs presented in Chapter 4 pertain only to plants located in areas where brine can readily be discharged to either a receiving water body, a sewer/storm drain, or a salinity interceptor. Plants located in areas where this is not an option will have significantly higher waste stream treatment and handling costs.