Engineering Feasibility Study on the Costs of Treatment Options for Reducing Disinfection Byproducts in Public Drinking Water Systems

Volume I

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Vermont Department of Environmental Conservation
Water Supply Division

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March 2010

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TABLE OF CONTENTS

Section — Page No.

ES EXECUTIVE SUMMARY ES-1
ES.1 INTRODUCTION ES-1
ES.2 STUDY OBJECTIVES ES-1
ES.3 REGULATORY REQUIREMENTS ES-2
ES.4 EXISTING AND EMERGING TECHNOLOGIES ES-3
ES.5 PERFORMANCE ASSESSMENT AND TECHNOLOGY SELECTION ES-4

1.0 INTRODUCTION AND BACKGROUND 1-1

1.1 INTRODUCTION 1-1

1.2 BACKGROUND 1-1

1.3 ABBREVIATIONS 1-4

2.0 STUDY OBJECTIVES AND METHODOLOGY 2-1

2.1 STUDY OBJECTIVES 2-1
2.2 METHODOLOGY 2-2
2.3 LIMITATIONS 2-5

3.0 REGULATORY REQUIREMENTS 3-1

3.1 INTRODUCTION 3-1

3.2 UNITED STATES ENVIRONMENTAL PROTECTION AGENCY 3-1
3.2.1 Stage 1 Disinfectants and Disinfection Byproducts Rule (DBPR) 3-2
3.2.2 Stage 2 Disinfectants and Disinfection Byproducts Rule (DBPR) 3-4
3.2.3 Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) 3-8
3.2.4 Lead and Copper Rule (LCR) 3-10
3.2.5 Total Coliform Rule (TCR) 3-12
3.2.6 Ground Water Rule (GWR) 3-15
3.2.7 Simultaneous Compliance Issues 3-18

3.3 VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION AND DEPARTMENT OF HEALTH 3-23

4.0 EXISTING AND EMERGING TECHNOLOGIES 4-1

4.1 PURPOSE OF DISINFECTION AND DISINFECTION BYPRODUCT FORMATION 4-1
4.1.1 Disinfection Purpose 4-1
4.1.2 Disinfection Byproduct Formation 4-1
4.1.3 Strategies for Reducing Disinfection Byproduct Formation 4-3

TABLE OF CONTENTS (Continued)

Section — Page No.

4.2 SOURCE WATER 4-5
4.2.1 Introduction 4-5
4.2.2 Source Water Characteristics 4-6
4.2.3 Strategies for Source Water Protection 4-11
4.2.4 Development/Research in Source Water Quality 4-18

4.3 PHYSICAL AND CHEMICAL PARAMETER TREATMENT PROCESSES 4-20
4.3.1 Introduction 4-20
4.3.2 Preoxidation 4-21
4.3.2.1 Prechlorination 4-22
4.3.2.2 Preozonation 4-22
4.3.2.3 Advanced Oxidation Processes 4-25
4.3.2.4 Chlorine Dioxide 4-26
4.3.2.5 Potassium Permanganate 4-27
4.3.2.6 Hydrogen Peroxide 4-28
4.3.3 Conventional Filtration 4-29
4.3.4 Biologically Active Filtration 4-29
4.3.4.1 DBP Precursor Removal 4-30
4.3.4.2 Process Interactions 4-32
4.3.4.3 DBP Removal 4-32
4.3.5 Conventional Coagulation/Flocculation 4-33
4.3.5.1 DBP Removal 4-33
4.3.5.2 Process Interactions 4-41
4.3.5.3 DBP Removal 4-41
4.3.6 Clarification 4-42
4.3.6.1 Sedimentation 4-42
4.3.6.2 Dissolved Air Flotation 4-43
4.3.6.3 Flottazone 4-44
4.3.7 Membrane Separation 4-44
4.3.7.1 DBP Precursor Removal 4-44
4.3.7.2 DBP Removal 4-48
4.3.8 Activated Carbon Adsorption 4-48
4.3.8.1 DBP Precursor Removal 4-49
4.3.8.2 Process Interactions 4-52
4.3.8.3 DBP Removal 4-52
4.3.9 Precipitative Softening 4-54
4.3.9.1 DBP Precursor removal 4-55
4.3.9.2 Process Interactions 4-57
4.3.9.3 DBP Removal 4-58
4.3.10 Ion Exchange (MIEX) 4-58
4.3.11 Summary 4-59

TABLE OF CONTENTS (Continued)

Section — Page No.

4.4 DISINFECTION PROCESSES AND INJECTION LOCATIONS 4-61
4.4.1 Introduction 4-61
4.4.2 Ozone 4-62
4.4.2.1 Disinfecting Characteristics 4-62
4.4.2.2 DBP Formation 4-65
4.4.2.3 Process Interactions 4-66
4.4.2.4 Application Point 4-66

4.4.3 Ultraviolet Light 4-67
4.4.3.1 Disinfection Characteristics 4-68
4.4.3.2 DBP Formation 4-71
4.4.3.3 Process Interactions 4-72
4.4.3.4 Application Points 4-73
4.4.4 Chlorine Dioxide 4-74
4.4.4.1 Disinfection Characteristics 4-75
4.4.4.2 Disinfection Byproducts Formation 4-78
4.4.4.3 Process Interactions 4-78
4.4.4.4 Application Points 4-79
4.4.5 Chloramine 4-79
4.4.5.1 Disinfection Characteristics 4-80
4.4.5.2 Process Interaction 4-83
4.4.5.3 Disinfection Byproducts Rule Formation 4-84
4.4.5.4 Application Point 4-91
4.4.6 Peroxone (Ozone/Hydrogen Peroxide) 4-91
4.4.7 Combinations of Disinfectants 4-92
4.4.7.1 Primary Disinfectants 4-93
4.4.7.2 Secondary Disinfectants 4-93
4.4.7.3 Disinfectant Combinations 4-94
4.4.7.4 DBP and Disinfectant Combinations 4-94
4.4.8 Physical Chemical Processes 4-100
4.4.8.1 Conventional Treatment 4-100
4.4.8.2 Direct Filtration 4-100
4.4.8.3 Slow Sand and Diatomaceous Earth Filtration 4-101
4.4.8.4 Membranes 4-101
4.4.8.5 Miscellaneous Filtration Technologies 4-105

4.5 STORAGE AND DISTRIBUTION 4-105
4.5.1 Introduction 4-105
4.5.2 Storage and Distribution System Characteristics 4-106
4.5.3 Effect of Treatment and Disinfection on Distribution System Water Quality 4-107
4.5.4 Strategies to Improve Water Quality in Distribution Systems 4-113
4.5.5 Development/Research in Distribution System Water Quality 4-117

TABLE OF CONTENTS (Continued)

Section — Page No.

4.6 POTENTIAL GROUNDWATER SOURCES 4-118
4.6.1 Introduction 4-118
4.6.2 Sources of Data on Groundwater Favorability 4-119
4.6.3 Groundwater Favorability Tables 4-127
4.6.4 Alburgh Village - Summary of Groundwater Favorability 4-129
4.6.5 Bennington - Summary of Groundwater Favorability 4-129
4.6.6 Burlington - Summary of Groundwater Favorability 4-131
4.6.7 Catamount-Bolton - Summary of Groundwater Favorability 4-131
4.6.8 Champlain Water District - Summary of Groundwater Favorability 4-133
4.6.9 Grand Isle - Summary of Groundwater Favorability 4-138
4.6.10 North Hero - Summary of Groundwater Favorability 4-138
4.6.11 Proctor - Summary of Groundwater Favorability 4-138
4.6.12 Readsboro - Summary of Groundwater Favorability 4-139
4.6.13 Richford - Summary of Groundwater Favorability 4-139
4.6.14 Rutland City - Summary of Groundwater Favorability 4-139
4.6.15 St. Johnsbury - Summary of Groundwater Favorability 4-143
4.6.16 Swanton - Summary of Groundwater Favorability 4-145
4.6.17 Tri-Town - Summary of Groundwater Favorability 4-145
4.6.18 Vergennes-Panton - Summary of Groundwater Favorability 4-147
4.6.19 Summary 4-147

5.0 SYSTEM SUMMARIES 5-1

5.1 DATA COLLECTION AND MANAGEMENT 5-1

5.2 TREATMENT TECHNOLOGY SELECTION 5-1
5.2.1 Modeling Existing System Performance and Recommended Alternative 5-1
5.2.1.1 Synthetic Water Quality for Model Runs 5-3
5.2.2 Simultaneous Compliance with Other Regulatory Rules 5-6
5.2.3 System Summary and Technology Recommendations 5-8
5.2.3.1 Alburgh Village - Existing System 5-12
5.2.3.2 Alburgh Village - Proposed Alternatives 5-13
5.2.3.3 Alburgh Village - Other DBP Reduction Alternatives 5-15
5.2.3.4 Bennington - Existing System 5-23
5.2.3.5 Bennington - Proposed Alternatives 5-26
5.2.3.6 Bennington - Other DBP Reduction Alternatives 5-29
5.2.3.7 Burlington - Existing System 5-36
5.2.3.8 Burlington - Proposed Alternatives 5-38
5.2.3.9 Burlington - Other DBP Reduction Alternatives 5-41
5.2.3.10 Catamount-Bolton Existing System 5-47
5.2.3.11 Catamount-Bolton - Proposed Alternatives 5-50
5.2.3.12 Catamount-Bolton - Other DBP Reduction Alternatives 5-53
5.2.3.13 Champlain Water District - Existing System 5-59
5.2.3.14 Champlain Water District - Proposed Alternatives 5-66
5.2.3.15 Champlain Water District - Other DBP Reduction Options 5-74

TABLE OF CONTENTS (Continued)

Section | Page No.

5.2.3.16 Grand Isle Consolidated Water District - Existing System 5-77
5.2.3.17 Grand Isle Consolidated Water District - Proposed Alternatives 5-80
5.2.3.18 Grand Isle - Other DBP Reduction Alternatives 5-83
5.2.3.19 North Hero - Existing System 5-88
5.2.3.20 North Hero - Proposed Alternatives 5-90
5.2.3.21 North Hero - Other DBP Reduction Alternatives 5-93
5.2.3.22 Proctor - Existing System 5-99
5.2.3.23 Proctor - Proposed Alternatives 5-100
5.2.3.24 Proctor - Other DBP Reduction Alternatives 5-105
5.2.3.25 Readsboro Village - Existing System 5-112
5.2.3.26 Readsboro - Proposed Alternatives 5-114
5.2.3.27 Readsboro - Other DBP Reduction Alternatives 5-118
5.2.3.28 Richford - Existing System 5-123
5.2.3.29 Richford - Proposed Toolbox System 5-124
5.2.3.30 Richford - Other DBP Reduction Alternatives 5-128
5.2.3.31 Rutland City - Existing System 5-133
5.2.3.32 Rutland - Proposed Alternatives 5-135
5.2.3.33 Rutland - Other DBP Reduction Alternatives 5-138
5.2.3.34 St. Johnsbury - Existing System 5-144
5.2.3.35 St. Johnsbury - Proposed Alternatives 5-147
5.2.3.36 St. Johnsbury - Other DBP Reduction Alternatives 5-150
5.2.3.37 Swanton - Existing System 5-156
5.2.3.38 Swanton - Proposed Alternatives 5-159
5.2.3.39 Swanton - Other DBP Reduction Alternatives 5-160
5.2.3.40 Tri-Town Water District - Existing System 5-168
5.2.3.41 Tri Town Water District - Proposed Alternatives 5-171
5.2.3.42 Tri Town Water District - Other DBP Reduction Alternatives 5-174
5.2.3.43 Vergennes-Panton Water District - Existing System 5-179
5.2.3.44 Vergennes-Panton Water District - Proposed Alternatives 5-181
5.2.3.45 Vergennes-Panton Water District - Other DBP Reduction Alternatives 5-185

5.3 CONCEPTUAL COST COMPARISONS 5-190
5.3.1 Capital Cost Estimates 5-190
5.3.2 Capital Cost Development 192

6.0 CONCLUSIONS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

Table — Page No.

ES-1 Range of DBPs within each System Participant since 2005 to 2009 ..... ES-7
ES-2 20 Year Present Worth Cost Summary - Chlorine and Chloramine with UV Option ES-11
ES-3 20 Year Present Worth Cost Summary - Chloramine with UV Option and Chloramine Only Option ES-12
ES-4 20 Year Annual Cost Summary - Chlorine and Chloramine with UV Option ES-13
ES-5 20 Year Annual Cost Summary - Chloramine with UV and Chloramine Only Option ES-14
1-1 United States Cities that Use Chloramines for Disinfection 1-4
2-1 Utility Participation 2-1
3-1 Maximum Contaminant Level Guidelines and Maximum Contaminant Levels for Disinfection Byproducts 3-2
3-2 Maximum Residual Disinfectant Level Goals (MRDLGS) and Maximum Residual Disinfectant Levels (MRDLS) for Disinfectants 3-3
3-3 Total Organic Carbon Removal Requirements 3-4
3-4 Stage 2 DBPR MCLS and MCLGS for Disinfection Byproducts 3-6
3-5 Stage 2 DBPR Compliance Dates 3-7
3-6 BIN Classification for Filtered Systems Under the LT2WSWTR 3-9
3-7 Unfiltered System Requirements under the LT2ESWTR 3-9
3-8 LCR Impacts on ESWTRS and Stage 1 & 2 DBPR Requirements 3-18
3-9 Stage 1&2 DBPR and ESWTR Impacts on LCR Requirements 3-19
3-10 General Relationships Between pH and Effectiveness of Disinfectants Used for Secondary Disinfection 3-21
4-1 Factors Affecting Source Water Quality 4-6
4-2 Lake Nutrient Characteristics 4-9
4-3 Qualitative Assessment of Reservoir Management Techniques 4-17
4-4 General Guidelines on Coagulant Dosages 4-37
4-5 Guidelines on Nature of NOM and Expected Removals by Coagulation 4-39
4-6 Treatment Technologies Summary for DBP Reduction 4-60
4-7 CT Values for 4.0-Log Inactivation of Viruses by Ozone 4-63
4-8 CT Values for 3.0-Log Inactivation of Giardia Cysts by Ozone 4-64
4-9 CT Values for Inactivation of Cryptosporidium by Ozone 4-64
4-10 UV Dose Requirements under the LT2ESWTR (MJ/CM^2) 4-70
4-11 CT Values for 4.0-log Inactivation of Viruses by Chlorine Dioxide 4-77
4-12 CT Values for 3.0-log Inactivation of Giardia Cysts by Chlorine Dioxide 4-77
4-13 CT Values for 4.0-log Inactivation of Viruses by Chloramine 4-82
4-14 CT Values for 3.0-Log Inactivation of Giardia Cysts by Chloramine 4-83
4-15 Odor Thresholds for Chloramine and Chlorine 4-84
4-16 Cancer Risks for Nitrosamines by the State of California Department of Public Health 4-86
4-17 Cancer Risk Levels for Nitrosamines set by the U.S. DHH 4-87
4-18 Summary of Disinfection Technologies 4-92
4-19 Potential Primary Disinfectants 4-93
4-20 Primary/Secondary Disinfectant Combinations and Typical Applications in Water Treatment 4-95

TABLE OF CONTENTS (Continued)

LIST OF TABLES

Table — Page No.

4-21 Summary of Potential Benefits and Adverse Effects Associated with Different Combinations of Primary and Secondary Disinfectants 4-96
4-22 DBPS Associated with Various Combined Oxidation/Disinfection Processes 4-98
4-23 Removal Capabilities of Filtration Processes 4-101
4-24 Removal Capabilities of MF and UF Membrane Filtration Processes 4-102
4-25 Typical Disinfectant Residual Comparison Based on Bulk Decay Rate 4-108
4-26 Groundwater Favorability Summary Sand-and-Gravel Aquifers 4-120
4-27 Groundwater Favorability Summary Fractures Bedrock Aquifers 4-123
5-1 Synthetic Raw Water Quality 5-4
5-2 Required CT Values for Giardia Inactivation at Temperature of 0.5 C for Free Chlorine 5-5
5-3 Required CT Values Giardia Inactivation at a Temperature of 10 C for Free Chlorine 5-5
5-4 Virus Inactivation Requirements at pH 6-9 for Free Chlorine 5-5
5-5 Credited and Required Inactivation Values per Treatment System 5-6
5-6 Alburgh System Summary 5-13
5-7 Proposed Alternatives for Alburgh 5-15
5-8 Other DBP Reduction Options for Alburgh 5-16
5-9 Bennington System Summary 5-25
5-10 Proposed Alternatives for Bennington 5-29
5-11 Other DBP Reduction Options for Bennington 5-31
5-12 Burlington System Summary 5-37
5-13 Proposed Alternatives for Burlington 5-41
5-14 Other DBP Reduction Options for Burlington 5-42
5-15 Catamount-Bolton System Summary 5-49
5-16 Proposed Alternatives for Catamount-Bolton 5-53
5-17 Other DBP Reduction Options for Catamount-Bolton 5-54
5-18 Champlain Water District System Summary 5-60
5-19 Champlain Water District's System Tanks 63
5-20 Comparison of GAC and Nanofiltration for CWD 5-73
5-21 Proposed Alternatives for Champlain Water District 5-74
5-22 Other DBP Reduction Options for Champlain Water District 5-77
5-23 Statistics for TOC Removed through the WTF from October 2008 to December 2009 5-78
5-24 Grand Isle Consolidated Water District System Summary 5-79
5-25 Proposed Alternatives for Grand Isle 5-83
5-26 Other DBP Reduction Options for Grand Isle 5-83
5-27 North Hero System Summary 5-89
5-28 Proposed Alternatives for North Hero 5-93
5-29 Other DBP Reduction Options for North Hero 5-94
5-30 Historical Disinfection Byproduct Levels at WTF and Distribution System 5-100
5-31 Proctor System Summary 5-101
5-32 Proposed Alternatives for Proctor 5-105
5-33 Other DBP Reduction Alternatives for Proctor 5-106
5-34 Readsboro System Summary 5-113
5-35 Proposed Alternatives for Readsboro 5-115

TABLE OF CONTENTS (Continued)

LIST OF TABLES

Table | Page No.

5-36 Readsboro - Other DBP Reduction Alternatives 5-118
5-37 Richford System Summary 5-124
5-38 Proposed Alternatives for Richford 5-128
5-39 Richford Other DBP Reduction Alternatives 5-128
5-40 Rutland System Summary 5-134
5-41 Proposed Alternatives for Rutland 5-138
5-42 Rutland - Other DBP Reduction Alternatives 5-139
5-43 St Johnsbury System Summary 5-146
5-44 Proposed Alternatives for St. Johnsbury 5-150
5-45 St. Johnsbury - Other DBP Reduction Alternatives 5-151
5-46 Swanton System Summary 5-158
5-47 Proposed Alternatives for Swanton 5-160
5-48 Swanton - Other DBP Reduction Alternatives 5-163
5-49 Tri-Town Water District System Summary 5-170
5-50 Proposed Alternatives for Tri Town Water District 5-174
5-51 Tri Town Water District - Other DBP Reduction Alternatives 5-174
5-52 Vergennes-Panton Water District System Summary 5-181
5-53 Proposed Alternatives for Vergennes-Panton Water District 5-185
5-54 Vergennes-Panton Water District - Other DBP Reduction Alternatives 5-185
5-55 Site Summary of Proposed Treatment to Address DBPs 5-191
5-56 Chlorine Option Capital Cost Summary 5-200
5-57 Chloramine Option Capital Cost Summary 5-202
5-58 Operation and Maintenance Summary 5-203
5-59 Labor Hours Summary 5-208
5-60 Present Worth Cost Summary (Chlorine and Chloramine with UV Option) 5-212
5-61 Present Worth Cost Summary (Chloramine with UV and Chloramine Only Option) 5-213
5-62 Annualized Cost Summary (Chlorine and Chloramine with UV Option) 5-214
5-63 Annualized Cost Summary (Chloramine UV and Chloramine Only Option) 5-218
5-64 Present Worth Breakdown Summary 5-222

LIST OF FIGURES

Figure — Page No.

ES-1 Present Worth of Most Likely Treatment Options ES-15
ES-2 Total Annual Cost of Most Likely Treatment Options ES-16
ES-3 Total Annual Cost per Person of Most Likely Treatment Options ES-17
ES-4 Total Annual Cost per Household of Most Likely Treatment Options Es-18
2-1 Location Map for the 15 Participating Utilities 2-3
2-2 Project Flow Chart 2-4
4-1 Impact of Raw Water TOC on TOC Removal by Alum Coagulation/Filtration 4-35
4-2 Impact of Coagulation pH on TOC Removal by Alum Coagulation/Filtration 4-36
4-3 Chemistry of Chloramines 4-81
4-4 Chlorine Breakpoint Curve 4-88
4-5 Lead Scales Likely to Form on Lead Containing Services and Other Lead Containing Appurtenances (source: Lytle and Schock, JAWWA November, 2005) 4-111
4-6 Surficial Geology - Alburgh, North Hero, Swanton and Grand Isle CWD ..... Map Pocket
4-7 Surficial Geology - Bennington 4-130
4-8 Surficial Geology - Burlington, Champlain Water District ..... Map Pocket
4-9 Surficial Geology - Catamount Bolton 4-134
4-7a West Milton Parcel 4-137
4-10 Surficial Geology - Proctor and Rutland City 4-140
4-11 Surficial Geology - Readsboro 4-141
4-12 Surficial Geology - Richford 4-142
4-13 Surficial Geology - St. Johnsbury 4-144
4-14 Surficial Geology - TriTown Water District 4-146
4-15 Surficial Geology - Vergennes-Panton 4-148
5-1 Process for Developing Toolbox of Options for Water Treatment Facilities 5-9
5-2 Alburgh TTHM Levels (2006-2009) 5-17
5-3 Alburgh HAA5 Levels (2006-2009) 5-18
5-4A Alburgh Village Existing Process Flow Diagram 5-19
5-4B Alburgh Village Updated Process Flow Diagram for Chlorine (Updates In Red) 5-20
5-4C Alburgh Village Updated Process Flow Diagram for Chloramines (Updates In Red) 5-21
5-4D Alburgh Village Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-22
5-5 Bennington TTHM Levels (2006-2009) 5-27
5-6 Bennington HAA5 Levels (2006-2009) 5-28
5-7A Bennington Existing Process Flow Diagram 5-32
5-7B Bennington Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-33
5-7C Bennington Proposed Process Flow Diagram for Chloramine (Updates In Red) 5-34
5-7D Bennington Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-35
5-8 Burlington TTHM Levels (2006-2009) 5-39
5-9 Burlington HAA5 Levels (2006-2009) 5-40
5-10A Burlington Process Flow Diagram 5-43
5-10B Burlington Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-44
5-10C Burlington Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-45
5-10D Burlington Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-46
5-11 Catamount TTHM Levels (2006-2009) 5-51
5-12 Catamount HAA5 Levels (2006-2009) 5-52

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

Figure — Page No.

5-13A Catamount-Bolton Process Existing Flow Diagram 5-55
5-13B Catamount-Bolton Process Proposed Flow Diagram for Chlorine (Updates In Red) 5-56
5-13C Catamount-Bolton Process Proposed Flow Diagram for Chloramines (Updates In Red) 5-57
5-13D Catamount-Bolton Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-58
5-14A Champlain Water District Existing Process Flow Diagram 5-64
5-14B Champlain Water District Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-65
5-15 CWD TTHM Levels NF Alternative (1994-2006) 5-67
5-16 CWD TTHM Levels GAC Alternative (1994-2006) 5-68
5-17 CWD HAA5 Levels NF Alternative (1994-2006) 5-69
5-18 CWD HAA5 Levels GAC Alternative (1994-2006) 5-70
5-19 CWD TTHM Levels with Chloramine (2006-2009) 5-71
5-20 Schematic of MIEX Resin 5-72
5-21 Sampling Results Total Haloacetic Acids (HAA5) 5-75
5-22 Sampling Results Total Trihalomethanes 5-76
5-23 Grand Isle TTHM Levels (2006-2009) 5-81
5-24 Grand Isle HAA5 Levels (2006-2009) 5-82
5-25A Grand Isle Consolidated Water District Existing Process Flow Diagram 5-84
5-25B Grand Isle Consolidated Water District Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-85
5-25C Grand Isle Consolidated Water District Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-86
5-25D Grand Isle CWD Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-87
5-26 North Hero TTHM Levels (2006-2009) 5-91
5-27 North Hero HAA5 Levels (2006-2009) 5-92
5-28A North Hero Existing Process Flow Diagram 5-95
5-28B North Hero Proposed Process Flow Diagram For Chlorine (Updates In Red) 5-96
5-28C North Hero Proposed Process Flow Diagram For Chloramines (Updates In Red) 5-97
5-28D North Hero Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-98
5-29 Proctor TTHM Levels (2006-2009) 5-102
5-30 Proctor HAA5 Levels (2006-2009) 5-103
5-31A Proctor Existing Process Flow Diagram 5-107
5-31B Proctor Alternative No. 1 Process Flow Diagram for Chlorine (Updates In Red) 5-108
5-31C Proctor Alternative No. 2 Process Flow Diagram for Chlorine (Updates In Red) 5-109
5-31D Proctor Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-110
5-31E Proctor Process Flow Diagram for Other DBP Reduction Option (Updates In Red) 5-111
5-32 Readsboro TTHM Levels (2006-2009) 5-116
5-33 Readsboro HAA5 Levels (2006-2009) 5-117
5-34A Readsboro Existing Process Flow Diagram 5-119
5-34B Readsboro Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-120
5-34C Readsboro Process Flow Diagram for Chloramine (Updates In Red) 5-121
5-34D Readsboro Process Flow Diagram for Other DBP Reduction Option (Updates In Red) 5-122

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

Figure — Page No.

5-35 Richford TTHM Levels (2006-2009) 5-126
5-36 Richford HAA5 Levels (2006-2009) 5-127
5-37A Richford Existing Process Flow Diagram 5-129
5-37B Richford Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-130
5-37C Richford Proposed Process Flow Diagram for Chloramine (Updates In Red) 5-131
5-37D Richford Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-132
5-38 Rutland City TTHM Levels (2006-2009) 5-136
5-39 Rutland City HAA5 Levels (2006-2009) 5-137
5-40A Rutland City Existing Process Flow Diagram 5-140
5-40B Rutland City Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-141
5-40C Rutland City Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-142
5-40D Rutland Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-143
5-41 St. Johnsbury TTHM Levels (2006-2009) 5-148
5-42 St. Johnsbury HAA5 Levels (2006-2009) 5-149
5-43A St. Johnsbury Existing Process Flow Diagram 5-152
5-43B St. Johnsbury Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-153
5-43C St .Johnsbury Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-154
5-43D St. Johnsbury Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-155
5-44 Swanton TTHM Levels (2006-2009) 5-161
5-45 Swanton HAA5 Levels (2006-2009) 5-162
5-46A Swanton Existing Process Flow Diagram 5-164
5-46B Swanton Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-165
5-46C Swanton Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-166
5-46D Swanton Proposed Process Flow Diagram for Other DBP Reduction Options 5-167
5-47 TriTown TTHM Levels (2006-2009) 5-172
5-48 TriTown HAA5 Levels (2006-2009) 5-173
5-49A Tri Town Water District Existing Process Flow Diagram 5-175
5-49B Tri Town Water District Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-176
5-49C Tri Town Water District Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-177
5-49D Tri Town Water District Proposed Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-178
5-50 Vergennes-Panton TTHM Levels (2006-2009) 5-182
5-51 Vergennes-Panton HAA5 Levels (2006-2009) 5-183
5-52A Vergennes-Panton Water District Existing Process Flow Diagram 5-186
5-52B Vergennes-Panton Water District Proposed Process Flow Diagram for Chlorine (Updates In Red) 5-187
5-52C Vergennes-Panton Water District Proposed Process Flow Diagram for Chloramines (Updates In Red) 5-188
5-52D Vergennes-Panton Water District Proposed Process Flow Diagram for Other DBP Reduction Options (Updates In Red) 5-189

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

Figure — Page No.

5-53 Present Worth of Most Likely Treatment Options 5-216
5-54 Total Annual Cost of Most Likely Treatment Options 5-217
5-55 Total Annual Cost Per Person of Most Likely Treatment Options 5-218
5-56 Total Annual Cost Per Household of Most Likely Treatment Options 5-219

APPENDICES

Appendix 1 References
Appendix 2 Computer Model Output Summaries
Appendix 3 Operational Data and Survey Information
Appendix 4 Distribution System Disinfection Byproducts Rule Data Over Time
Appendix 5 Consecutive System Disinfection Byproducts Rule Data Over Time
Appendix 6 Distribution Maps
Appendix 7 Concept Site Plans
Appendix 8 Questions and Responses on Draft Report
Appendix 9 Service Line Scale Analysis
Appendix 10 Draft Report Workshop Video (DVD)
Appendix 11 Cost Basis for Capital and O&M

`EXECUTIVE SUMMARY

Water utilities in the State of Vermont currently have to comply with the Stage 1 Disinfection Byproduct Rule (Stage 1 DBPR). Systems having a population served of less than 49,999 people will have to comply with the Stage 2 Disinfection Byproduct Rule (Stage 2 DBPR) by 10/1/2013. Larger systems serving from 50,000 to 99,999 persons such as Champlain Water District, will have to comply with Stage 2 DBPR by 10/1/2012. This rule changes the method in which utilities are regulated in respect to chlorinated disinfection byproducts (DBPs). High levels of chlorinated DBPs have been shown to be associated with elevated cancer risks. Under the current Stage 1 DBPR, distribution sites are sampled for chlorinated DBPs and the results of all the sampling sites are averaged from the previous four sampling quarters, on a running annual average (RAA) basis. The new Stage 2 DBPR requires that the DBP levels are averaged over the past four quarters at each sampling site and compliance is calculated on a locational running annual average (LRAA) basis, taking into account the areas with long detention times, where DBP formation can be the greatest.

Disinfection byproduct formation can be normally described as follows:

$$TOC + Cl_2 \xrightarrow{temp, time, pH} DBP$$

Where: TOC is the total organic carbon in the raw source water
Cl_2 is the applied chlorine to the water for disinfection
DBP is the chlorinated (i.e. halogenated) disinfection byproducts
Temp is the temperature of the water
Time is the contact time in which the chlorine interacts with the TOC
pH is the measure of how acidic or alkaline the water is during the interaction.

In general, the more TOC in the source water, the higher the chlorine dose, the higher the pH, and the longer contact time with free chlorine, the more DBPs will form.

Public water suppliers are mandated by the State of Vermont to provide a primary and secondary disinfectant. Primary disinfection is typically provided as a multiple barrier approach, where more than one technique is applied to remove or inactivate the pathogen. The target pathogens are *Cryptosporidium*, *Giardia*, and Viruses. It is also noted that the State has a *Legionnella* goal of zero. The most typical scheme is some form of filtration followed by a chemical disinfectant, typically chlorine. Following

primary disinfection, public water suppliers are mandated by the State of Vermont to provide a secondary disinfectant prior to entering the distribution system. The goal of the secondary disinfectant is to maintain the integrity of the treated water as it travels through the distribution system and provide safe water at the consumer tap. A measureable residual of secondary disinfectant must be maintained in the entire distribution system. This, in conjunction with regular monitoring of pathogen indicator organisms (Total and *E-Coli*), provides confidence that the water in the distribution is safe for public consumption. Currently, only three disinfectants are approved by the United States Environmental Protection Agency (US EPA) and the State of Vermont as secondary disinfectants that can provide the required residual. Those disinfectants are chlorine, chloramine (monochloramine), and chlorine dioxide. No other disinfectants have been approved as a secondary disinfectant.

Historically, chlorine has been the preferred secondary disinfectant of choice by water suppliers for many reasons such as simplicity, availability, history, cost, etc. Chlorine dioxide historically has not been used as a secondary disinfectant due to a shorter residual life in the distribution system and inability to dose higher levels due to a specific DBP (e.g. chlorite) as well as potential taste and odor issues. Chloramine has been in use since 1930's, historically used in large public supply systems where maintaining a chlorine residual in a distribution system with a long residence time is more difficult. Examples of systems using chloramine are Boston, MA, since 1932 and more recently Washington, DC.

Chlorine forms halogenated DBPs. There are many halogenated DBPs and as such it is difficult to regulate them all individually. Small concentrations of byproducts (dichloramine and trichloramine) can be formed during chloramination depending on how it is applied. If chloramine is applied correctly, these byproducts are minimize and not of concern.

The US EPA selected two DBP groups as the surrogate compounds to regulate DBPs. These surrogate compounds are trihalomethanes (TTHM) and haloacetic acids (HAA5). When these values are reduced, the classical research suggests that other possible DBPs are reduced also.

One common alternative to reduce halogenated DBPs and meet the Stage 1 and 2 DBPR is to switch the distribution system residual disinfectant from chlorine to chloramine, because chloramine will not continue to form chlorinated DBPs during the detention time within the distribution system. In Vermont, only one system, the Champlain Water District, has made the switch from chlorine to chloramine for residual disinfection. Concerned citizens have raised the issue that chloramines are negatively affecting

the health of some of the consumers. The specific complaints were investigated by the Center for Disease Control (CDC). The CDC concluded that there was insufficient evidence to confirm a linkage between the use of chloramine disinfectant and the reported health problems; and they recommended a need for further study as noted: "Not having baseline data, the post hoc analysis is difficult especially since there are strong biases with this data. Information provided by this questionnaire should be utilized to explore the design of future prospective studies eliminating biases and allowing for a baseline understanding of the symptoms that pre-exist in the community." (CDC, 2008).

There is some concern within the scientific community that chloramines produce nitrogen based DBPs, under some conditions, and that they may be more toxic than chlorinated DBPs. It should be noted that chlorination also produces nitrogen based DBPs under the right conditions. These nitrogen based DBPs are currently unregulated in public water supply. Knowledge of how nitrogen based DBPs form is still developing and the real world risk to the public through drinking water, of these nitrogen based DBPs from chloramination or chlorination is still under review. The US EPA is currently undertaking a utility water quality survey as part of the Unregulated Contaminant Monitoring Regulation to determine the occurrence frequency and level of what are believed to be the most likely nitrogen containing DBPs to occur in treated potable water.

In order to evaluate the cost impact to utilities to comply with the Stage 2 DBPR using chlorine or chloramines as the distribution system (secondary) disinfectant, the Vermont Department of Environmental Conservation (DEC) retained AECOM to develop system specific costs for 15 utilities across the state.

AECOM implemented a multi-step process to conduct this evaluation that involved: a kick off meeting, collection of available data from both the systems and the DEC, individual site visits, workshop with stakeholders to review the draft report, solicitation of comments on the draft report, responses to comments, and finalization of the draft report. A kickoff meeting was held in which the project goals were discussed with the stakeholders. An electronic survey was solicited in which each water system could provide water quality and operational data. Most utilities also provided record drawings of their treatment facilities as well as a water distribution map. Supplementing this data, the DEC also provided operational and compliance data (coagulant doses, chlorine residuals, DBP levels, pH values etc.). This information can be found in the Appendices.

Once this data was received, AECOM met with most utilities individually, in a workshop fashion, to discuss the data and proposed treatment alternatives which AECOM was considering based on the information provided to date. AECOM used US EPA's Water Treatment Model along with our historical experience in selecting recommended treatment alternatives. The alternatives were modified based on the input we received during these individual utility workshop meetings. This information was compiled and presented in a draft report. A draft report workshop was held to discuss the results and to solicit more input on the report. A DVD of the draft report workshop is provided in the Appendix 10.

A comment period was provided to the stakeholders. AECOM reviewed the comments, and where appropriate, provided responses and incorporated changes into the report. The individual comments and responses can be found in the Appendix 8. From this input, a final report was produced.

The existing water system was modeled using US EPA's Water Treatment Plant Model (model) V. 2.1. The intention of performing this analysis was to obtain a baseline of performance for each system and compare the expected performance in DBP precursor removal, DBP reduction, and where applicable areas of possible improvements. In addition to obtaining a baseline, the model may provide an indication to a poorly performing existing facility. The model was run based on the average water quality provided by each system and where available, was taken from the synthetic water quality database as discussed below. The model was calibrated to the typical summer DBP levels.

The results of the model runs are provided in Appendix 2 - Computer Model Output Summaries. For each community the historical DBP RAA and reduced DBP RAA with the proposed treatment in operation during the peak DBP months are provided in figure format in Section 5.0 of the report. The location RAA doesn't exist at this time and cannot be shown.

Based on data provided by the DEC and each utility, AECOM developed DBP reduction alternatives using chlorine for secondary disinfection and using chloramine for secondary disinfection, for each system, except for the Champlain Water District which already practices chloramination for secondary disinfection. The alternatives that were considered for implementing chlorine and chloramine approaches to disinfection are listed below.

Chlorine: (Free chlorine is used for both primary and secondary disinfection)

1. Stop or minimize pre-chlorination prior to filtration
2. Optimize the coagulant dose (enhanced coagulation) to remove as much TOC as possible prior to adding chlorine Improve clarification prior to filtration to allow for more flexibility in coagulant dosing
3. Add additional treatment processes, to specifically remove TOC:
    a. ion exchange,
    b. granular activated carbon adsorption, or
    c. nanofiltration
4. Minimize the chlorine contact time to the extent possible, without sacrificing primary disinfection
5. Switch to sources waters (groundwater wells) with lower TOC levels or TOC that is less reactive with chlorine

Chloramine:

1. Use free chlorine for primary disinfection and use chloramine as a secondary disinfectant
2. Use UV for primary disinfection and free chlorine only for primary disinfection of viruses, and use chloramine as secondary disinfectant
3. Use UV and chloramine for primary disinfection, and use chloramine as secondary disinfectant

AECOM reviewed the existing compliance with Stage 1 DBP requirements of each system. It is important to note that there was very little historical TOC information for raw or treated waters available from most of the systems, on which to evaluate existing treatment or base treatment considerations for compliance. Starting with the options listed above, AECOM used empirical experience, knowledge of water treatment processes and modeling of site specific (each system) water quality and treatment conditions, where available, to identify the most likely process alternatives that could achieve compliance with Stage 2 DBPR requirements using either disinfectant approach. The target level for compliance was assumed to be a locational running annual average of 64 µg/L for TTHM and 48 µg/L for HAA5, which would be 20% below the regulated MCL of 80 µg/L for TTHM and 60 µg/L for HAA5. Setting the goal for compliance at 20% less than the regulated level provides an operational margin of safety for compliance. Most utilities set compliance goals for DBPs in this fashion. In some cases more than one possible treatment alternative for achieving compliance was developed and carried forward in the cost analysis.

AECOM assumed a finished water TOC of 1.5 mg/L or less prior to primary disinfection could be required to achieve compliance when using free chlorine for primary and secondary disinfection. For large utilities with more wide spread distribution, such as CWD, a finished water TOC target of 1.0 mg/L was assumed prior to primary disinfection.

AECOM also provided for each community, a list of other less costly alternatives for compliance that may also be viable for each system under the header. "Other DBP Reduction Alternatives" in Section 5.0. However, these options were listed in this category for one or more of the following reasons:

1. Not enough specific site information to confirm performance
2. Not enough historical performance information in Vermont and/or the United States to gauge the success of the option (i.e. the theory suggests the process or method will work)
3. Operational and/or constraining issues (e.g. DBP formation, residual life, etc)

AECOM recommended well sources as a primary option for three communities:

1. Proctor, VT
2. Rutland, VT
3. Readsboro, VT

The underlying assumption in using well water to achieve DBP compliance is that the well water has lower TOC levels and/or TOC that is less reactive with chlorine, which is often the case. In addition, these communities have smaller demands and were considered good candidates for wells whereas utilities with higher demands would require more wells and thus the uncertainty of well applicability for meeting the Stage 1 and 2 DBPR requirements increases. However, wells may be a local alternative to some communities, supplying water in regions of the distribution system which are a long distance away from the water treatment plant (i.e. high water age/residence time).

As a reference, Table ES-1 provides the historical chlorinated DBPs measured in each of the 15 communities which are part of this study.

TABLE ES-1
RANGE OF DBPS WITHIN EACH SYSTEM PARTICIPANT SINCE 2005 TO 2009

	CWD-Chlorine	CWD-Chloramine	Burlington	Bennington	Rutland	Vergennes-Panton	Alburgh	Catamount-Bolton	Grand Isle CWD	North Hero	Proctor	Readsboro	Richford	St. Johnsbury	Swanton	Tri-Town
TTHM																
Min.	ND	10.0	15.7	10	12.2	11.3	15.2	ND	39.7	34.7	28.8	6	15.7	3.3	16	35.6
Max.	100	40.8	118	174	99.0	113	91.7	277	92.5	167	211	181	148	137	131	125
HAA5																
Min.	ND	15.2	24.6	6.5	25	21.7	24.1	ND	29	17	25.5	ND	12	2.3	ND	27
Max.	91	52.3	86.1	258	117	125	72.2	352	123	150	361	153	120	116	71	116

Notes:
1. CWD chlorine dataset was from 1990 to 2005; chloramines dataset from 2006 to 2009.
2. ND = Non-detect

In our cost estimating, AECOM has provided site specific concept level costs for each alternative approach. In many cases, recommendations for each utility consist of relatively small changes to existing practices and also some more significant modifications, for example, adding a new unit process or replacing an existing process (refer to Table 5-55 for proposed treatment schemes). The utility may be able to meet regulations by making the small changes only, which could consist of optimization of the current treatment process as well as optimization of the distribution operation, and therefore might not incur the full costs which are presented here for chlorine based disinfection approach. We have also provided the cost for chloramine and also chloramine with UV as separate approaches as discussed in Section 5.0. In addition, in Section 5.0 we have provided a discussion of the manpower requirements as well as the costs in the Appendices. It should be noted that some alternatives are more complex operationally and would require more operational skill than is typically required for small systems. Thus more training and labor would be required, and AECOM attempted to estimate these additional staffing and labor requirements.

For each community and each alternative, a 20 year present worth life cycle cost is presented in Table ES – 2 and Table ES – 3. Both the capital and present worth O&M costs are detailed for each community based on the treatment options developed in Section 5-2 and the cost assumptions discussed in Section 5-3. The present worth (sometimes referred to as net present value) is the amount that a future sum of money is worth today, in today's dollars and assuming a fixed period and interest rate (for this project, 20 years and 2.7 %, respectively). Present worth therefore is a useful way of comparing alternatives from a financial perspective. Table ES – 2 compares the present worth summaries of the chlorine approach to the approach using chloramines with UV treatment, and provides the total cost for both approaches. Table ES – 3 compares the present worth summaries of the chloramines approach, with and without UV, and provides the total costs.

Using another approach, capital and O&M costs are presented on an annual basis and shown in Tables ES-4 and ES-5. This approach assumes that the cost of the project construction and the annual cost of operations would be spread out over 20 years and paid for, in annual increments, through the user base. A 4.4 % interest rate on the borrowed capital was applied to construction cost, and the annualized costs were normalized to population by showing the costs per person and per household (assuming 3 persons per household) for each community. Graphical presentations of the total present worth, total annualized cost, cost per person, and cost per household for each of the communities are provided in Figure ES – 1 through Figure ES – 4. For the free chlorine options, only the treatment alternatives considered most likely to enable compliance with DBP limits (as described in Section 5-3) are included in the Figure ES – 1 through Figure ES – 4. In other words, if an overall compliance strategy at a particular facility included some relatively minor changes (optimizing coagulation for example) plus some major changes such as a new unit process, it was assumed for the purposes of developing the total costs that both minor and major changes would be needed, thus providing the most conservative cost estimates and most conservative treatment approaches. Details of the methodologies used for both the present worth and annual costs are included in Section 5.0.

A few important conclusions can be drawn from this study.

1. Secondary disinfection with chloramine to achieve DBP compliance is the least expensive approach on a total present worth basis considering both capital and O&M costs.
2. UV, if required to achieve DBP compliance while using chloramine for secondary disinfection, increases the both the capital and O&M cost of a chloramine approach.

3. Secondary disinfection using chlorine to achieve DBP compliance is a significantly more expensive approach.
4. The total capital cost, annualized over a 20 year basis, of implementing chlorine as a secondary disinfection approach for all of the systems evaluated would be approximately $4.6 MM compared to only $348,000 for a chloramine approach with UV, or compared to $118,000 for a chloramine approach without UV.
5. Considering capital cost and annual operating costs, the total per household cost of implementing a chlorine secondary disinfection approach averages $155/yr but ranges widely according to system size with small systems having the larger cost impact; while a chloramine approach with UV would have an average per household cost of approximately $16/yr; and an even lower average per household cost of about $7/yr, with a chloramine approach without UV. For chloramine approaches, as for chlorine approaches, there is a greater cost impact on smaller systems.

It should be understood that the information and costs provided in this report are based on best practices, both from an engineering and operational perspective, as well as a limited dataset provided from each utility and the DEC. The accuracy of engineering construction costs estimates is proportional to the level of design detail. Accuracy is accounted for by use of contingency factors. AECOM uses the industry standard and widely accepted American Association of Cost Estimating (AACE) classification system to assign contingency. According to the AACE standard, preliminary estimates used for feasibility and conceptual design purposes are considered a Class 4 estimate, where Class 5 is for the most preliminary designs and Class 1 for the most complete. A Class 4 estimate carries a -30 to + 50 range of accuracy and is based on the use of factors, ratios, and other parametric techniques, as well as actual prices for major items. A contingency of 30% has been assigned to the direct capital cost estimates. It must be emphasized that Class 4 estimates are not inclusive but are structured to capture the major cost elements of the various treatment alternatives. Thus, costs presented in this report should not be considered budgetary, but should be considered order of magnitude and be used as comparative when evaluating each alternative. In order to confirm the values provided in this document, pilot studies and site exploration are necessary to validate system performance and associated costs as well as further engineering evaluations for compliance with existing regulations.

Operations and maintenance costs were determined by estimating chemical dosages, power costs, and manpower requirements. If applicable and when available, replacement costs for equipment including membranes, media, UV lamps, and cartridge filters were also considered and provided by equipment vendors. Power costs were estimated assuming a rate of $0.13/kW-hr, and a loaded rate of approximately $29/hr was applied for labor requirements. Chemical and power usage was based on average community flows.

Please note that AECOM's scope for this project only covers evaluation of alternatives for compliance with the requirements of the Stage 2 DPBR, using either chlorine or chloramine for secondary disinfection, and to provide cost estimates for the DEC and Legislature to consider in their decision making process. Issues of possible health concerns for chlorine or chloramine were not AECOM's purview. Further, AECOM, makes no recommendation as to which disinfectant approach to use.

TABLE ES-2
20-YEAR PRESENT WORTH COST SUMMARY-CHLORINE AND CHLORAMINE WITH UV OPTIONS

Treatment Site	Pop.	Present Worth (Chlorine)			Present Worth (Chloramine with UV)		
		Capital	O&M	Total	Capital	O&M	Total
Alburgh (DAF)	578	$ 1,793,000	$ 2,243,000	$ 4,036,000	$ 159,000	$ 421,000	$ 580,000
Burlington (DAF)	38,889	4,013,000	2,711,000	6,724,000	675,000	857,000	1,532,000
Bennington (GAC)	12,000	3,301,000	5,450,000	8,751,000	513,000	729,000	1,242,000
Catamount-Bolton (UV)	850	166,000	429,000	595,000	145,000	412,000	557,000
Champlain (NF)	68,923	27,337,000	25,591,000	52,928,000	*N/A - Already practicing chloramine disinfection*		
Champlain (GAC)		21,127,000	32,720,000	53,847,000			
Proctor (DAF,UV)	2,200	1,689,000	3,583,000	5,272,000	259,000	459,000	718,000
Proctor (Well)		1,926,000	1,295,000	3,221,000			
Grand Isle (GAC)	1,113	801,000	692,000	1,493,000	194,000	437,000	631,000
North Hero (GAC)	600	579,000	1,410,000	1,989,000	194,000	437,000	631,000
Readsboro (GAC)	440	187,000	695,000	882,000	145,000	412,000	557,000
Readsboro (Well)		1,925,000	463,000	2,388,000			
Richford (GAC)	1,700	660,000	593,000	1,253,000	156,000	423,000	579,000
Rutland City (GAC)	18,500	7,470,000	17,634,000	25,104,000	672,000	779,000	1,451,000
Rutland City (MIEX)		6,075,000	2,441,000	8,516,000			
Rutland City (Well)		5,742,000	1,959,000	7,701,000			
St Johnsbury (GAC)	3,800	3,285,000	2,791,000	6,076,000	511,000	716,000	1,227,000
Swanton (DAF,GAC)	3,500	4,011,000	4,575,000	8,586,000	308,000	463,000	771,000
Tri-Town (GAC)	3,500	1,561,000	1,673,000	3,234,000	369,000	525,000	894,000
Vergennes-Panton (NF)	5,000	5,306,000	2,932,000	8,238,000	308,000	461,000	769,000
TOTAL[(1)]	**161,593**	**60,764,000**	**57,809,000**	**118,573,000**	**4,608,000**	**7,531,000**	**12,139,000**

[(1)] Chlorine Option totals only include most likely treatment option, NF for the Champlain Water District, MIEX for Rutland City, and GAC (non-well) options for Proctor and Readsboro

Notes: DAF = Dissolved Air Flotation followed by sand filtration
GAC = Post Granular Activated Carbon Absorption after filtration
UV = Ultraviolet Disinfection after filtration
NF = Nanofiltration after sand filtration
MIEX = Magnetic Ion Exchange Resin prior to filtration
Well = New or supplemental water source

TABLE ES-3
20-YEAR PRESENT WORTH COST SUMMARY-CHLORAMINE WITH UV AND CHLORAMINE ONLY OPTIONS

Treatment Site	Pop.	Present Worth (Chloramine with UV)			Present Worth (Chloramine Only)		
		Capital	O&M	Total	Capital	O&M	Total
Alburgh	578	$ 159,000	$ 421,000	$ 580,000	$ 58,000	$ 192,000	$ 250,000
Burlington	38,889	675,000	857,000	1,532,000	82,000	325,000	407,000
Bennington	12,000	513,000	729,000	1,242,000	74,000	228,000	302,000
Catamount-Bolton	850	145,000	412,000	557,000	56,000	191,000	247,000
Champlain[1]	68,923	*N/A - Already practicing chloramine disinfection*			641,000	1,041,000	1,682,000
Proctor	2,200	259,000	459,000	718,000	81,000	199,000	280,000
Grand Isle	1,113	194,000	437,000	631,000	60,000	193,000	253,000
North Hero	600	194,000	437,000	631,000	60,000	191,000	251,000
Readsboro	440	145,000	412,000	557,000	56,000	190,000	246,000
Richford	1,700	156,000	423,000	579,000	57,000	193,000	250,000
Rutland City	18,500	672,000	779,000	1,451,000	81,000	245,000	326,000
St Johnsbury	3,800	511,000	716,000	1,227,000	73,000	216,000	289,000
Swanton	3,500	308,000	463,000	771,000	63,000	204,000	267,000
Tri-Town	3,500	369,000	525,000	894,000	68,000	206,000	274,000
Vergennes-Panton	5,000	308,000	461,000	769,000	63,000	203,000	266,000
TOTAL	**161,593**	**4,608,000**	**7,531,000**	**12,139,000**	**1,573,000**	**4,017,000**	**5,590,000**

[1] Chloramine Option Capital and O&M costs based on existing information provided by the Champlain Water District

TABLE ES-4
20-YEAR ANNUAL COST SUMMARY- CHLORINE AND CHLORAMINE WITH UV OPTIONS

ent Site	Pop.	Annualized Cost (Chlorine)					Annualized Cost (Chloramine with UV)				
		Capital	O&M	Total	Per Person	Per Household[1]	Capital	O&M	Total	Per Person	P Hous
h (DAF)	578	$ 135,000	$ 147,000	$ 282,000	$ 488	$ 1,467	$ 12,000	$ 28,000	$ 40,000	$ 69	$
ton (DAF)	38,889	302,000	177,000	479,000	12	37	51,000	56,000	107,000	3	
gton (GAC)	12,000	248,000	356,000	604,000	50	151	39,000	48,000	87,000	7	
unt-Bolton (UV)	850	12,000	28,000	40,000	47	143	11,000	27,000	38,000	45	
lain (NF)[1]	68,923	2,058,000	1,673,000	3,731,000	54	162	*N/A - Already practicing chloramine disinfection*				
lain (GAC)[1]		1,590,000	2,139,000	3,729,000	54	162					
(DAF,UV)	2,200	127,000	234,000	361,000	164	493	19,000	30,000	49,000	22	
(Well)		145,000	85,000	230,000	105	313					
sle (GAC)	1,113	60,000	45,000	105,000	94	284	15,000	29,000	44,000	40	
Hero (GAC)	600	44,000	92,000	136,000	227	679	15,000	29,000	44,000	73	
oro (GAC)	440	14,000	45,000	59,000	134	408	11,000	27,000	38,000	86	
oro (Well)		145,000	30,000	175,000	398	1,200					
rd (GAC)	1,700	50,000	39,000	89,000	52	156	12,000	28,000	40,000	24	
d City (GAC)	18,500	562,000	1,153,000	1,715,000	93	278	51,000	51,000	102,000	6	
d City (MIEX)		457,000	160,000	617,000	33	100					
d City (Well)		432,000	128,000	560,000	30	91					
sbury (GAC)	3,800	247,000	182,000	429,000	113	339	38,000	47,000	85,000	22	
n (DAF,GAC)	3,500	302,000	299,000	601,000	172	515	23,000	30,000	53,000	15	
vn (GAC)	3,500	117,000	109,000	226,000	65	195	28,000	34,000	62,000	18	
nes-Panton (NF)	5,000	399,000	192,000	591,000	118	355	23,000	30,000	53,000	11	
L[2]	**161,593**	**4,572,000**	**3,778,000**	**8,350,000**	**52**	**155**	**348,000**	**494,000**	**842,000**	**5**	

hold represents three persons

ne Option totals only include most likely treatment option, NF for the Champlain Water District, MIEX for Rutland City, and GAC (non-well) options for Proctor and Readsboro

DAF = Dissolved Air Flotation followed by sand filtration
GAC = Post Granular Activated Carbon Absorption after filtration
UV = Ultraviolet Disinfection after filtration
NF = Nanofiltration after sand filtration
MIEX = Magnetic Ion Exchange Resin prior to filtration
Well = New or supplemental water source

TABLE ES-5
20-YEAR ANNUAL COST SUMMARY- CHLORAMINE WITH UV AND CHLORAMINE ONLY OPTIONS

ent Site	Pop.	Annualized Cost (Chloramine with UV)					Annualized Cost (Chloramine Only)				
		Capital	O&M	Total	Per Person	Per Household[1]	Capital	O&M	Total	Per Person	P Hous
h	578	$ 12,000	$ 28,000	$ 40,000	$ 69	$ 206	$ 4,000	$ 13,000	$ 17,000	$ 29	$
ton	38,889	51,000	56,000	107,000	3	8	6,000	21,000	27,000	1	
gton	12,000	39,000	48,000	87,000	7	22	6,000	15,000	21,000	2	
unt-Bolton	850	11,000	27,000	38,000	45	134	4,000	12,000	16,000	19	
lain[2]	68,923	*N/A - Already practicing chloramine disinfection*					48,249	68,045	116,294	2	
	2,200	19,000	30,000	49,000	22	68	6,000	13,000	19,000	9	
sle	1,113	15,000	29,000	44,000	40	116	5,000	13,000	18,000	16	
Iero	600	15,000	29,000	44,000	73	216	5,000	12,000	17,000	28	
oro	440	11,000	27,000	38,000	86	259	4,000	12,000	16,000	36	
rd	1,700	12,000	28,000	40,000	24	70	4,000	13,000	17,000	10	
l City	18,500	51,000	51,000	102,000	6	16	6,000	16,000	22,000	1	
sbury	3,800	38,000	47,000	85,000	22	67	5,000	14,000	19,000	5	
n	3,500	23,000	30,000	53,000	15	46	5,000	13,000	18,000	5	
vn	3,500	28,000	34,000	62,000	18	53	5,000	13,000	18,000	5	
nes-Panton	5,000	23,000	30,000	53,000	11	32	5,000	13,000	18,000	4	
L	**161,593**	**348,000**	**494,000**	**842,000**	**5**	**16**	**118,000**	**261,000**	**379,000**	**2**	

hold represents three persons

mine Option Capital and O&M costs based on existing information provided by the Champlain Water District

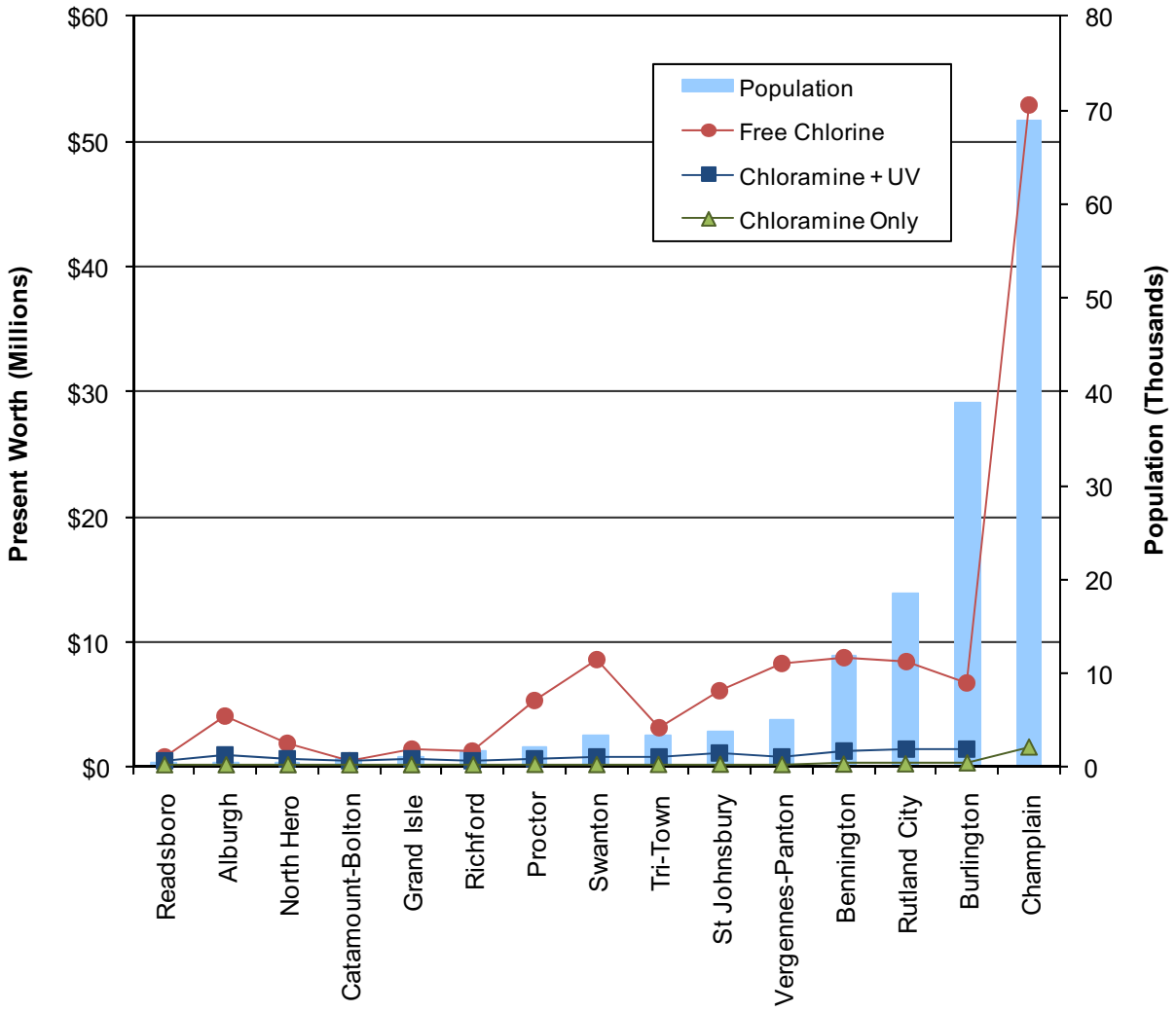


FIGURE ES-1:
PRESENT WORTH OF MOST LIKELY TREATMENT OPTIONS

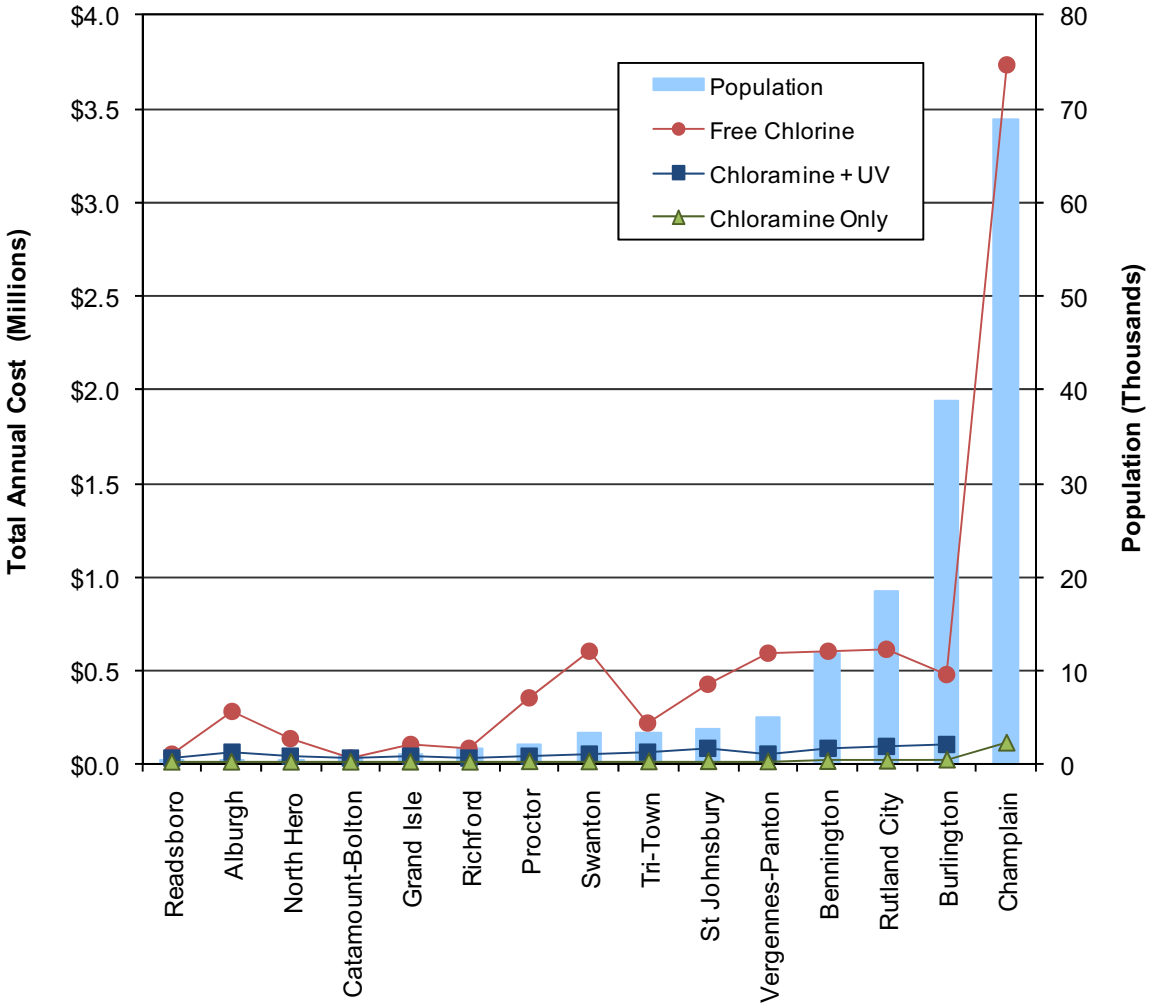


FIGURE ES-2:
TOTAL ANNUAL COST OF MOST LIKELY TREATMENT OPTIONS

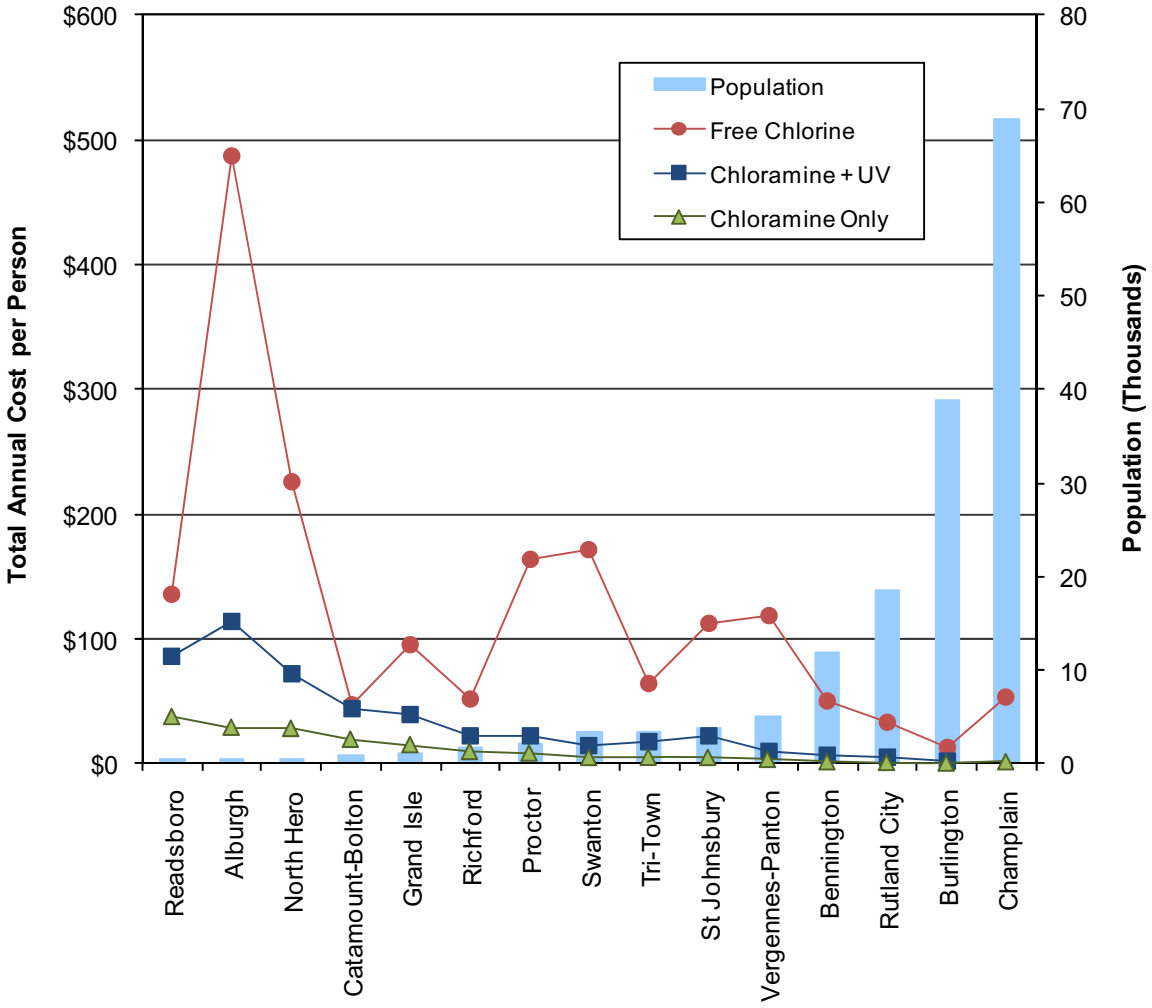


FIGURE ES-3:
TOTAL ANNUAL COST PER PERSON OF MOST LIKELY TREATMENT OPTIONS

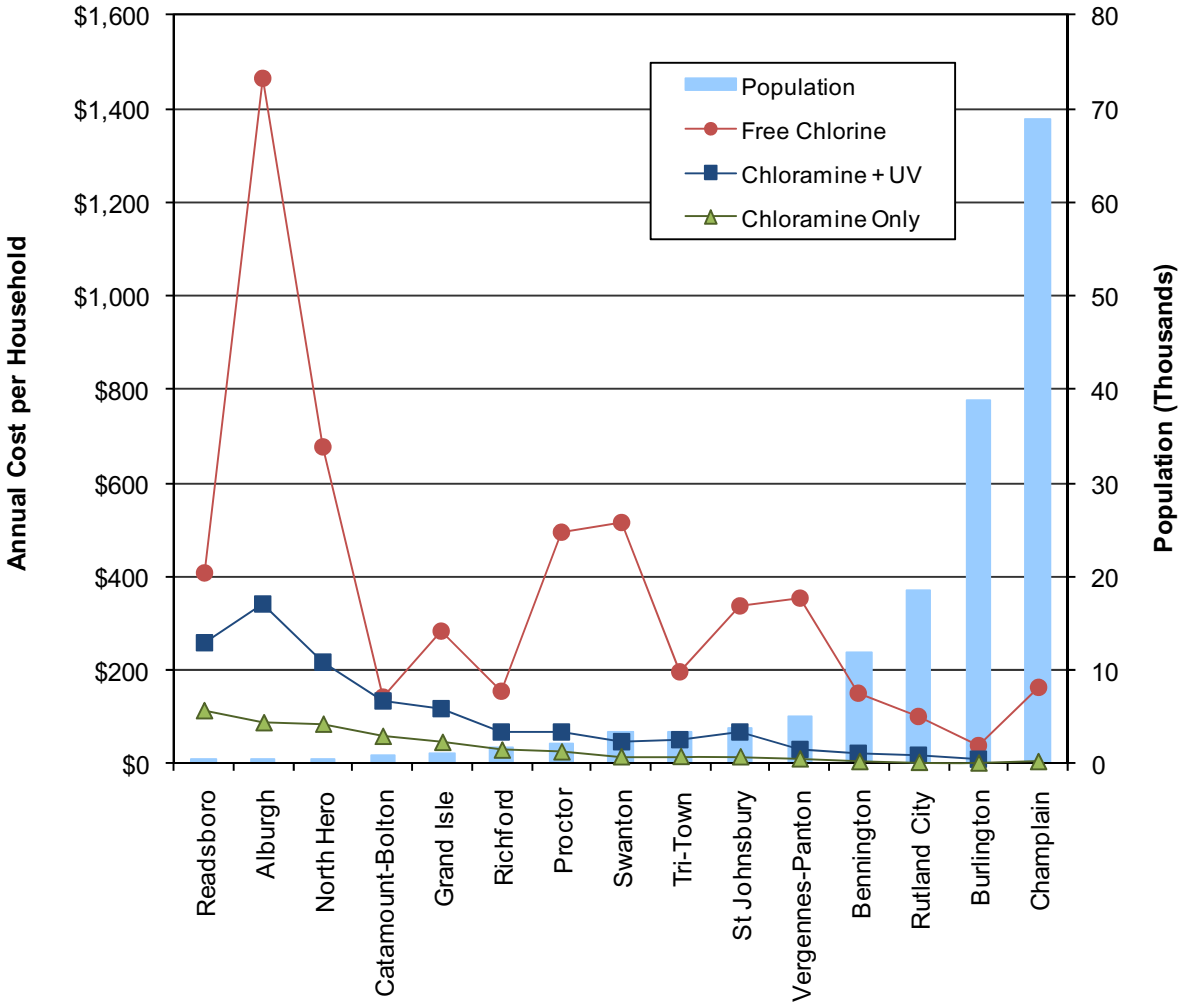


FIGURE ES-4:
TOTAL ANNUAL COST PER HOUSEHOLD OF MOST LIKELY TREATMENT OPTIONS

SECTION 1.0 - INTRODUCTION AND BACKGROUND

1.1 INTRODUCTION

This report is a result of a Request for Proposal (RFP) dated July 29, 2009 published by the Vermont Department of Environmental Conservation (DEC). The RFP called for an Engineering Feasibility Study on the costs of treatment options for reducing disinfection byproducts (DBPs) in 15 public drinking water systems (PWSs) located in the State of Vermont. This RFP was a result of a recently enacted law (May 2009) by the Vermont General Assembly to obtain an impartial analysis and engineering study, of the costs and treatment options available to PWSs in order to comply with the Stage 1 and Stage 2 Disinfectants and Disinfection Byproducts Rule (DBPR). The objective of the study was to identify and assess the appropriate treatment options that will enable each of the stated 15 PWSs to comply with the Stage 1 and Stage 2 DBPR simultaneously with the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR), the Lead and Copper Rule (LCR), the Groundwater Rule (GWR), and the Total Coliform Rule (TCR).

AECOM was selected to provide engineering services in accordance with the RFP and entered into a contract on October 1, 2009 with the DEC. This report outlines the treatment alternatives for each of the 15 PWSs and summarizes the results of the engineering evaluations based on the findings and inputs from the kick-off workshop, site visits, data review, and modeling. An overview of the need for disinfection, DBP formation, and the current strategies for reducing DBP formation are also discussed in this report.

1.2 BACKGROUND

The United States Environmental Protection Agency (EPA) requires PWSs using surface water or groundwater under the direct influence of surface water (GWUDI) as their source water, to treat the finished water prior to delivery to their customers against microbial contaminants (i.e. Coliform Bacteria, *Giardia* Cysts, Virus, and *Cryptosporidium*). This is commonly referred to as primary disinfection. In addition, PWSs are required to maintain a residual disinfectant throughout the water distribution system (referred to as secondary disinfection) to control bacterial re-growth. PWSs have several options or methods to remove these microbial contaminants such as conventional treatment (coagulation, flocculation, clarification, and filtration), direct filtration, and membrane filtration, in conjunction with disinfectants such as chlorine, chloramine, chlorine dioxide, ozone and ultraviolet light (UV).

The use of chlorine for disinfection by United States municipal drinking water systems has been routinely practiced since the early 1900's. Prior to treating water with chlorine, thousands of people died from outbreaks of cholera, typhoid fever, dysentery and hepatitis A. Worldwide, significant strides in public health and the quality of life are directly linked to the adoption of drinking water chlorination.

Chlorine is a very effective, relatively easy to use, and cost effective means of providing primary and secondary disinfection of public drinking water. In addition, chlorine is a powerful oxidant that can be used to remove taste and odors as well as iron and manganese. Equally important is the fact that only chlorine based chemicals are able to provide disinfectant residuals to prevent microbial re-growth and help protect treated water throughout the distribution system.

A disadvantage of using chlorine as a disinfectant is that it reacts with natural organic matter (NOM) contained in source water to form DBPs. Although chlorine disinfection has proven to provide public health protection from microbiological contaminants, concerns arose from possible health effects associated with certain by-products created when surface waters were disinfected with chlorine. Thus the health risk has shifted from an acute concern to a chronic concern.

One of the subgroups of DBPs formed when NOM reacts with chlorine are Trihalomethanes (THMs). The most common and currently regulated THMs are chloroform, bromodichloromethane, chlorodibromomethane, and bromoform. Haloacetic acids are another subgroup of DBPs. The five most common and currently regulated haloacetic acids are monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, bromoacetic acid, and dibromoacetic acid. These five haloacetic acids are commonly referred to as HAA5. The US EPA has selected to use TTHM and HAA5 as a surrogate parameter for regulating all DBP's as noted in the following quote form the Stage 2 DBPR, Page 391: "these two groups of DBP's [TTHM and HAA5] act as indicators for the various byproducts that are present in water disinfected with chlorine or chloramine. This means that concentrations of TTHM and HAA5 are monitored for compliance, but their presence in drinking water is representative of many other chlorinated DBHP's that may also occur in water; thus a reduction in TTHM and HAA5 generally indicates an overall reduction of DBP's."

New information since 1993 added epidemiological evidence of a link between chlorinated disinfection byproducts (CDBP) and bladder, colon or rectal cancer (Marrett and King [1995], Cantor, et al., [1998]). Studies have also linked THMs and spontaneous abortion, stillbirth and pre-term delivery. A study by Swan, et al., (1998) provided evidence of a link between spontaneous abortion and the THM compound bromodichloromethane. In response, the EPA on December 16, 1998, finalized the Stage 1 DBPR that established maximum contaminant level goals (MCLGs) and maximum contaminant levels (MCLs) for total THMs (the sum of the concentrations of bromodichloromethane, dibromochloromethane, bromoform, and chloroform) and HAA5 (the sum of the concentrations of monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid). Note that the link between chlorinated DBP's and health effects is not clear (AwwaRF, 2005). In promulgating the Stage 2 DBPR, EPA notes " evaluation of the best available studies, particularly epidemiology studies is that they do not support a conclusion at this time as to whether exposure to chlorinated drinking or disinfection byproducts cause adverse development and reproductive health effects, but do provide an indication of a potential health concern that warrants incremental regulatory action beyond the Stage 1 DBPR".

The use of chloramine for disinfecting drinking water has been in practice before World War II. Using chloramine as a secondary disinfectant for drinking water systems is becoming more common since disinfection with chloramine allows systems to maintain the required detectable residual throughout the distribution system while simultaneously controlling regulated disinfection byproducts (TTHMs, and HAA5). An abbreviated list of cities in the United States that currently of historically have used chloramine for disinfection and the approximate year chloramination was employed are listed in Table 1-1. The health concerns for chloramination are outlined in Section 4.

TABLE 1-1
UNITED STATES CITIES THAT USE CHLORAMINE FOR DISINFECTION

City	Approximate Year Chloramination Employed
Denver, CO	1917
Portland, OR	1924
Boston, MA	1932*
St. Louis, MO	1934
Portland, ME*	1938
Indianapolis, IN	1954
Minneapolis, MN	1954
Dallas, TX	1959
Kansas City, MO	1964
Milwaukee, WI	1964
Philadelphia, PA	1969
Houston, TX	1982
Miami, FL	1982
San Diego, CA	1982

Source: Trussell and Kreft, 1984
*Provided by Champlain Water District as part of Draft Report review comments.

Existing and emerging technologies to reduce DBP formation, remove DBPs, and corresponding disinfection processes are discussed in the upcoming sections of this report. In addition, the effect of treatment and disinfection on the distribution system water quality and ongoing research in distribution water quality will be discussed.

1.3 ABBREVIATIONS

The following is a list of abbreviations used in this document.

AA	Alternative Analysis
AIP	Agreement in Principle
AL	Action Level: the level in which prescribed actions are required.
AOC	Assimilable Organic Carbon
AOM	Assimilable Organic Matter
AOP	Advanced Oxidation Process
AFT	Alternative Filtration Technologies
AWWA	American Water Works Association

AWWARF	American Water Works Association Research Foundation, now called the Water Research Foundation
BASINS	Better Assessment Science Integrating Point and Nonpoint Sources
BAT	Best Available Technology
BDCM	Bromodichloromethane
BDOL	Biodegradable organic carbon
BF	Baffle factor
BOM	Biodegradable Organic Matter
CCT	Corrosion Control Treatment
CDBM	Chlorodibromomethane
CDPH	California Department of Public Health
CDBPs	Chlorinated Disinfection Byproducts
CDC	Centers for Disease Control and Prevention
CFD	Computational Fluid Dynamics
CFE	Combined Filter Effluent
Chloramine	Defined here as monochloramine
CT	Disinfectant Concentration x Time (Disinfection Capacity)
CWD	Champlain Water District
CWS	Community Water System
DBPR	Disinfectant/Disinfection Byproduct Rule
DAF	Dissolved Air Flotation
Dalton	A unit of mass used to express atomic and molecular masses and typically used as a unit of measure for the MWCO of a UF, NF, or RO membrane. It is equivalent to 1/12th the mass of a carbon-12 atom (i.e., one atomic mass unit (amu)).
DBPs	Disinfection Byproducts
DEC	Department of Environmental Conservation - Water Supply Division
DEC	Vermont Department of Environmental Conservation
DHH	United States Department of Health and Human Services
DNA	Deoxyribonucleic Acid
DOC	Dissolved Organic Carbon
DOH	Department of Health
DWC	Drinking Water Committee
EBCT	Empty Bed Contact Time. This is the theoretical hydraulic time in filter bed assuming a void space of 100% (i.e. no media)

EC	European Community
ESWTR	Enhanced Surface Water Treatment Rule. A general term used to reference the IESWTR, LT1ESWTR, and LT2ESWTR.
EPA	Environmental Protection Agency
FACA	Federal Advisory Committee Act
FBR	Filter Backwash Rule
FBT	Fluidized Bed Treatment
GAC	Granular Activated Carbon
GWR	Ground Water Rule
GWUDI	Groundwater Under Direct Influence of Surface Water
IESWTR	Interim Enhanced Surface Water Treatment Rule. .
HAA	Haloacetic Acid
HAA5	Sum of five (5) Haloacetic Acids (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid)
HAAFP	Haloacetic Acid Formation Potential
HAN	Haloacetonitriles
HIOP	Heated Iron Oxide Particle
HSP	High Service Pump
ICR	Information Collection Rule
IMS	Integrated Membrane System
IOH	International Ozone Association
IOCO	Iron Oxide-coated Olivine
LCR	Lead and Copper Rule
LLP	Low Lift Pump
LSLR	Lead Service Line Replacement
LT1 ESWTR	Long Term 1 Enhanced Surface Water Treatment Rule
LT2 ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
MAC	Maximum Acceptable Concentration
MCL	Maximum Contaminant Level. An MCL is the maximum permissible level of a contaminant in water that is delivered to any user of a public water system. MCLs are enforceable. These are set as close as feasibly possible to MCLGs using the best available treatment technology. An MCL is not established for an MCLG until sufficient information is available.
MCLG	Maximum Contaminant Level Goal. MCLGs are non-enforceable public health goals that are set at a level where no known or anticipated adverse health risks would occur and which allow for an adequate margin of safety.

MF	Microfiltration
MIEX	Magnetic Ion Exchange Resin
MRDL	Maximum Residual Disinfectant Level. The DBPR introduced Maximum Residual Disinfectant Levels as enforceable levels analogous to the MCLs that recognize the benefits of adding a disinfectant to water for pathogen control but set legally enforceable limits.
MRDLG	Maximum Residual Disinfectant Level Goal. The Stage 1 Disinfectants and Disinfection Byproducts Rule (DBPR) introduced the new term "Maximum Residual Disinfectant Level Goal" (MRDLG). MRDLGs, like MCLGs, are non-enforceable health goals. These levels pertain to the residual disinfectant levels in product water. The term was included in the Rule to reflect the concept that these substances are beneficial and to avoid the situation where a treatment plant operator might be reluctant to apply disinfectant dosages above the MRDLG during short periods of time to control a microbial risk.
MWCO	Molecular Weight Cutoff : a measure of the removal characteristic of a membrane in terms of atomic weight (or mass), as opposed to pore size and vary with the type of filter.
NCWS	Non-Community Water System
NDEA	N-Nitrosodiethylamine
NDBA	N-Nitrosodi-n-butylamine
NDMA	N-Nitrosodimethylamine
NDPA	N-Nitrosodi-n-propylamine
NF	Nanofiltration
NMEA	N-Nitrosomethylethylamine
NOM	Natural Organic Matter
NPDOC	Non-purgeable Dissolved Organic Carbon
NPIP	N-Nitrosopiperidine
NTU	Nephelometric Turbidity Unit
NYPR	N-Nitrosopyrrolidine
O&M	Operation and Maintenance
PAA	Peracetic Acid
PAC	Powdered Activated Carbon
PAC1	Polyaluminum Chloride
PE	Public Education
pH	Log of hydrogen ion centration in molar units
PN	Public Notification

PODR	Point of Diminishing Return
POTW	Publically owned water treatment works. A term typically used for wastewater treatment facilities.
PWS	Public Water System
RFP	Request for Proposals
RO	Reverse Osmosis
SAB	Science Advisory Board
SDWA	Safe Drinking Water Act
SDSTHMFP	Simulated Distribution System Trihalomethane Formation Potential
SWMWT	Source Water Monitoring and Source Water Treatment
SPA	Source Protection Area
SPP	Source Protection Plan
SUVA	Specific Ultraviolet Absorbance at 254 nm
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TCRDSAC	TCR/Distribution System Advisory Committee
THMs	Trihalomethanes (a group of four chemicals; bromodichloromethane, dibromochloromethane, bromoform, and chloroform, formed when chlorine or other disinfectants used to control microbial contaminants in drinking water react with naturally occurring organic and inorganic matter in water)
TT	Treatment Technique. Where there is no reliable method that is economically and technically feasible to measure a contaminant at particularly low concentrations, a Treatment Technique is set rather than an MCL. A Treatment Technique is an enforceable procedure or level of technological performance that public water systems must follow to ensure control of a contaminant.
TTHMs	Total Trihalomethanes (the sum of the concentrations of bromodichloromethane, dibromochloromethane, bromoform, and chloroform)
THMFP	Trihalomethane Formation Potential
TOC	Total Organic Carbon
TOXFP	Total Organic Halide Formation Potential
TT	Treatment Technique
TTHM	Total Trihalomethane
UDF	Unidirectional Flushing
UF	Ultrafiltration
UMCR2	Unregulated Contaminant Monitoring Rule 2

UV	Ultraviolet light
UV254	Ultraviolet light at 253.7 nm
VGS	Vermont Geological Survey
VCGI	Vermont Center for Geological Information
WAP	Water Quality Parameters
WSR	Water Supply Rule
WTF	Water Treatment Facility
XDBP	Halogenated Disinfection Byproducts

SECTION 2.0 - STUDY OBJECTIVES AND METHODOLOGY

2.1 STUDY OBJECTIVES

The objective of this study requested by the DEC is to identify and assess the appropriate treatment options that will enable 15 Vermont water systems shown in Table 2-1 to comply with the Stage 1 and Stage 2 DBPRs. The costs of the current treatment will be compared to the treatment options including using chloramine for secondary disinfection.

The engineering assessment consisted of the following:

1. A comparative assessment of treatment options;
2. An analysis of the technical feasibility of implementing each of the assessed treatment options;
3. An evaluation of whether implementation of an assessed treatment option will result in simultaneous compliance with applicable Federal and State rules including LCR and TCR;
4. A 20-year present worth analysis of the estimated capital, operating, and maintenance costs associated with implementation of each assessed treatment option. Costs shall be presented on an annual cost per equivalent user basis together with the assumptions used in deriving the costs.

TABLE 2-1
UTILITY PARTICIPATION

Water System Name
Alburgh Village
Bennington
Burlington
Catamount Bolton
Champlain Water District
Grand Isle Consolidated Water District
North Hero
Proctor
Readsboro
Richford
Rutland
St. Johnsbury
Swanton
Tri Town Water District
Vergennes-Panton Water District

5. An assessment of whether the water system's existing operating capability restricts implementation of an assessed treatment option or would require additional operating resources.

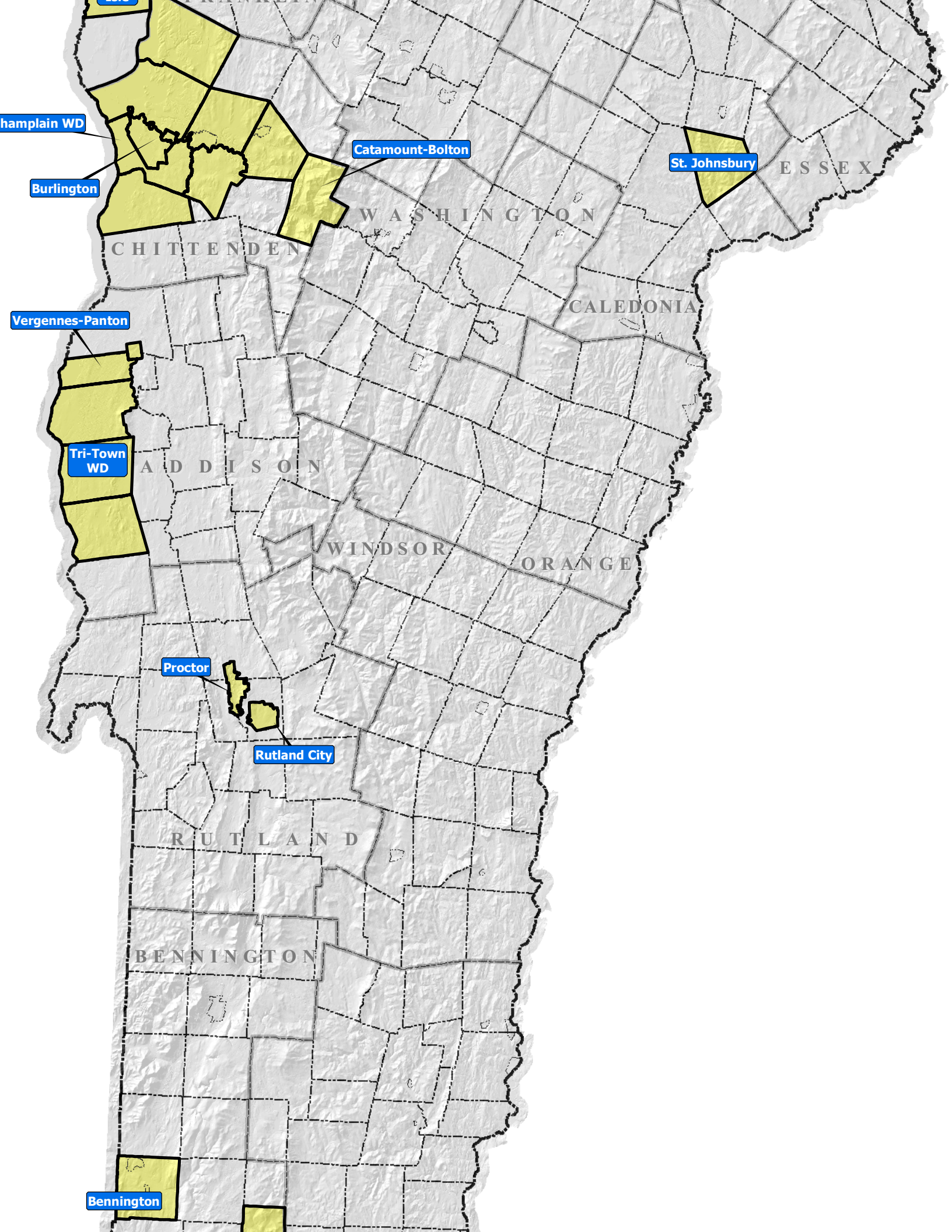
Specific discussions were required by DEC as part of the evaluation for each utility:

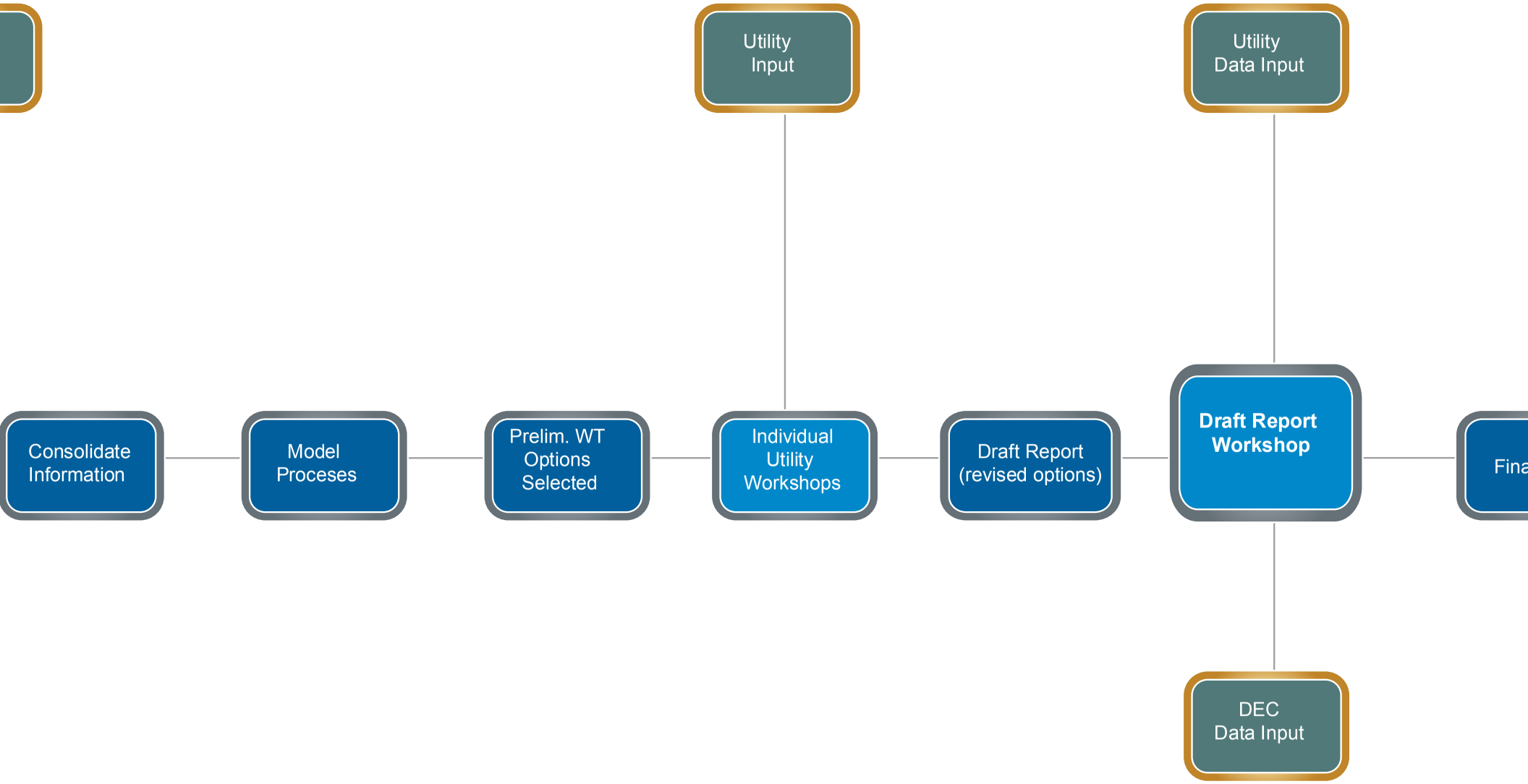
1. A narrative description of the existing treatment process, including size of component tanks and pumps, a flow schematic diagram showing flow directions and functional components, and a table summarizing available water quality data for source water and treated water.
2. A discussion of the treatment technologies and plant modifications to be considered if free chlorine is used as a secondary disinfectant.
3. A discussion of the treatment technologies and plant modifications considered if monochloramine is used as a secondary disinfectant.
4. A narrative description of the conceptual alternatives together with a schematic diagram showing flow directions and functional components. A narrative describing the assumptions and rationale for selection of treatment options and sizing of components.
5. An assessment of the water system's operating capabilities shall be performed and shall include identification of any additional resources, such as the number of operators and their qualifications, additional land and power needed, etc. that may be required to operate the recommended modifications.

2.2 METHODOLOGY

In order to address the scope above, AECOM develop a project work plan as shown in Figure 2-2. This work plan included a kickoff meeting in which the work plan was discussed as well as the associated information which AECOM would be soliciting. A survey was electronically mailed to the participants and data was mailed back to AECOM. During the interim period, water quality and operational data was provided by DEC. This data was reviewed and assembled into tabular and is some case graphical format. Based on the data, AECOM developed preliminary technology selections for each utility.

AECOM proceeded in meeting with each utility as part of the Utility Workshop, and reviewed the dataset and preliminary alternatives in addressing the Stage 1 and 2 DBPR. Based on the discussion during the workshop, the alternatives and datasets were revised accordingly.


hamplain WD
Burlington
Catamount-Bolton
St. Johnsbury
ESSEX
WASHINGTON
CHITTENDEN
CALEDONIA
Vergennes-Panton
Tri-Town WD
ADDISON
WINDSOR
ORANGE
Proctor
Rutland City
RUTLAND
BENNINGTON
Bennington


Utility Input
Utility Data Input
Consolidate Information
Model Proceses
Prelim. WT Options Selected
Individual Utility Workshops
Draft Report (revised options)
Draft Report Workshop
Fina
DEC Data Input

The report and associated cost for each alternative were developed as outlined in Section 5 using standard practices used in the industry. The draft report was distributed to the stake holder group and provided roughly a two week period in which each utility could review the draft document and comment. Individual comment forms were provided so review comments could be recorded.

The draft workshop was held at the state campus in Waterbury, VT and report discussed. Comments were recorded and submitted, and where appropriate, the report was revised to reflect those comments. A final report was produced and legislative testimony was provided by AECOM.

3 LIMITATIONS

It should be understood that the information and costs provided in this report are based on best practices, both from an engineering and operational perspective, as well as a limited dataset provided from each utility and the State. Costs presented in this report should not be considered budgetary, but rather as an order of magnitude cost allowing comparative evaluation for each alternative. In order to confirm the values provided in this document, pilot studies are necessary to validate system performance and associated costs as well as further engineering evaluations for compliance with existing regulations.

The raw historical data summarized in this report is believed by AECOM to present the actual data as found in each system. Care has been taken by each party involved to handle the data in a fashion to maintain data integrity and represent each community correctly. However, it should be noted that due to the large volume of data handled, AECOM cannot guarantee that each and every value is correct.

SECTION 3.0 - REGULATORY REQUIREMENTS

3.1 INTRODUCTION

Historically, public health concerns with drinking water have focused on microbiological contaminants. In the United States, this concern resulted in public health legislation designed to control pathogens. Until the mid to late 1980's, legislation and the resulting regulations written by the EPA under the Safe Drinking Water Act (SDWA), were primarily based on the use of chlorine as the disinfecting agent. As a result, bacterial and viral outbreaks from contaminated drinking water were significantly diminished. However, as more research was conducted, concerns arose for the possible health effects associated with certain byproducts created when surface waters were disinfected with chlorine and other chemical disinfectants.

To further protect public health, the SDWA was amended in 1986 and 1996 to regulate the nation's water supply. In response to these amendments, the EPA wrote several regulations regarding the protection of source water, control of microbiological contaminants, and the control of disinfection byproducts (DBPs). The following is a brief summary of the legal instruments currently in place to control DBPs. Note that some of these regulations may have conflicting effects on the control of DBPs.

3.2 UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

The control of the nation's water supply begins as a federal responsibility under the SDWA. As a result, the EPA has a number of regulations that interrelate and address DBPs, microbiological drinking water quality, and distribution system drinking water quality. Some of these regulations that will be discussed further in this section include the following:

- Stage 1 Disinfectants and Disinfection Byproducts Rule (DBPR).
- Stage 2 Disinfectants and Disinfection Byproducts Rule (DBPR).
- Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR).
- Lead and Copper Rule (LCR).
- Total Coliform Rule (TCR).
- Ground Water Rule (GWR).

The following definitions are important in understanding the requirements under the above rules.

3.2.1 Stage 1 Disinfectants and Disinfection Byproducts Rule (DBPR)

This rule was promulgated on December 16, 1998. Revisions to the rule were published in May 2001. The goal of the Stage 1 DBPR is to reduce the amount of DBPs due to the possible long-term adverse health effects associated with these compounds. The rule applies to all community water systems and non-transient, non-community water systems that utilized either primary or secondary disinfection as part of their treatment process. In addition, the Stage 1 DBPR applies to transient, non-community water systems using chlorine dioxide for any purpose. The rule sets maximum residual disinfectant level goals and maximum contaminant level goals for certain disinfection byproducts. The specific MCLGs, MCLs, MRDLGs, and MDRLs of this rule can be found in Tables 3-1 and 3.2.

TABLE 3-1
MAXIMUM CONTAMINANT LEVEL GUIDELINES AND MAXIMUM CONTAMINANT LEVELS FOR DISINFECTION BYPRODUCTS

Disinfection Byproducts	MCLs (mg/L)	MCLGs (mg/L)	Compliance Basis
Total Trihalomethanes (TTHM) (sum of next 4 contaminants)	0.080		Running Annual Average
Chloroform		-	
Bromodichloromethane		0	
Dibromochloromethane		0.06	
Bromoform		0	
Five Haloacetic Acids (sum of next 5 contaminants)	0.060		Running Annual Average
Monochloroacetic		-	
Dichloroacetic Acid		0	
Trichloroacetic Acid		0.3	
Bromoacetic Acid		-	
Dibromoacetic Acid		-	
Bromate	0.010	0	Annual Average
Chlorite	1.0	0.8	Monthly Average

TABLE 3-2
MAXIMUM RESIDUAL DISINFECTANT LEVEL GOALS (MRDLGS) AND MAXIMUM RESIDUAL DISINFECTANT LEVELS (MRDLS) FOR DISINFECTANTS

Disinfectant	MRDL (mg/L)	MRDLG (mg/L)	Compliance Basis
Chlorine	4.0 (as Cl_2)	4.0 (as Cl_2)	Annual Average
Chloramine	4.0 (as Cl_2)	4.0 (as Cl_2)	Annual Average
Chlorine Dioxide	0.8 (as ClO_2)	0.8 (as ClO_2)	Daily Sample

A petition to review the MCLG for chloroform and the method by which the guideline was developed was filed in the US Court of Appeals for the District of Columbia Circuit. On March 31, 2000 the US District Court issued a ruling vacating the MCLG for chloroform. The EPA, on May 30, 2000, removed the zero MCLG for chloroform from the National Primary Drinking Water Regulations.

The Stage 1 DBPR also requires that plants practicing conventional filtration treatment or softening treatment adopt an Enhanced Coagulation or Enhanced Precipitative Softening technique as Treatment Techniques for compliance with DBPs. The Enhanced Coagulation and Enhanced Precipitative Softening provisions require that conventional treatment plants or precipitative softening plants operate their processes in order to achieve greater reductions in byproduct precursors, as indicated by total organic carbon (TOC). The TOC removal values were established with the intent that 90 percent of the water treatment systems would be capable of meeting those removals.

The required TOC removal is based on the alkalinity of the source water. This is due to the fact that optimal TOC removal takes place at a pH between 5.5 and 6.6 and considering that coagulants are alkalinity consumers. In water sources where the alkalinity levels are high, it might not be economically viable to lower the pH with the addition of a coagulant alone. Enhanced coagulation as prescribed by the EPA Stage 1 DBPR does not consider optimizing the process by lowering the pH of high alkalinity water by the addition of pH adjusting chemical.

Table 3-3 lists the TOC reduction requirements for source waters of varying alkalinity. Conventional treatment plants are required to meet the appropriate TOC removal percent listed. Precipitative softening treatment plants must meet the requirements listed in the right hand column. These removal requirements were determined after a statistical review of the operating conditions at existing plants was completed and adjusted so that 90 percent of plants could comply without unreasonable coagulant dosage.

TABLE 3-3
TOTAL ORGANIC CARBON REMOVAL REQUIREMENTS

TOC Levels In Source Water (mg/L)	TOC Removal Required		
	0 – <60 mg/L Alkalinity	≥60 – <120 mg/L Alkalinity	≥120 mg/L Alkalinity
>2.0 – 4.0	35%	25%	15%
>4.0 – 8.0	45%	35%	25%
>8.0	50%	40%	30%

Source: EPA Stage 1 DBPR Quick Reference Guide, May 2001.

The Compliance deadline for large surface waters (serving 10,000 customers or more) or ground water systems under direct influence of surface waters (GWUDI) was January 1, 2002. The deadline for surface waters (serving 10,000 customers or less) or GWUDIs was January 1, 2004.

3.2.2 Stage 2 Disinfectants and Disinfection Byproducts Rule (DBPR)

The EPA published the Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 DBPR) on January 4, 2006. The Stage 2 DBPR builds on the Stage 1 DBPR regulations by requiring water systems to meet DBP MCLs at each monitoring site in the distribution system, rather than an average of all monitoring sites, to better protect public health.

The Stage 2 DBPR, was proposed along with the LT2ESWTR, and applies to all community water systems (CWSs) and nontransient noncommunity water systems (NTNCWSs) that add a disinfectant other than ultraviolet (UV) light. The Stage 2 DBPR retains 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 changed the way compliance with these MCLs is determined. Under Stage 1, compliance with the TTHM and HAA5 MCLs was based on a running annual average of all monitoring points within a distribution system. Under the

Stage 2 DBPR, compliance is based on a locational running annual average, which means the running annual average at each monitoring point within a distribution system would have to be less than the MCL. The Stage 2 DBPR also requires systems to conduct an initial distribution system evaluation which identifies the areas with the highest concentrations of TTHM and HAA5; compliance monitoring will be conducted at those locations. The Stage 2 DBPR also maintains the enhanced coagulation and enhanced precipitative softening provisions for TOC removal based on the alkalinity and TOC concentrations of the source water.

As mentioned, the Stage 2 DBPR includes a provision requiring all CWSs and only NTNCWSs serving more than 10,000 people to conduct an initial distribution system evaluation (IDSE). NTNCWS serving less than 10,000 are exempted from IDSE requirements, but will need to comply with the Stage 2 DBPR compliance monitoring requirements. The goal of the IDSE is to characterize the distribution system and identify monitoring sites where customers may be exposed to high levels of TTHMs and HAA5. There are four ways to comply with the IDSE requirements: Standard Monitoring, System Specific Study (SSS), 40/30 Certification (40/30), and Very Small System (VSS) Waiver.

The Standard Monitoring option requires the system to collect 1 year of TTHM and HAA5 data at a specified frequency and locations to characterize TTHM and HAA5 levels in the distribution system. In addition to this data, the system must use available Stage 1 DBPR compliance data to determine the best locations for Stage 2 DBPR compliance monitoring. Any system may conduct Standard Monitoring to meet the IDSE requirements of the Stage 2 DBPR.

System specific study (SSS) is an option for water systems that have extensive existing DBP data or have prepared a hydraulic model that can be used to determine locations of high DBP levels in their distribution system. Systems will have to meet minimum requirements to perform either option.

The 40/30 and the VSS Waiver allows a system to comply with the IDSE requirement without having to conduct additional distribution system monitoring. To be eligible for a 40/30 Waiver, the water system must meet all of the following requirements for eight consecutive quarters, as dictated by its Schedule:

1. Collected all required Stage 1 DBPR samples.
2. No individual TTHM samples exceeded 0.040 mg/L and no individual HAA5 samples exceeded 0.030 mg/L.
3. The system has not had any TTHM or HAA5 monitoring violations.

To be eligible for a VSS Waiver, the system must meet all of the following requirements:

1. System serves less than 500 people.
2. System has collected eligible TTHM & HAA5 data.

As mentioned above, the Stage 2 DBPR retains the MCLs of 80 µg/L for TTHM and 60 µg/L for HAA5 established by the Stage 1 DBPR. However, MCLGs for chloroform, monochloracetic acid, and trichloracetic acid were increased from the Stage 1 DBPR. The MCLs and MCLGs for the Stage 2 DBPR rule are illustrated in Table 3-4.

TABLE 3-4
STAGE 2 DBPR MCLS AND MCLGS FOR DISINFECTION BYPRODUCTS

Disinfection Byproducts	**MCLs (mg/L)**	**MCLGs (mg/L)**	**Compliance Basis**
Total Trihalomethanes (TTHM) (sum of 4 next contaminants)	0.080		Locational Running Annual Average
Chloroform		**0.07**	
Bromodichloromethane		0	
Dibromochloromethane		0.06	
Bromoform		0	
Five Haloacetic Acids	0.060		Locational Running Annual Average
Monochloroacetic Acid		**0.07**	
Dichloroacetic Acid		0	
Trichloroacetic Acid		**0.02**	
Bromoacetic Acid		-	
Dibromoacetic Acid		-	

As part of the Stage 2 DBPR, an operational evaluation is required based on a forecast of the DBP levels as described below from USEPA. "If (Q1 + Q2 + 2Q3)/l >MCL, then the system must conduct an operational evaluation where: Q3 = current quarter measurement, Q2 = previous of water measurement, and Q1 = quarter before previous quarter measurement. MCL = Stage 2 MCL for THM or Stage 2 MCL for HAA5. The operational evaluation includes an examination of system treatment of distribution operational practices, including changes in services or source water quality, storage tank operations, and excess storage capacity that may contribute to high TTHM and HAA5 formation. The system must submit their operational evaluation report to the state for review within 90 days after being notified of the analytical results that initiates the operational evaluation".

The Compliance deadline for CWSs and NTNCWSs varies depending on the number of customers served and whether the CWS and NTNCWS are part of a combined distribution system. Compliance dates are provide in Table 3-5.

TABLE 3-5
STAGE 2 DBPR COMPLIANCE DATES

Water System Name	WSID	Population	Compliance Date
Alburgh Village	5136	576	10/1/13
Bennington	5016	12,000	10/1/13
Catamount Bolton	5051	1,000	10/1/13
Burlington	5053	47,600	10/1/13
Grand Isle CWD	20614	1,113	10/1/13
North Hero	20562	600	10/1/13
Proctor	5228	2,200	10/1/13
Rutland City	5229	18,500	10/1/13
St. Johnsbury	5045	3,800	10/1/13
Swanton Village	5132	3,500	10/1/13
Tri Town WD	5001	3,400	10/1/13
Readsboro	5028	440	10/1/13
Richford Water	5126	1,700	10/1/13
Champlain Water District	5092	69,923	10/1/12
Vergennes-Panton	5010	5,080	10/1/13

3.2.3 Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

The Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) was published in the Federal Registrar on January 5, 2006. The purpose of LT2ESWTR is to reduce illness linked with the microbiological contaminant *Cryptosporidium* and other pathogenic microorganisms in drinking water. The LT2ESWTR supplements existing SDWA regulations by targeting additional *Cryptosporidium* treatment requirements for those public water supplies (PWSs) at high risk. These higher risk systems include filtered water systems with high levels of *Cryptosporidium* in their source waters and all unfiltered water systems, which do not treat for *Cryptosporidium*. The rule requires PWSs to monitor their source water, calculate an average *Cryptosporidium* concentration, and use those results to determine if their source water is susceptible to contamination and may required additional treatment. The rule also contains provisions to reduce risks from uncovered finished water reservoirs and provisions to ensure that PWSs maintain microbial protection when they take steps to decrease the formation of DBPs.

The LT2ESWTR requires all surface water treatment systems, including GWUDI, that serve at least 10,000 people to conduct two years of source water monitoring for *Cryptosporidium*. Filtered systems that already provide at least 5.5 log of treatment for *Cryptosporidium* and unfiltered systems providing at least 3-log of treatment for *Cryptosporidium* are not required to conduct source water monitoring. Those systems that intend to install this level of treatment are not required to conduct source water monitoring as well.

Along with monitoring and recording *Cryptosporidium* concentrations, filtered water treatment systems must also record source water *E. coli* and turbidity levels. Based on the results of the source water monitoring, filtered systems are classified into one of four "Bins". Additional treatment must be provided for *Cryptosporidium* based on their bin classification (average source water Cryptosporidium concentration), using treatment options available from a "microbial toolbox." Treatment options included in this "microbial toolbox" include filtration processes such as granular media filtration, cartridge filters, or membranes. Other treatment options in the "microbial toolbox" include utilizing disinfectants that are effective against *Cryptosporidium*, such as chlorine

dioxide, UV, and ozone. Note: chlorine and chloramine are largely ineffective at inactivating *Cryptosporidium*. The Bin Classifications for filtered systems are summarized in Table 3-6.

TABLE 3-6
BIN CLASSIFICATION FOR FILTERED SYSTEMS UNDER THE LT2ESWTR

***Cryptosporidium* Concentration (oocysts/L)**	**Bin Classification**	**Additional *Cryptosporidium* Treatment Required**			**Alternative Filtration**
		Conventional Filtration	**Direct Filtration**	**Slow Sand or Diatomaceous Earth Filtration**	
< 0.075	1	None	None	None	None
0.075 to < 1.0	2	1 log	1.5 log	1 log	(1)
1.0 to 3.0	3	2 log	2.5 log	2 log	(2)
> or equal to 3.0	4	2.5 log	3 log	2.5 log	(3)

Source: EPA UV Design Guidance Manual, November 2006.
(1) As determined by the state (or primacy agency) such that total removal/inactivation is greater than 4.0 log.
(2) As determined by the state (or primacy agency) such that total removal/inactivation is greater than 5.0 log.
(3) As determined by the state (or primacy agency) such that total removal/inactivation is greater than 5.5 log.

Unfiltered systems must provide additional treatment for *Cryptosporidium* using chlorine dioxide, ozone or UV to achieve 2- or 3-log inactivation of *Cryptosporidium*, depending on their source water concentration. Treatment requirements for unfiltered PWSs are summarized in Table 3-7. All existing requirements for unfiltered systems remain in effect, including disinfection to achieve at least 3-log inactivation of *Giardia* and 4-log inactivation of viruses.

TABLE 3-7
UNFILTERED SYSTEM REQUIREMENTS UNDER THE LT2ESWTR

***Cryptosporidium* Concentration (oocysts/L)**	**Additional *Cryptosporidium* Treatment Required**
< or equal to 0.01	2 log*
> 0.01	3 log*

Source: EPA UV Design Guidance Manual, November 2006.
* Overall disinfection requirements must be met with a minimum of two disinfectants [40 CFR 141.12(d)]

The LT2ESWTR was promulgated simultaneously with the Stage 2 DBPR to address concerns about the risk tradeoffs associated with microbial disinfection and the DBPs commonly formed with typical disinfectants (i.e. chlorine, ozone, etc.). Therefore, when evaluating disinfection strategy, PWSs should consider the disinfectant effectiveness against the target microbial pathogens and the DBPs formed as a result of the disinfectant.

As with the Stage 2 DBPR, compliance deadlines for PWSs varies depending on the number of customers served. Treatment installation dates pertain only to PWSs that are required to provide additional treatment for *Cryptosporidium*. The earliest date (those PWSs that fall under Schedule 1) that PWSs must install and operate additional treatment in accordance with their bin classification is March 2012. The deadline for those that fall under Schedule 4 is September 2014.

3.2.4 Lead and Copper Rule (LCR)

The Lead and Copper Rule (LCR) was published on June 7, 1991 and revised in 2000, 2004, 2006, and final revisions in June 2007. The final revisions in 2007 were developed to address questions regarding tap water sample calculations and monitoring requirements.

Long term exposure to lead may lead to damage of the brain, red blood cells, and kidneys especially for young children and pregnant women. Long term exposure to copper may cause stomach and intestinal problems, and liver or kidney damage. The goal of the LCR is to provide maximum human health protection by reducing lead and copper levels at consumers' taps to as close to the MCLGs as is feasible. To accomplish this goal, the LCR establishes requirements for CWSs and NTNCWSs to optimize corrosion control and conduct periodic monitoring. All CWSs and NTNCWSs collect samples at cold water taps in homes and buildings that are at high risk of lead/copper contamination as identified in 40 CFR 141.86(a). The number of sample sites is based on PWS size.

The LCR establishes an action level (AL) of 0.015 mg/l for lead and 1.3 mg/l for copper based on the 90th percentile level of tap water samples. Public water systems are required to perform the following treatment techniques and sampling requirements when lead AL exceedances occur at more than 10 percent of the taps that are sampled:

- Water Quality Parameter (WQP) Monitoring: WQPs include pH, alkalinity, calcium, conductivity, orthophosphate (if phosphate based inhibitor is used), silica (if silica based inhibitor is used), and temperature.

- Public Education (PE): Applicable to systems that exceed the lead AL only. Purpose of PE is to educate consumers about lead health effects, sources, and steps to minimize exposure.

- Source Water Monitoring and Source Water Treatment (SOWT): Purpose of source water monitoring is to determine the contribution of lead and copper from the source water to total tap water lead and copper levels and the need for additional SOWT.

- Corrosion Control Treatment (CCT): CCT is applicable to all large systems except those meeting requirements of 40 CFR 141.81(b)(2) or (b)(3).

- Lead Service line Replacement (LSLR): Applicable to systems that continue to exceed the lead AL at the tap after installing optimal CCT and/or SOWT.

Conflicts between the Stage 1 and Stage 2 DBPR, ESWTR and the LCR can occur when the chemical stability of the finished water in the distribution system is affected. Since lead and copper are primarily released into the distribution system by corrosion of pipe or plumbing materials, specific chemical measures such as pH and alkalinity adjustment and the addition of corrosion inhibitors are often necessary. A further discussion of these compliance issues can be found later in this section.

3.2.5 Total Coliform Rule (TCR)

The existing Total Coliform Rule (TCR) was first published in 1989 and became effective in 1990. The TCR is the only microbial drinking water regulation that applies to all Public water systems (PWSs). The rule establishes a MCLG and a MCL based on the presence or absence of total coliforms as well as establishing a measurable disinfectant level. The rule also modifies monitoring requirements including testing for fecal coliforms or E. coli, requires use of a sample siting plan, and also requires sanitary surveys for PWSs collecting fewer than five samples per month. The rule also sets Public Notification (PN) requirements for both acute and non-acute (monthly) violations. The objectives of the TCR are to assist in ensuring the reliability of the distribution system, to identify the effectiveness of treatment and to indicate possible fecal contamination within the system (USEPA, 2001).

The TCR requires all PWSs to routinely monitor for the presence of total coliforms in the distribution system on a monthly basis based on the number of people served. If any sample tests positive for total coliforms, the system must perform additional testing within a 24-hour period. For each positive total coliform sample, the PWS must collect 3-4 repeat samples. A monthly MCL violation is triggered if a PWS collecting fewer than 40 samples per month has greater than 1 routine/repeat sample per month which is total coliform-positive. For those PWSs collecting at least 40 samples per month, a monthly MCL violation is triggered when greater than five percent of the routine/repeat samples in a month are positive for total coliform. An acute MCL violation is triggered if any PWS has any fecal coliform or *E. coli*-positive repeat sample or has a fecal coliform or *E. coli*-positive routine sample followed by a total coliform-positive repeat sample (USEPA, 2001 and 2009).

The EPA is required to review existing national primary drinking water regulations every six years. In 2003, the EPA completed its review of the TCR. The purpose of the review was to identify current health risk assessments, changes in technology, and other factors that would provide a health or technological basis to support a regulatory revision that will maintain or improve public health protection. In the 2003 announcement of the completion of the Six Year Review, EPA published its intent to revise the TCR. Between 2003 and 2007, the EPA and industry experts conducted workshops and developed white papers to assess the effectiveness of the current TCR in three principal areas:

1. Reducing public health risk: Are total coliforms and *E. coli* appropriate public health indicators for MCLs?
2. Monitoring strategies: Sampling locations, frequency and economic burden. What monitoring strategies are available that would decrease economic burden while maintaining or improving public health protection? How risks associated with distribution systems should be addressed.
3. Corrective action strategies: Currently, corrective action is not required for violations of the TCR. How can this be improved? Public notifications (PN) for total coliform violations are somewhat confusing and may cause unnecessary concern (USEPA, 2009).

In 2007, the EPA also decided to establish a committee under the Federal Advisory Committee Act (FACA). The Total Coliform Rule/Distribution System Advisory Committee (TCRDSAC) was established to develop an Agreement in Principle (AIP) based on recommended revisions to the TCR and recommendations for further research and information collection to better understand and address possible public health impacts from potential degradation of drinking water quality in distribution systems (USEPA, 2009). The committee met 13 times from July 2007 through September 2008 to develop this AIP based in part on findings from distribution system white papers and a series of TCR Issue Papers developed by the EPA and the AWWA to present available information on a range of issues related to the TCR (USEPA, 2009).

In early 2009, the TCRDSAC developed an agreement that represented the consensus of the parties involved based on the best information available to the committee.

The complete Agreement in Principle (AIP) and a comparison of the current TCR and the AIP can be found at the following EPA website:

http://www.epa.gov/safewater/disinfection/tcr/regulation_revisions.html

In addition to the AIP, the EPA requested that the Science Advisory Board (SAB) Drinking Water Committee (DWC) review the current TCR and the proposed Revised Total Coliform Rule as written in the AIP. As a result, an Alternative Analysis (AA) was completed to provide another perspective and to compare the current TCR with the AIP.

The proposed new TCR is schedule to be published for public comment in August 2010. The final TCR is schedule to be published in August 2012.

3.2.6 Ground Water Rule (GWR)

The Ground Water Rule (GWR) was published in the Federal Register in November 2006. The purpose of the rule is to provide for increased protection against microbial pathogens in public water systems that use ground water sources. The EPA is particularly concerned about ground water systems that are susceptible to fecal contamination since disease-causing pathogens may be found in fecal contamination.

The GWR will apply to PWSs that use ground water as their source. The rule also applies to any system that mixes surface and ground water if the ground water is added directly to the distribution system and provided to consumers without treatment (USEPA, 2006).

The rule establishes a risk-targeted strategy to identify ground water systems that are at high risk for fecal contamination. The GWR also specifies when corrective action (which may include disinfection) is required to protect consumers who receive water from ground water systems from bacteria and viruses.

Ground water occurrence studies and recent outbreak data show that pathogenic viruses and bacteria can occur in public water systems that use ground water and that people may become ill due to exposure to contaminated ground water (USEPA, 2006).

Viral and bacterial pathogens are present in human and animal feces, which can, in turn, contaminate drinking water. Fecal contamination can reach ground water sources, including drinking water wells, from failed septic systems, leaking sewer lines, and by passing through the soil and large cracks in the ground. Fecal contamination from the surface may also find its way into a drinking water well along its casing or through cracks if the well is not properly constructed, protected, or maintained (USEPA, 2006).

The EPA does not believe all ground water systems are fecally contaminated. In fact, data indicates that only a small percentage of ground water systems are fecally contaminated. However, the severity of health impacts and the number of people potentially exposed to microbial pathogens in ground water indicate that a regulatory response is warranted.

As mentioned previously, the GWR addresses risks through a risk-targeting approach. This approach relies on the four major components of the GWR. These major components are summarized below.

Periodic Sanitary Surveys

Periodic sanitary surveys of ground water systems are required based on the evaluation of eight critical elements and the identification of significant deficiencies (e.g., a well located near a leaking septic system). States must complete the initial sanitary survey by December 31, 2012 for most community water systems (CWSs) and by December 31, 2014 for CWSs with outstanding performance and for all non-community water systems (NCWSs) (USEPA, 2006).

Source Water Monitoring

Source water monitoring is required by PWSs to test for the presence of *E. coli*, enterococci, or coliphage. There are two monitoring provisions:

1. *Triggered monitoring*. Triggered monitoring is for systems that do not already provide treatment that achieves at least 99.99 percent (4-log) inactivation or removal of viruses and that have a total coliform positive routine sample under the TCR in the distribution system.
2. *Assessment monitoring*. Assessment monitoring is a complement to triggered monitoring. A state has the option to require systems, at any time, to conduct source water assessment monitoring to help identify high risk systems (USEPA, 2006).

Corrective Action

Corrective actions are required for any PWS with a significant deficiency or source water fecal contamination. The system must implement one or more of the following correction action options:

1. Correct all significant deficiencies.
2. Eliminate the source of contamination.
3. Provide an alternate source of water.
4. Provide treatment which reliably achieves 99.99 percent (4-log) inactivation or removal of viruses. Treatment technologies may include disinfection with chlorine, chlorine dioxide, or ozone and filtration with nanofiltration (NF). Note that due to the current required doses to receive credit for inactivation of viruses by the EPA and the existing validation procedures; UV light disinfection is not a cost effective disinfection alternative compared to chlorine or chlorine dioxide (USEPA, 2006).

Compliance monitoring

Compliance monitoring is required to ensure that treatment technology installed to treat drinking water reliably achieves at least 99.99 percent (4-log) inactivation or removal of viruses (USEPA, 2006).

3.2.7 Simultaneous Compliance Issues

Simultaneous compliance issues are a common issue for PWSs that must comply with a multitude of varying regulations to protect the health and safety of the public. PWSs may be required to increase/upgrade their current disinfection practice to reduce the risk of microbial contamination in finished water, while at the same time minimize the formation of DBPs.

A brief discussion on the above most common compliance issues is included below.

Stage 1 and Stage 2 DBPR and the EWSTRs

The Stage 1 and Stage 2 DBPRs focuses on minimizing the formation of DBPs in the distribution system of PWSs to reduce the long-term exposure of customers to these potentially carcinogenic compounds through enhanced coagulation or enhanced softening. In contrast, the ESWTR focuses primarily on achieving adequate disinfection and removal of pathogens to protect PWS customers from acute pathogenic exposure that can cause outbreaks of waterborne disease. Since the Stage 1 DBPR is intended to minimize the formation of DBPs and residual disinfectants, this rule may conflict with the ESWTR which specifies levels of treatment techniques required for the control of *Cryptosporidium* (USEPA, 1999).

Modifications to the disinfection process to aid in complying with the Stage 1 and Stage 2 DBPR regulations, such as changing the disinfectant type (e.g. chlorine to chloramine) and moving the point of disinfection application (e.g. from raw water inlet to post filtration) may have an effect on secondary disinfectant residual levels in the distribution system. This may negatively affect compliance with the ESWTRs. Therefore, a PWS should continually try to optimize their water treatment processes to balance particle removal, disinfection of pathogens, and control of DBPs.

Stage 1 and Stage 2 DBPR, the EWSTR and the LCR

Conflicts between the Stage 1 and Stage 2 DBPR, ESWTR and the LCR can occur when the chemical stability of the finished water in the distribution system is affected. Since lead and copper are primarily released into the distribution system by corrosion of pipe or plumbing materials, specific chemical measures such as pH and alkalinity adjustment and the addition of corrosion inhibitors are often necessary. In addition, certain treatment techniques that may be necessary for PWSs to comply with the Stage 1 DBPR, such as enhanced coagulation can alter the existing water chemistry in the system by lowering the pH. Similarly, certain treatment techniques that may be necessary for PWSs to comply with the ESWTRs, such as removing additional turbidity, can be affected by the addition of chemicals that inhibit corrosion (USEPA, 1999).

The control measures for LCR compliance consist of chemical addition to raise pH and alkalinity, and/or the addition of corrosion inhibitors such as orthophosphate. Table 3-8 summarizes the potential impacts of LCR control strategies on the compliance requirements for the Stage 1 and Stage 2 DBPR and the ESWTR.

Compliance with the Stage 1 and Stage 2 DBPR may require changing the secondary disinfectant (to provide a residual in the distribution system). Changes to the secondary disinfectant may impact the redox potential of the finished water as well as overall water quality. This change in water quality, and especially in redox potential, may impact lead and copper corrosion rates.

For example, the use of chloramine as a secondary disinfectant may result in a significant reduction in TTHM and HAA5 formation potential in the distribution system. This has led many PWSs to switch from using chlorine to utilizing chloramine as a secondary disinfectant. If lead (IV) scales are present on the interior distribution piping and in contact with chloramine, these lead (IV) scales can deteriorate due to the change in redox potential associated with chloramine chemistry and lead to possible compliance issues with the LCP.

TABLE 3-8
LCR IMPACTS ON ESWTRs AND STAGE 1 & 2 DBPR REQUIREMENTS

Corrosion Control Treatment for LCR	Impacts on ESWTR Compliance	Impacts on Stage 1 & 2 DBPR Compliance
Increase pH	• Requires higher CT values for chlorine disinfection. • Use of coagulants at higher pH may increase residual aluminum concentrations.	• Increased THM formation. • Higher CT requirement may increase disinfectant dosage and increase DBP formation potential.
Alkalinity Adjustment	• Coagulant chemical dosage may need to be adjusted to achieve optimal pH.	• None.
Corrosion Control Inhibitor addition (Ortho or Polyphosphates)	• May need to reduce optimum pH for aluminum precipitation. • Turbidity increases from zinc carbonate precipitation (if zinc orthophosphate is used).	• None.
Silicate Addition	• High silica levels can form precipitates, causing an increase in turbidity levels.	• None.

In addition, nitrification caused in part by excess ammonia used during the chloramination process can occur in the distribution system. The excess ammonia leads to the growth of nitrifying bacteria that convert ammonia to nitrates. Nitrification in the distribution system can lower the pH of the water and consequently hinder lead and copper corrosion control. Nitrification may also lead to a loss of chlorine residual which allows an increase in HPC bacteria. This increase in HPC bacteria can potentially increase total coliform counts that may result in a positive sample under the TCR.

Nitrification may be controlled by reducing the detention time, keeping water temperatures low, increasing the chlorine to ammonia ratio, checking the ammonia concentration, and maintaining chloramine residuals > 2 mg/L.

The feasibility of switching to an alternate disinfectant may depend on how well the disinfectant can be managed and the compatibility of the disinfectant with existing operating systems. Table 3-9 summarizes the Stage 1 and Stage 2 DBPR and ESWTR impacts on the LCR requirements (USEPA, 1999).

TABLE 3-9
STAGE 1 & 2 DBPR AND ESWTR IMPACTS ON LCR REQUIREMENTS

Stage 1 & 2 DBPR and ESWTR Control Strategy	Impacts on the LCR
• Increase disinfectant dose to meet CT requirements. • Enhance DBP precursor removal by optimizing coagulation. • Disinfectant change.	• Alters water chemistry for treated water. • May impact the corrosion rate and metal release of lead and copper.

Adjusting pH for optimal NOM removal and reduced DBP formation (i.e. decreasing pH for enhanced coagulation) can increase the corrosion rate by disrupting the films and scales that accumulate on distribution system piping. A change in the coagulant type (i.e. from alum to ferric chloride or reverse) and/or dose may impact the levels of chloride and sulfate ions present in the finished water. These changes may affect the water chemistry in the distribution system an locally lower the pH at the pipe surface encouraging galvanic corrosion between solder and copper, thus leading to corrosion control issues under the LCR.

Lastly, systems that must switch to a different secondary disinfectant or must change coagulants or coagulant dosages to comply with enhanced coagulation requirements under the Stage 1 & 2 DBPR, can control potential increases in lead and copper corrosion by adjusting the pH of the finished water, adding lime or another source of calcium so the finished water is saturated with calcium carbonate and forms a protective coating on the pipes; or adding a corrosion inhibitor (i.e., a substance that is phosphate- or silica-based) to the finished water to form a protective coating on the pipes (USEPA, 1999).

The TCR, the Stage 1 and Stage 2 DBPR and the EWSTRs

Primarily, the simultaneous compliance issues associated with meeting the TCR, the Stage 1 and Stage 2 DBPR and the EWSTRs are due to PWSs switching their secondary disinfectant in order to first meet the Stage 1 and Stage 2 DBPR. However, other issues such as the oxidation of NOM into biodegradable organic carbon (BDOC) for those PWSs using ozone for a primary disinfectant and issues associated with the use of enhanced coagulation or enhanced softening for NOM removal (pH and alkalinity levels) may also result in simultaneous compliance issues.

The majority of PWSs use chlorine as their secondary disinfectant. However, as previously discussed, chlorine reacts with NOM to produce a variety of DBPs, including THMs, HAAs, and brominated products (if bromide ion is present). Use of chloramine as an alternative secondary disinfectant can reduce the formation of DBPs to comply with the DBPR requirements. However, the presence of ammonia from using chloramine may increase the potential for biological nitrification in the distribution system. As mentioned above, nitrification can cause a loss of total chlorine and ammonia residuals, and an increase in bacteria. The greater number of bacteria increases the potential for TCR violations (USEPA, 1999).

The use of ozone as a primary disinfectant results in the partial oxidation NOM into BDOC. These biodegradable organics could provide nutrients for bacteria and promote biological regrowth in the distribution system. The biological regrowth within the distribution system may result in an increased number of total coliform bacteria detections under the existing TCR. An additional problem for those PWSs that use

chlorine as their secondary disinfectant is the potential for continued formation of DBPs by reaction of the biodegradable organics with the chlorine residual in the distribution system. This may lead to compliance issues with the Stage 1 and Stage 2 DBPRs (USEPA, 1999).

Enhanced coagulation and enhanced softening can indirectly influence coliform growth in the distribution system by lessening disinfectant effectiveness when pH is changed. Table 3-10 illustrates the general relationships between changes in pH and the subsequent impact(s) on secondary disinfectant effectiveness.

TABLE 3-10
GENERAL RELATIONSHIPS BETWEEN pH AND EFFECTIVENESS OF DISINFECTANTS USED FOR SECONDARY DISINFECTION

Enhanced Process	Secondary Disinfectant Effectiveness			
	pH Impact	Chlorine	Chloramine	Chlorine Dioxide
Coagulation	Decrease	Increase	Decrease	Decrease
Softening	Increase	Decrease	Increase	Increase

Source: Microbial and Disinfection Byproducts Simultaneous Compliance Guidance Manual, USEPA, 1999.

The enhanced coagulation process requires lower pH levels (generally between 5 and 6) for optimal removal of NOM. The target pH for enhanced coagulation also decreases with decreasing alkalinity. As observed in Table 3-10, chloramine and chlorine dioxide effectiveness decreases with a decrease in pH. Chloramine is destabilized at lower pH levels, resulting in a lower proportion of the mono-species, and more of the di- and tri-species but typically this is avoided by increasing the pH prior to the formation of chloramine.

As previously mentioned above, another potential compliance issue for PWSs that use chloramine as a secondary disinfectant is the increased potential for the occurrence of biological nitrification in the distribution system. Nitrification can cause a loss of total chlorine and ammonia residuals, and an increase in bacteria. The greater number of bacteria increases the potential for TCR violations (USEPA, 1999).

To summarize, modifying treatment practices to comply with the Stage 1 and 2 DBPR and ESWTRs may cause violations of the TCR. These problems can arise from a number of changes to the chemistry and biology of the distribution system which may include the following:

- pH changes resulting from Stage 1 and 2 DBPR compliance which may affect the disinfection effectiveness of chlorine and chloramine.
- Increases in the amount of substrate available for biological regrowth in the distribution system when the primary disinfectant is changed to ozone.
- Changes in disinfectants to comply with DBP limits, such as chloramine for secondary disinfection. Chloramine can provide the ammonia for regrowth if the process is not controlled (USEPA, 1999).
- Switching back from chloramine to chlorine could make it difficult to maintain a chlorine residual.

The Stage 2 DBPR and the LT2EWSTR

Possible issues that may arise through simultaneous compliance with the Stage 2 DBPR and the LT2ESWTR involve a multitude of treatment or operational changes that may impact the distribution system. Some of the treatment changes or operational changes involve modifying or optimizing current operations or installing new technologies such as GAC or membrane filtration. Alternative disinfection strategies may also affect compliance with the Stage 2 DBPR and the LT2EWSTR and other regulations under the SDWA.

Some of these treatment or operational changes may include the following:

1. Modifying pH.
2. Change in reduction/oxidation potential (redox)
3. Switching coagulant.
4. Switching disinfectants (either primary and/or secondary disinfectants).
5. Modifying chlorine dose.
6. Installing GAC.
7. Installing membrane filtration.

8. Installing ozone as a pre-oxidant.
9. Enhanced coagulation or softening.

These treatment or operational changes have potential benefits and issues that may affect the PWSs ability to simultaneously comply with the Stage 2 DBPR and the LT2EWSTR. A summary of the benefits and potential conflicts associated with the most common system modifications and compliance technologies used to comply with the Stage 2 DBPR and LT2ESWTR is illustrated in the *Simultaneous Compliance Guidance Manual for the Long Term 2 and Stage 2 DBP Rules* published by the EPA (Exhibit 2.3).

The manual is an excellent resource and provides a comprehensive overview of technical information for water systems and states to assist them with complying with the Stage 2 DBPR, the LT2ESWTR and other SDWA regulations. The manual can be found at the following website:

http://www.epa.gov/safewater/disinfection/stage2/pdfs/guide_st2_pws_simultaneous-compliance.pdf

Before considering a treatment or operational change to achieve compliance with the LT2ESWTR and/or Stage 2 DBPR, water systems should make sure their water sources are well managed for both quantity and quality and existing treatment processes are working well. For surface water systems, treatment plant performance should be optimized for DBP precursor removal, filtered water turbidity, and disinfection for inactivation of microbial pathogens.

3.3 VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION AND DEPARTMENT OF HEALTH

The Vermont Department of Environmental Conservation's (DEC) manages water quality and issues most of the state's environmental permits and assures compliance with state and federal regulations. In addition to the regulations set by the EPA and the SDWA, the State of Vermont, under the DEC's Water Supply Division, created and administers regulations under Chapter 21 of the Vermont Environmental Protection Rules called the Water Supply Rule. The final adopted Water Supply Rule (WSR) was revised and published in April, 2005. The WSR contains regulations pertaining to the administration of permits, source water protection, drinking water

quality requirements, facility operating requirements, reporting requirements, and water system design standards. The WSR is intended to regulate water systems in the state so they provide clean and safe drinking water to Vermont's citizens.

The WSR applies to all water systems in Vermont, which include PWSs, bottled water systems, Non-Public water systems, and privately owned water sources. Only portions of the WSR apply to each type of water system. The Rule also establishes well construction standards which apply to every constructed well in Vermont regardless of the type of facility it serves. By implementing the WSR, Vermont qualifies to retain "primacy" for the Safe Drinking Water Act from the EPA. The Rule refers to and adopts the authority of the Federal Safe Drinking Water Act: 42 U.S.C. 300 f. et. seq. and 40 CFR, Parts 141, 142, and 143, under an agreement with the EPA.

As it relates to DBPs, the DEC administers the provisions of 40 CFR, Part 141, Subpart C, and Section 141.30 for Trihalomethanes Sampling, Analytical and Other Requirements. The DEC also administers Subpart L, Disinfectant Residuals, Disinfection Byproducts, and Disinfection Byproduct Precursors. Public water systems which add a chemical disinfectant to the water in any part of the drinking water treatment process must comply with these provisions. No additional regulations under the WSR regarding DBPs are administered.

The DEC in conjunction with the Vermont Department of Health (DOH), has been on the forefront of educating stakeholders regarding the use of chloramine as a secondary disinfectant by public water systems. Valuable sources of information can be found on the DEC website regarding chloramination including; fact sheets, press releases from the DOH with expanded information and resources about the use of monochloramine as a drinking water disinfectant, and a link to the EPAs website where a multitude of information regarding chloramination is located.

The DOH has a dedicated webpage regarding DBPs and monochloramine. The DBP website contains multiple links to general information regarding DBPs and contains health information on other chemicals used for disinfection of public water supplies (i.e. chlorine, ozone, chlorine dioxide, etc.). In addition to the breadth of information available on the DBP website, the monochloramine website contains extensive information regarding the use of monochloramine as a disinfectant. The site contains background information regarding monochloramine, and contains a thorough discussion regarding the use of monochloramine in the State of Vermont, a review of health concerns, a health care provider summary, and a summary of the Centers for Disease Control & Prevention (CDC) visit to Vermont, and the corresponding report by the CDC.

The Champlain Water District (CWD) began using monochloramine in April 2006. Since then, the DOH has heard from a number of citizens regarding health concerns that may be related to monochloramine in their drinking water. Community members reported a wide range of health symptoms, including upper respiratory symptoms, watery eyes and nose, scratchy throat, gastrointestinal ailments, skin rashes, and "itchy" skin. Officials from the DOH began meeting with representatives from concerned citizens groups and the CWD to better understand their concerns, and provided testimony to the Vermont Legislature. An investigation of these complaints was conducted jointly by the Center for Disease Control and Prevention (CDC) and U.S. Environmental Protection Agency (US EPA) on September 25, 2007. Results of the investigation were reported by the CDC on January 16, 2008. The report concluded that there was no baseline data on which to base the analysis and that strong biases were present with the data that made interpretation difficult - "Not having baseline data, the post hoc analysis is difficult especially since there are strong biases with the data." In addition, the DOH medical and toxicological experts have spoken with many citizens who have called with health concerns, and continue to review all available scientific articles and studies. To date, the Health Department has found no credible evidence to show that the use of monochloramine to disinfect public water is a threat to public health or has been associated with adverse side effects. However, it is possible there are sensitive subpopulations of chloramination as well as chlorine which have not been documented to date.

SECTION 4.0 - EXISTING AND EMERGING TECHNOLOGIES

The overall goal of this section is to discuss the existing and emerging approaches, technologies, and philosophies to control DBPs, while maintaining water quality that meets the regulatory requirements discussed in Section 3. The discussion has been divided into five subsections:

1. A brief overview of the purpose of disinfection, DBP formation and strategies to reduce them.
2. Actions that can be taken to protect the source water.
3. Physical and chemical treatment technologies.
4. Disinfection processes and injection locations.
5. Effect of treatment and disinfection on the distribution system and water quality.

4.1 PURPOSE OF DISINFECTION AND DISINFECTION BYPRODUCT FORMATION

4.1.1 Disinfection Purpose

Disinfectants are primarily used in water treatment to limit waterborne disease and inactivate pathogenic organisms found in the source water (i.e. lakes, rivers, reservoir, etc.) used by a PWS. The use of disinfectants for this purpose is termed primary disinfection. One of the first disinfectants used in water treatment was chlorine. The use of chlorine as a continuous disinfectant in water treatment dates back to the 1908 starting with Chicago and Jersey City

. Since the introduction of filtration and disinfection at water treatment plants in the United States, waterborne diseases such as typhoid and cholera have been virtually eliminated.

Disinfectants are also used to achieve other specific objectives in drinking water treatment. These other objectives include nuisance control (i.e. zebra mussels and Asiatic clams), oxidation of specific compounds (i.e. taste and odor causing compounds, iron, and manganese), prevention of re-growth in the distribution system (secondary disinfection for Legionella, etc.), and use as a coagulant and filtration aid.

4.1.2 Disinfection Byproduct Formation

Disinfection byproducts are formed when disinfectants used in water treatment plants react with bromide and/or natural organic matter (NOM) (i.e., decaying vegetation present in the source water). Different disinfectants produce different types and amounts

of DBPs. The type and amount of DBPs produced during water treatment depends largely on the disinfectant type, water quality, treatment processes, contact time, and environmental factors such as water temperature and pH. Disinfection byproducts for which regulations have been established by the EPA include trihalomethanes (THMs), haloacetic acids (HAA5), bromate, and chlorite.

THMs are a group of four chemicals that are formed primarily when chlorine used to disinfect drinking water reacts with NOM and inorganic matter in water. The regulated THMs are chloroform, bromodichloromethane, dibromochloromethane, and bromoform. As mentioned previously in Section 3, the EPA has published the Stage 2 DBPR to regulate total trihalomethanes (TTHMs) at a locational running average level of 80 parts per billion, calculated for each pre-established monitoring location. This standard replaces the previous standard set by the Stage 1 DBPR of a maximum allowable running annual average of 80 parts per billion for all pre-established monitoring locations (USEPA, 2009).

Haloacetic acids are a group of chemicals that also are formed primarily when chlorine used to disinfect drinking water reacts with NOM and inorganic matter in water. The regulated haloacetic acids, known as HAA5, are: monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid. The EPA has published the Stage 2 DBPR to regulate total HAA5 at a locational running average level of 60 parts per billion, calculated at each pre-established monitoring location. This standard replaces the previous standard set by the Stage 1 DBPR of a maximum allowable running annual average of 60 parts per billion for all pre-established monitoring locations (USEPA, 2009).

Bromate is a chemical that is formed when ozone used to disinfect drinking water reacts with naturally occurring bromide found in the source water. The EPA has established the Stage 1 DBPR to regulate bromate at a running annual average of 10 parts per billion for those water treatment plants that use ozone as their primary disinfectant.

Chlorite is a byproduct formed when chlorine dioxide is used to disinfect water. The EPA has published the Stage 1 DBPR to regulate chlorite at a monthly average level of 1.0 part per million for those water treatment plants that use chlorine dioxide as their primary disinfectant.

In most source waters, NOM is the major constituent of organic substances and DBP precursors. Total organic carbon (TOC) is typically used as a surrogate measure for precursor levels and is used in the Stage 1 DBPR to determine precursor removal compliance. Dissolved organic carbon (DOC) and UV absorption at 254 nm (UV254) are also often used as surrogate parameters for monitoring precursor levels.

Studies conducted with different fractions of NOM have indicated the reaction between chlorine and NOM with high aromatic content tends to form higher DBP levels than NOM with low aromatic content. For this reason, UV254, which is generally linked to the aromatic and unsaturated components of NOM, is considered a good predictor of the tendency of source water to form THMs and HAAs.

Specific ultraviolet light absorbance (SUVA) is also often used to characterize aromaticity and molecular weight distribution of NOM. This parameter is defined as the ratio between UV254 and the DOC concentration of water. It should be noted the more highly aromatic precursors characterized by high UV254 in source waters, the easier it is to remove these precursors by coagulation. Therefore, the UV254 measurement immediately upstream of the point(s) of chlorination within a treatment plant can be directly related to THM and HAA formation potential (MWH, 2005).

4.1.3 Strategies for Reducing Disinfection Byproduct Formation

The EPA has published numerous manuals and reference material regarding strategies for reducing DBP formation, simultaneous compliance with multiple SWDA rules, and several other guidance manuals that may help PWSs with their process needs. Two guidance manuals in particular provide numerous strategies for reducing DBP formation; the *Alternative Disinfectants and Oxidants Guidance Manual* and the *Simultaneous Compliance Guidance Manual for the LT2ESWTR and the Stage 2 DBPR*. These manuals identify the most up-to-date water treatment techniques, disinfection

technologies and plant modifications that PWSs may use in order to control the formation of DBPs. A few of the principal treatment modifications identified by the EPA in these manuals include the following:

1. Moving the point of chlorination downstream in the water treatment plant.
2. Improving the coagulation process to enhance the removal of DBP precursors (i.e. NOM).
3. Using chloramine to supplement or replace the use of free chlorine.

Moving the point of chlorination downstream in the treatment train is very effective in reducing DBP formation, because it allows the NOM precursor concentration to be reduced during treatment prior to chlorine addition. Replacing prechlorination with an alternate pre-oxidant that produces less chlorinated DBPs is another option for meeting the DBPRs (USEPA, 2007).

Improving the coagulation process to enhance the removal DBP precursors, either by increasing the coagulant dose, changing the coagulant, changing the pH or adding a polymer, has been demonstrated to be an effective strategy to reduce DBP precursors. However, these practices may adversely impact the filtration process, corrosion control, and residuals handling processes (USEPA, 2007).

Using chloramine to supplement or replace the use of free chlorine can lower DBP formation, are more persistent than free chlorine residuals, and help to control the growth of biofilms in the distribution system. Replacing chlorine with chloramine as a secondary disinfectant typically reduces TTHM levels 40 to 80 percent. Conversely, chloramine is not as effective as the disinfectants in the bulk water, as other disinfectants, such as chlorine, ozone, and chlorine dioxide. Also, chloramine cannot be used to effectively oxidize iron, manganese, and sulfides. The use of chloramine may also lead to nitrification in the distribution system. Nitrification can lower the alkalinity and pH of the water in the distribution system and lead to increased corrosion and metal release; especially for lead and copper pipes and appurtenances. This may also lead to violations under the Total Coliform Rule (TCR). In addition, there have been documented taste and odor issues associated with the use of chloramine, if operated improperly as well as

creating biofilms. This is also true for free chorine (USEPA, 2007). In addition, chloramines require notification when this is chosen as the secondary disinfection due to negative effects with fish and dialysis patients. Beside the three treatment modifications mentioned above, there are a number of other strategies that PWSs can use to reduce DBP formation. These strategies include:

- Improvements in source water quality.
- DBP precursor removal by other physical and chemical treatment processes.
- Alternative disinfection strategies.

4.2 SOURCE WATER

4.2.1 Introduction

The objective of this section is to provide a general overview of common source water characteristics, describe present strategies that are used to protect source water quality, and present a brief summary of current development and research that is occurring in this area.

Source water considerations are an important topic since the quality of the source water influences the nature of the treatment process required to produce the desired finished water quality. For example, excess nutrient loading (nitrogen and/or phosphorus) from either point or non-point sources to source waters can result in an algae bloom and increased NOM levels. Non-point sources such as run off from agricultural sites can be a source of protozoa contamination. Further examples of point and non-point sources are illustrated in Table 4-1.

Source water is defined as water obtained from a watershed. The watershed is defined as the water producing area upstream of where water is withdrawn by the supplier. Land areas contributing to both the surface and groundwater components are considered part of the watershed. This includes water storage in the upper watershed, creeks, and natural lakes. Watershed protection is the prevention of pollution to any lakes, rivers, streams, and groundwater that may serve as a source for drinking water (USEPA, 2007).

4.2.2 Source Water Characteristics

Both human activity and natural processes affect source water quality. These influences will dictate which methods of source water protection can be implemented. As listed in Table 4-1, there are numerous natural factors and human factors that can affect source water quality. The natural factors are inherent to the watershed. Human factors affecting water quality are separated into point sources and non-point sources. Point sources are specific locations of where contamination can occur such as wastewater treatment or industrial discharges, pipe discharges from urban runoff, or specific sites where erosion is occurring. Point sources are easier to control due to their specific location. Non-point sources can include agricultural sites (fertilizer runoff from crops), forestry and timber harvesting areas, cattle grazing areas, or recreation activity areas.

TABLE 4-1
FACTORS AFFECTING SOURCE WATER QUALITY

Natural Factors	Human Factors	
	Non-Point Sources	**Point Sources**
Climate	Agricultural cropland runoff	Industrial discharges
Topography	Livestock/grazing	Wastewater discharges
Geology	Dairies and feedlots	Hazardous waste facilities
Soil cover	Urban development runoff	Mine drainage
Vegetation	Septic tanks	Spills and releases
Fire	Erosion	Urban runoff that enters drainage system
Wildlife	Forest management	Combined sewer overflows
Saltwater Intrusion	Mining	
Density/thermal stratification	Recreational activities	
Erosion	Atmospheric deposition	

Table based on Effective Watershed Management for Surface Water Supplies, AwwaRF, AWWA (1991)

Watershed protection is the practice of protecting the surface and groundwater from contamination and minimizing the influence of the factors listed in Table 4-1. The protection is for both present and future users of the watershed. Watershed protection can contribute to the control of DBP precursor levels and to the risk of microbial contamination. The core components of a watershed protection plan are to:

1. Identify the stakeholders, involve them, and define individual goals for watershed protection;

2. Determine common goals with stakeholders. Prioritize goals for watershed;
3. Delineate watershed area both for surface runoff and for groundwater. Identify source water areas to be protected;
4. Identify sources of contamination that may affect those areas;
5. Develop a plan to measure and manage those sources;
6. Look ahead to foresee potential problems (USEPA, 2007).

A multi-barrier approach is a comprehensive approach to providing safe drinking water. It involves the combination of watershed land-use controls, reservoir management, water treatment, disinfection, and water distribution protection. By utilizing some form of watershed protection, a supplier can improve the raw water quality and minimize the impact of negative factors on the subsequent water treatment and distribution processes.

Throughout the United States there are water systems that have limited water treatment, some with only disinfection, and some with no treatment. In some cases, good raw water quality has delayed the implementation of water treatment. The perception of many of these communities is that their sources are pristine and do not require treatment. The positive aspect of this is that the watershed is protected to a higher level. Alternately, communities with full water treatment plants may not be as vigilant in protection of their watersheds as there is a sense of security with the present treatment process in place.

Surface Water Characteristics

Surface water is defined as all water open to the atmosphere and subject to surface runoff. Surface water supplies are generally developed from streams, rivers or lakes and may be collected in large holding reservoirs for future use. Sometimes their location is developed based on minimizing the need to pump into the distribution system. Surface water sources, particularly, the streams and rivers, are subject to quickly changing water quality characteristics. Events such as snowmelt, high flows or summer storms may result in spikes in turbidity levels, elevated silt and sediment or changes in color. These have an immediate effect on the treatment process.

On an annual basis, surface water quality changes with seasonal variations. Water quality may be the highest during the winter months when the surface water influence is lowest. Conversely, taste and odor problems may be experienced under ice conditions.

Alkalinity is typically highest during these months with the greater contribution through groundwater flow. Times when there is a high snowmelt, the alkalinity may be the lowest due to the proportionally lower groundwater contribution and the characteristics of the snowmelt such as acidity due to acid precipitation.

Lakes or holding reservoirs can allow the water utility to allow poor quality runoff events to bypass the withdrawal locations for a period of time. There are seasonal changes in these water-retaining structures. In large lakes, there are temperature changes, possible lake stratification, and the opportunity for an anaerobic zone to develop at the lower elevations of the reservoir or lake. This could lead to decay products forming. Lake stratification can lead to seasonal high concentrations of iron and manganese. If there is a nutrient source such as nitrogen or phosphorus, the upper lake levels may develop algae blooms resulting in taste, odor, pH, and alkalinity changes.

Of the major physical processes that occur, the most common one is high flow volume in rivers and creeks. The scouring of banks due to high velocities is a natural occurrence and can result in slope failures and high silt and sediment loading of the watercourse. Localized erosion can also result from improper storm water management. These loadings in turn can cause higher nutrient levels in the receiving waters.

Microbiological Growth (Nutrients)

There are three general states of nutrient levels and microbiological activity in surface water sources known as trophic levels. These trophic levels for surface water lakes and reservoirs are defined as:

- Oligotrophic: low nutrient levels and minimal microbiological activity.
- Mesotrophic: moderate nutrient levels and moderate activity.
- Eutrophic: high nutrient levels and high activity.

Table 4-2 defines characteristics of the trophic levels of reservoirs and lakes.

TABLE 4-2
LAKE NUTRIENT CHARACTERISTICS

Water Quality Parameter	Oligotrophic	Mesotrophic	Eutrophic
Total phosphorus, µg/L	<10	10 – 20	>20
Chlorophyll a, µg/L	<4	4 – 10	>10
Secchi depth, m	>4	2 – 4	<2
Hypolimnetic oxygen, percent saturation	>80	10 – 80	<10

Thomann, R. V., and J. A. Mueller, *Principles of Surface Water Quality Modelling and Control*, Harper and Row, New York, 1987;

Organic Influences on Surface Waters

The nutrient and organic loading to surface waters is a natural process. These organic compounds include humic substances and fulvic acids.

Humic Substances are the organic portions that remain after prolonged decomposition of wood, leaves or other plant matter. They can impart a yellowish-brown tea color or brownish-black color to the water. They are classified based on solubility into three categories:

1. Humin: An extracted plant residue with a strong base from humus that is acidified. It is typically black in color, has a high molecular weight, and has a lower degree of solubility than the humic and fulvic acids.

2. Humic acid: The organic fraction which precipitates from the acidified solution of humus. It is usually dark brown to gray-black in color with chemical properties between fulvic acid and humin.

3. Fulvic acid: The dissolved portion of the acidified solution. It is light yellow to yellow-brown in color, has a lower molecular weight and a higher degree of solubility than the other humic substances.

These substances react with chlorine to form DBPs that are present in virtually all chlorinated water supplies. The amount of algae in the source water can influence the type of DBPs formed. Algae tend to be precursors for HAAs rather than THMs. In these situations, HAA levels may increase during periods of high amounts of algae in the source water, to levels that equal or exceed the THM levels.

Color

Color can be caused by a number of substances in the water. It is measured as either "apparent color" or as "true color". True color is defined as the color of water from which the turbidity has been removed (Standard Methods 19th Edition). Standard Methods provides for turbidity removal through either filtration through a 0.45 micron filter or centrifugation. True color is found in surface waters with its source being decaying organic matter. Groundwater does not usually have color, except when the aquifer flows through a layer of buried organic matter or iron rich ground. Color, although an aesthetic parameter is an indicator of NOM in the source water. Traditional treatment methods for color control include preoxidation, activated carbon filters, or chemical coagulation and filtration. Chemical coagulation would utilize either some form of iron or aluminum based salts such as ferric chloride or aluminum sulfate (alum).

Giardia* and *Cryptosporidium

Microbiological contaminants may originate from wildlife activity, cattle grazing or point source contaminants in the watershed. The most prominent parasites are *Giardia* and *Cryptosporidium*. It is noted that *Giardia* and *Cryptosporidium* are present in nearly all surface water sources. A 1991 AwwaRF Study illustrated these parasites were present in 98 percent of surface waters. *Cryptosporidium* was present in 87 percent of the 66 surface water sources sampled. *Giardia* was present in 81 percent of the locations sampled. It is also noted that the highest incidences occur where the source water received discharges from industrial wastes or treated municipal sewage effluent.

Statistical models for occurrence of these parasites suggest that levels may be associated with water quality parameters including turbidity and total and fecal coliform levels.

Groundwater Characteristics

Groundwater varies with each location and is dependent on the hydrogeological conditions that exist in the substrata. Groundwater can have a higher development cost than surface waters and a higher on-going supply cost due to pumping. In certain strata, the water quality, however, may be higher, resulting in lower treatment costs. Typically if the aquifer is protected, deep enough, and has low risk of surface contamination, the groundwater will have low turbidity, bacteriological and organic levels. This natural

filtering process allows wells to typically have reduced disinfection requirements (virus only) compared to surface waters. Alternately, the level of metals such as iron or manganese may result in treatment being required. Alkalinity, pH, turbidity, color and other water quality parameters are typically stable throughout the year.

Groundwater from springs, infiltration galleries, and shallow wells may be hydraulically connected to surface water sources. These sources pose a risk of contamination from septic tanks, leaching of herbicides, pesticides and organic contaminants. More discussion on groundwater sources is provided in Section 4.6.

4.2.3 Strategies for Source Water Protection

Watershed management can provide long-term benefits to the water system by helping to reduce the loading of DBP precursors and nutrients into source waters. However, the introduction of watershed management practices often does not have an immediate effect on TTHM or HAA5 concentrations.

As a starting point, a PWS should identify nonpoint and point sources of organic matter in the watershed. The PWS should delineate the watershed boundary and map out land uses, locations of permitted discharges, storm drains, other significant polluters, as well as natural sources of organic matter. Locations of potential sources of organic matter and other DBP precursors (or sources of DBPs that have already been formed) should be identified relative to the locations of tributaries to the reservoir. This should help watershed managers prioritize efforts to control inputs that are more likely to contribute to TTHM and HAA5 formation. Controlling organic contamination that is likely to immediately impact the intake should be given the highest priority.

Many successful watershed management programs rely on a committee of stakeholders working together to improve a lake or reservoir's water quality. Public water system representatives should consider coordinating with stakeholders such as:

- Local soil and water conservation districts.
- Nonprofit conservation groups.
- Farming organizations.

- Fish and game commissions.
- Officials from towns located in the watersheds.
- Other groups that may have an interest in land and water management.

By forming a watershed committee that meets regularly, committee members can identify the various issues and interests that need to be addressed in order to more effectively control nutrient and organic loading that contributes to TTHM and HAA5 formation. The EPA provides technical tools for watershed management at the following website:

http://www.epa.gov/owow/watershed/tools/.

Additional key elements that a PWS should include for a successful watershed management program include an in-depth understanding of the hydrology, land use and land ownership within the watershed, source water protection regulations within the watershed, proper implementation of source water monitoring programs, the development of a source water risk assessment, and the development and implementation of a emergency response plan.

Hydrology: Understanding of the capacity of the watershed on an annual basis is essential knowledge for a water utility. Drought forecasting, estimating flood levels for rivers and streams, and understanding the climate and water balance for the region, form the basis of knowledge for surface water sources. For groundwater supplies, the shape, size, and depth of the aquifer along with factors affecting its renewal, stability, and capacity, form the hydrogeological knowledge base. From this base, informed decisions on supply and quality can be developed.

The cost to carry out detailed hydrological or hydrogeological assessments varies with the size and characteristics of each watershed. For example, the more urban development and multiple land uses that are present, the costs are typically higher.

Legislation: The framework to protect the watershed from human activities is typically set at the local (i.e. local water conservation districts, town/city government) and state level (i.e. environmental department). The water utility should understand the local and state regulations in order to maximize their use for maintaining or improving raw water quality. In most instances, the state's environmental department creates policies or regulations for watershed management, source monitoring, licensing, and related activities in the watershed. For example in the State of Vermont, the delineation of Public Water Source Protection Areas (SPA) has been required for all proposed new water sources for Public Community Water Systems since 1985. Since 1992, a Public Community and Non-Transient, Non-Community Water System must have an approved Source Protection Plan (SPP**)** in order to receive an Operating Permit. Those systems which do not have a SPP are issued a Temporary Operating Permit that includes a schedule of compliance for submission of a Source Protection Plan. Generally, involvement from the water utility and the many stakeholder groups (such as landowners) may be required to find a solution for protecting water resources and meeting the objectives of the many stakeholders.

Public Involvement: Educational programs through the schools and mail-out pamphlets with the utility bills are very effective means of informing the public of issues in the watershed. Recreational activities, forestry and agriculture are typically the largest influences on a watershed. Involving the public in the development of source water protection plans provides the opportunity to channel the energies of the public into efforts to protect their water supplies. Involving the public is critical to building support for the protection program. The stakeholders' meetings with these parties must find common ground on how to protect the water quality. The largest cost in development of protection plans is often the time commitment required by staff.

Source Monitoring: Monitoring of specific water quality parameters provides information on whether or not specific contaminants are of concern. To determine changes in water quality conditions that may impact DBP levels and precursor removal, systems using either surface or ground water sources should consider monitoring the following parameters:

- Total organic carbon (TOC).
- Specific ultraviolet light absorbance (SUVA).

- Temperature.
- Bromide.
- Alkalinity.
- pH.

Systems using surface water sources may also find it useful to measure additional parameters such as turbidity and color to help identify conditions that adversely affect water treatment, such as storm events, stratification, and turnover. To detect the early stages of algal blooms, surface water systems should also consider measuring algae counts, chlorophyll *a*, and nutrients (particularly phosphorus).

Sampling protocols for frequency, location and methodology are critical for a monitoring program. Strategic locations for sampling can help to identify where pollutants or contaminants originate.

Wildlife Control: Excessive populations of sea gulls, geese, deer, and beavers may contribute to microbiological loading in surface waters. Depending on the levels of contamination or the level of protection provided downstream, the removal and/or relocation of these populations may be necessary. Depending on the size of the watershed, management of these contaminant sources may have a limited impact.

Forest Management: Management of timber resources are an important step in protecting the watershed. Setbacks from the riparian zone, methods for activation and deactivation of logging roads, size of cut blocks, and guidelines with respect to acceptable slope steepness, soil type, and logging practices are typically covered by state or federal legislation. Setbacks from streams and rivers allow filtration of runoff through natural vegetation and limits siltation and erosion problems.

Emergency Response: Emergency Response Plans drafted in advance to prepare for emergency conditions regarding source water supply are very important. Emergencies such as landslides, floods, oil or chemical spills, acts of terrorism, or failures of upstream wastewater treatment facilities should be prepared for. Lines of communication between the fire department, police, the water utility and local/state health officials should be in place. Response issues, methods, and protocols should be established.

Risk Assessments: Source water assessments identify all areas of concern within a source water supply, be it groundwater or surface water. Risk assessments are useful in order to evaluate the potential for contamination to the source water. The assessment should include a survey of all significant infrastructure features, activities within the watershed, sources of contamination, and tracking of their movement.

On-Going Watershed Inspections: Inspections and site visits by water utility staff provide a visual and sensory review of the condition of the watershed. Use of the watershed stakeholders to protect the natural resource is also advisable. If the watershed is not restricted, making information available to all stakeholders to increase their responsibility and commitment to the natural resource should be an objective. The involvement and responsibility of all stakeholders through visual inspections provides an increased ability to monitor and deal with potential contamination issues before they arise.

Watershed Control/Fencing: Erosion control, control of natural wildlife features, and control of natural erosion features are methods to reduce particulate and bed loading in rivers and streams. Typically problematic erosion areas are difficult to reach resulting in expensive remediation costs. Fencing may be necessary on a local level, primarily to reduce human and animal traffic around drinking water reservoirs.

Optimizing Source Water Withdrawal: Optimization at the site of water withdrawal is important with respect to quality. Poor water quality in a reservoir can result from a number of factors including flooding, thermal stratification, and eutrophication. In some cases, systems can avoid withdrawing poor quality water with high DBP formation potential by optimizing the management of raw water intake operations.

One method for avoiding withdrawing water with poor quality is to have raw water intakes located at several levels. Systems that are able to draw water from multiple depths should consider regularly measuring TOC, color, temperature, turbidity, and bromide to determine which depth is providing the highest water quality. During flood events, systems may hold water longer in reservoirs to allow turbidity associated with

agricultural or urban runoff to settle to lower levels. These systems can then draw water from an alternate intake level where the water quality is better. If thermal stratification occurs, systems can withdraw water from the intake level with the lowest potential for DBP formation. Some PWSs aerate their raw water reservoirs to minimize or prevent thermal stratification, but this option can lead to other water quality problems such as algal blooms and increased dissolved oxygen concentrations (USEPA, 2007).

Reservoir Management: Algal blooms can result in a variety of water quality problems including tastes and odors, shortened filter runs, increased chlorine demand, increased turbidity, pH fluctuations, and, in some cases, increased organic DBP precursors. There are several techniques including aeration, destratification, dredging, and aquatic weed harvesting that have been used with some success for managing eutrophication. Systems may also have the option of utilizing other sources of supply that do not have algal blooms. However, it is uncertain whether any of these techniques significantly reduce organic DBP precursors. Many water systems that use lakes or reservoirs for their surface water supply have been practicing algae control through the use of chemicals, such as copper sulfate.

Generally, it is possible for PWSs to detect the early stages of an algal bloom through an aggressive source water quality monitoring. If detected in the early stages, the algal bloom can be controlled with the use copper sulfate. Public water systems that are considering using copper sulfate should first consult with their state regulatory agency (i.e. environmental department) to determine if it will be allowed. In many states, copper sulfate application requires a pesticide permit application or certified pesticide applicator.

Table 4-3 summarizes other options for controlling water quality in surface water reservoirs.

TABLE 4-3
QUALITATIVE ASSESSMENT OF RESERVOIR MANAGEMENT TECHNIQUES

Management Technique	Short Term Effectiveness	Long Term Effectiveness	Cost	Probability of Negative Impact
Significant watershed management, wastewater diversion and advanced wastewater treatment	N/A	G / E	High	Low
Phosphorus interception	E	E	Med./high	Low
Dilution	G / E	G / E	Med./high	Low
Phosphorus inactivation	G / E	G / E	Medium	Low
Artificial circulation (mixing)	F / G	F / G	Low	Low
Selective discharge	F	F / G	Low	Medium
Copper sulfate	G	P	Med./high	Low
Grass carp	P	E	Low	Low medium
Insects	P	G	?	Low
Sediment removal	E	E	High	Low medium
Drawdown	G	F	Low	Low medium
Harvesting	E	P	Medium	Medium
Herbicides	E	P	Medium	High

E excellent, G good, F fair, P poor ? Unknown

Source: Cooke & Carlson (1989), *Reservoir Management for Water Quality and THM Precursor Control*

Groundwater

The physical characteristics of aquifers vary for every site, as do the threats to water quality. In some of the higher use areas in the United States, groundwater source management programs have been implemented. These "well head protection" plans generally consist of the following elements:

- Analysis of well-field hydrogeology.
- Determination of yields and zones of influence.
- Identification of potential for contamination sources.
- Monitoring wells for early warning.

- Preparation of monitoring and emergency response plans.
- Local institutional approaches for groundwater protection.
- Future licensing requirements.

4.2.4 Development/Research in Source Water Quality

There is a significant amount of research being conducted, some of which has been completed, to investigate methods of improving source water quality. The major funding institutions include the National Research Council, the EPA, and the Water Research Foundation (WRF, formerly AwwaRF). Much of the work is focused on understanding the mechanisms that occur in the environment prior treatment at the water treatment plant.

Riverbank Filtration (RBF): Drawing water through the riverbank rather than directly from the river or stream, helps remove pollutants, pathogens, viruses, and color. The success of this technique is dependent on the gradation of soils along the stream and the silt and sediment loading within the waterbody. The effectiveness of riverbank filtration for water treatment was investigated through a two-year, full-scale, RBF study conducted through the Water Research Foundation in Louisville, Kentucky. The study entitled, "*Evaluation of Riverbank Filtration as a Drinking Water Treatment Process*" (Project # 2622) was completed in 2001. The results of the study showed the RBF process was very effective for removing microbial contaminants in the river water. At the project site, 3.0 log removal of Cryptosporidium was demonstrated. The RBF process was also very effective for removing NOM in river water. The overall reduction of NOM as measured by TOC and DBP formation potentials was 40% to 60% through the RBF process. It was stated that both biological degradation and physical-chemical removal played important roles in the NOM removal. It was also noted that some mechanical clogging occurred during the first two years of pumping, which reduced the yield of the RBF well.

Virus Inactivation in Soils: Another WRF study entitled, *"Virus behavior in Saturated and Unsaturated Subsurface Media*" (Project #353) was completed in 2002. The goals of the study were to measure the inactivation rates of indicator bacteriophages and human enteric viruses in both saturated and unsaturated soils and aquifer materials, measure the inactivation rates of these microorganisms in soils under a variety of environmentally

relevant conditions, and identify key mechanisms responsible for the inactivation of microorganisms in the subsurface soil environment. The study concluded there is a very high variability in virus retention and transport through different soils and aquifer materials, depending on their properties, water quality, and the degree of water saturation. The study also concluded that it is very difficult or impossible to predict viral transport behavior in porous media. These results may impact existing groundwater systems that must meet regulations under the new Groundwater Rule to provide 4 logs removal of viruses.

***Cyst and Oocyst Survival*:** The University of Ottawa and the AwwaRF completed a study in 1999 that observed the effects of environmental factors that influence the survival or inactivation of protozoan parasite cysts and oocysts, and their susceptibility to subsequent disinfection. These factors include the physical, chemical and biological factors prevailing in natural or managed water environments, and the stressors that may be encountered or applied during in-plant treatment. Parasite survival experiments were performed in standard hard water as well as in natural water samples from different watersheds in Canada and the U.S. Findings of this study showed that *Cryptosporidium* oocysts survive well and travel far from the pollution source. This emphasizes the importance of applying best management practices throughout the watershed.

In addition the study concluded that aged/stressed oocysts were as resistant to disinfection as freshly excreted ones. Thus, it is important for utilities to implement effective removal and disinfection practices.

Source Assessments: Numerous assessments are underway to better understand the watershed processes affecting water quality. Much of the research is directed towards obtaining a better understanding of the mechanisms that result in microbial pollution in the watershed and the related factors in external and internal sources of NOM.

Watershed Analysis through Databases: An extensive watershed management program called Better Assessment Science Integrating Point and Nonpoint Sources, or BASINS is available from the USEPA. BASINS was originally introduced in 1996, and improved versions were released in 1998, 2001, 2004, and 2007. BASINS is a multipurpose

environmental analysis system designed for regional, state, and local agencies that perform watershed and water quality-based studies. This system makes it possible to quickly assess large amounts of point and non-point source data in a format that is easy to use and understand. Installed on a personal computer, BASINS allows the user to assess water quality at selected stream sites or throughout an entire watershed. This tool integrates environmental data, analytical tools, and modeling programs to support cost-effective approaches to watershed management and environmental protection, including the development of Total Maximum Daily Loads (TMDLs) (USEPA, 2009).

The new BASINS 4.0 was updated in 2007 to include the use of open source GIS software architecture. Analysts can now use BASINS 3.1 or BASINS 4.0 to examine environmental information, analyze environmental systems, and build a framework for assessing management alternatives (USEPA, 2009).

4.3 PHYSICAL AND CHEMICAL PARAMETER TREATMENT PROCESSES

4.3.1 Introduction

The EPA and American Water Works Association have been very active on researching and publishing information on the DBP issue. Originally the interest had been on chlorinated organic compounds and focused on trihalomethanes more recently there has been activity focused on nitrogen containing DBPs which are believed to be more prevalent in systems using chloramine. Over the years, the activity to control and study THMs has led to alternative means of disinfection and alternative DBPs. The EPA has published the results of these activities in various documents and has defined and refined an approach that this report follows.

It is commonly understood that, disinfection byproducts are formed as a result of either reactions between the disinfectant and certain compounds in the source water or as a natural decay product of the disinfectant itself. The following three strategies for DBP control have been the focus for the last several years:

- Treatment to remove DBP precursors prior to disinfection.
- The use of alternate disinfectants.
- Treatment to remove the DBPs.

Currently, the water treatment industry is focusing on treatments that remove DBP precursors prior to disinfection and the use of alternate disinfectants. This section presents and summarizes the published information related to the current treatment technologies, process changes to the current technologies and emerging technologies as they apply to the removal of DBP precursors and DBPs. The interactions of unit treatment processes were discussed as they apply to precursor or DBP removal.

The primary advantage of removing DBP precursors is the general reduction of material (i.e. NOM) the disinfectant can react with. This results in lower observed DBP concentrations. Situations do exist where precursor removal is not practicable due to competing water treatment objectives, such as zebra mussel control.

Other factors will also affect the formation of DBPs. For example, maintaining a disinfectant residual in the distribution system can result in continued DBP formation unless significant removal takes place at the water treatment plant prior to reaching the distribution system, or alternative disinfectants are used.

4.3.2 Preoxidation

Preoxidation is practiced to minimize operational problems associated with biological growth on filters, pipes or tanks, iron and manganese control, taste and odor control, or color control. This report discusses those effects relating to the use of preoxidation and DBP formation.

NOM molecules that are known precursors to a given DBP can be partially oxidized to form molecules that are not precursors to that DBP. However, NOM molecules that are not precursors to a given DBP can be oxidized to molecules that are precursors to that DBP. Therefore, some DBP precursors are partially destroyed by oxidation while others are created. The removal of DBPs created by preoxidation is discussed under the each treatment process below.

4.3.2.1 Prechlorination

Although there are concerns over DBP formation, prechlorination is still practiced by many utilities, particularly to control zebra mussels. Prechlorination may also impact NOM removal. Bench-scale studies conducted by Johnson and Randtke (1983) as found on (Awwarf, 1999) indicate that prechlorination will reduce the amount of NOM removal achieved with coagulation.

USEPA, (2007) studied DBP removal with enhanced coagulation with prechlorination. The results of this study showed reduced THM levels at 9 plants by an average of 17 percent and reduced HAA5 levels at 6 plants by an average of 4.7 percent. The point of prechlorination was no more than three minutes ahead of the rapid mix process at these 15 plants.

The EPA considered prechlorination during the preparation of the Stage 1 DBPR. The EPA concluded that the majority of utilities that currently practice prechlorination will require additional contact time or other process changes, for microbial protection if prechlorination is discontinued. New facilities generally avoid prechlorination in order to prevent the formation of THMs.

4.3.2.2 Preozonation

Ozonation does not produce a residual that survives through the distribution system, nor does it produce chlorinated byproducts. However, when added to raw water, ozone reacts with NOM and bromide to form various byproducts that must be considered in the control of DBPs. The EPA *Guidance Manual on Alternative Disinfectants and Oxidants* (1999) reports that DBP precursor removal and DBP formation is dependent on the ozone dose, bromide levels, pH, alkalinity, and the nature of the NOM.

Direct oxidation of NOM by ozone results in the production of a variety of organic byproducts including aldehydes, keytones, carboxylic acids, and unstable peroxides. Ozonation generates biodegradable organic matter through the oxidation of more complex compounds. Preozonation of raw water that contains

bromide and NOM can produce both organic and inorganic bromated byproducts including brominated THMs, brominated acetic acids, brominated acetontriles, bromopicrin, cyanogen bromide, and bromate.

Various researchers concluded that although preozonation alone has a negligible impact on the overall TOC concentration of raw water, the organic material is altered such that color and UV absorbance of the water are reduced. Both pilot and full-scale studies have shown that the use of preozonation in place of prechlorination may result in finished water THM concentrations that are as much as 85 percent lower (AWWA, 1999).

Researchers concluded that, at ozone doses less than 0.7 milligrams of ozone per milligram of TOC, NOM removal during conventional treatment is improved; and at higher doses, removal is impaired. Ozonation can partially oxidize NOM to lower weight molecules at doses typically applied for disinfection. This reaction can have a negative impact on NOM removal by conventional treatment. However, the lower molecular weight NOM is more readily biodegraded by biologically active filtration. In addition, smaller molecules may be more readily removed by activated carbon. Failure to remove the smaller organic molecules before the water leaves the treatment plant can result in increased bacterial growth and re-growth in the distribution system.

DBPs formed during ozonation of water containing NOM and bromide is of particular interest. Increasing the amount of NOM has been shown to increase the formation of brominated organics. Conversely, the reverse relationship has been shown between NOM concentrations and bromate formation. Bromate formation is a primary concern when utilizing source water with a bromide concentration of more than 0.10 milligrams per liter. An increase in initial bromide concentration results in increases in both organic DBPs and bromate (USEPA, 1999).

The pH level has shown opposite effects on organic brominated disinfection byproducts and bromate formation. An increase in pH results in a decrease of organic DBPs, but an increase in bromate.

It has been documented that alkalinity (bicarbonate and carbonate ions) decreases the oxidation rate of organics due to the removal of hydroxyl radicals formed during ozonation. This removal of hydroxyl radicals by bicarbonate ions has been demonstrated to slow the decomposition of ozone, thereby maximizing the exposure of the THM precursors to ozone (MWH, 2005). For example, Langlais, et al., (1991) demonstrated that an increase in alkalinity had a beneficial effect on brominated THMFP. A study of the effects of alkalinity on ozonated byproducts indicated that as alkalinity increased, the formation of dissolved organic bromide compounds was inhibited and no bromate was detected. However, two studies conducted found contradicting evidence. A study by the AwwaRF concluded that where carbonate ions were formed, bromate formation was promoted. Another study conducted by Legube et al, observed a general increase in THMFP with increased ozone dose in the absence of bicarbonate (AWWA, 1999).

Some studies have demonstrated that ammonia addition has a positive effect in reducing bromate formation by tying up hypobromous acid to form bromamine. In the chemical reactions leading to the formation of ozonation byproducts, hypobromous acid is in equilibrium with its conjugate base, hypobromite. When hypobromite reacts with either residual ozone or hydroxyl radicals present in the water, bromate is formed. With the addition of ammonia, the chemical reaction and therefore the equilibrium, is driven to the formation hypobromous acid rather than hypobromite. This shift in the equilibrium reduces the formation of bromate (MWH, 2005).

The principal advantage of using preozonation rather than prechlorination for DBP control is that preozonation also provides pathogen inactivation. This reduces the amount of chlorine or chloramine to be added further in the treatment train as secondary disinfectants. If chlorine is used following ozonation and the oxidized organics are not removed, the oxidized organics may have a negative effect on the formation of chlorinated DBPs. Therefore, removal of oxidized organics to prevent the formation of chlorinated DBPs and re-growth in the distribution system is recommended (AWWA, 1999).

4.3.2.3 Advanced Oxidation Processes

Advanced oxidation processes (AOPs) refer to such processes as combined ozone with hydrogen peroxide, combined UV light and hydrogen peroxide, combined ultraviolet (UV) light and ozone, and combined UV light with titanium dioxide. AOPs involve the generation of very strong oxidizing agents. Of these, hydroxyl radicals are the best known and most studied. Hydroxyl radicals are highly effective oxidants that can react very rapidly with electron-rich organic compounds. These AOPs provide alternatives to ozonation and other conventional oxidants (i.e. chlorine) since they are more effective at converting organic compounds into carbon dioxide, water, and mineral acids (i.e. HCl) (MWH, 2005). Of these processes, UV light and hydrogen peroxide is receiving most of the interest in drinking water treatment at the present time (Malley, 2006).

Ozone and hydrogen peroxide addition, also known as the PEROXONE process is an AOP that was studied extensively in the 1990's for taste and odor control and DBP control. The addition of hydrogen peroxide to ozone in water accelerates ozone decomposition by promoting the production of hydroxyl radicals. These hydroxyl radicals oxidize organic compounds and convert them to carbon dioxide, water, and mineral acids (i.e. HCl). Interest in the ozone and hydrogen peroxide process has diminished in drinking water applications due to operational complexities and issues related to bromate formation (Malley, 2006). However, recent preliminary research conducted by the International Ozone Association (IOA) has shown bromate to be less harmful than previously suspected. Based on these findings and perhaps an increase in the MCL for bromate may provide more opportunities for AOPs with ozone in the future.

UV light and hydrogen peroxide is another AOP receiving most of the interest in drinking water treatment at the present time. The basic components of the UV/hydrogen peroxide process includes hydrogen peroxide injection and mixing, followed by a UV reactor. The irradiation of hydrogen peroxide by UV light breaks the hydrogen peroxide molecule into two hydroxyl radicals. The

production of twice the amount of hydroxyl radicals makes this AOP advantageous. However, the reaction is relatively slow compared to that of the ozone/hydrogen peroxide reaction and requires significant concentrations of hydrogen peroxide to produce the hydroxyl radicals (Malley, 2006).

AOPs in combination with chloramine can successfully reduce the formation of DBPs. However, under typical AOP operating conditions there is little or no ozone residual, which results in little or no CT credit. Therefore, utilities that use AOPs and chloramine will typically use ozone for disinfection, then the AOP for micropollutant destruction, and finally chloramine for residual disinfection (Gramith, et al., 1991).

4.3.2.4 Chlorine Dioxide

Chlorine dioxide is used as a preoxidant to control tastes and odors, iron and manganese, hydrogen sulfide and phenolic compounds. The addition of chlorine dioxide oxidizes organic DBP precursors, thus reducing the formation of THMs and HAAs. However, chlorine dioxide does produce two inorganic DBPs; chlorite and chlorate. Process control monitoring is considered more difficult compared to traditional oxidants such as chlorine due to the wet chemistry skills required to maintain calibration of online meters. The chlorite ion is generally the primary byproduct of chlorine dioxide reduction. The distribution of chlorite and chlorate is primarily influenced by pH. Chlorate is primarily formed through the reaction of residual chlorite and free chlorine in downstream reactions when chlorine dioxide is used as the secondary disinfectant. Chlorate is typically stable in the finished water once formed. Since chlorite is the predominant ion formed from the decay of chlorine dioxide, the EPA set an MCL and MCLG of 1 mg/L and 0.8 mg/L under the Stage 1 DBPR, respectively. Chlorate was considered for regulation under the Stage 1 DBPR, however due to a lack of sufficient health effect studies, chlorate was not regulated under the rule. As a result, the EPA asked the National Toxicology Program to evaluate the health effects of chlorate. The EPA may regulate chlorate in the future. However, with the publication of the Stage 2 DBPR and the continued lack of health effects data, there are no plans to do so in the near future (AwwaRF, 2004).

It has been estimated that approximately 50 to 70 percent (by mass) of the chlorine dioxide applied during drinking water treatment applications is converted to chlorite. Therefore, the formation of chlorite will limit the chloride dioxide dose that can be used during treatment unless chlorine dioxide demand is reduced or removal technologies are implemented. Since the production of chlorine dioxide typically takes place on site, control over the generation process may minimize the chlorite and chlorate formation. Feasible options to remove chlorite include: reduction with ferrous ion, reduction with activated carbon, and oxidation with ozone (USEPA, 1999 and 2007).

Also, free chlorine and dilute chlorite concentrations can react in the distribution system to produce low concentrations of both chlorine dioxide and chlorate. This free chlorine dioxide, even at low concentrations, can escape from the tap causing a strong odor of chlorine. In addition, if new carpet has been recently installed, the free chlorine dioxide may react with the organic compounds released from the carpeting and produce offensive odors similar to cat urine or kerosene (USEPA, 2007).

4.3.2.5 Potassium Permanganate

Potassium permanganate is used primarily to control taste and odors, remove color, control biological growth in treatment plants, and remove iron and manganese and control of Zebra Mussels. Several studies evaluating the reduction in THMFP by using potassium permanganate indicated marginal results (Colthurst and Singer, 1982; Singer, et al., 1980). None of the permanganate dosages were capable of producing a detectable change in TOC. Full-scale evaluations of potassium permanganate were conducted by the Fairfax County Water Authority in Northern Virginia (Bonacquisti and Petrovitch, 1988). The authors of that study found no relationship between permanganate dose and TOC removal; although a 25 percent removal of THM was achieved. Other studies found no relationship between permanganate dose and TOC removal but 10 percent reduction in chloroform. However, permanganate can be used as a pre-oxident to reduce or eliminate the amount of chlorine required prior to filtration, thus reducing chlorinated DBP's.

4.3.2.6 Hydrogen Peroxide

Hydrogen peroxide (H_2O_2) is one of the strongest oxidants available; stronger than chlorine, chlorine dioxide, and potassium permanganate. Also, through catalysis, hydrogen peroxide can be converted into hydroxyl radicals with reactivity second only to fluorine. Hydrogen peroxide is a nearly colorless liquid that is soluble in water in all proportions. It is added to water as a dilute solution and degrades quickly when exposed to sunlight. Commercial solutions are typically 35 percent, 50 percent or 75 percent hydrogen peroxide (MWH, 2005).

Hydrogen peroxides use in drinking water applications can be traced back to the early 1900's. Typically, hydrogen peroxide is used effectively for the oxidation of sulfides and iron and to a lesser degree, manganese. Application points are typically upstream of coagulation/flocculation in the rapid mix stage. Performance has been site specific and generally been hampered by cost and availability. However, variations in activated hydrogen peroxide are being developed which could become more prominent in future applications.

Based on limited information available, it appears the use of hydrogen peroxide alone is ineffective for DBP precursor reduction. However, when hydrogen peroxide is used in conjunction with ozone and UV light as an AOP, increased removal in organic material (DOC) and a corresponding decrease in DBP formation potential has been observed.

Peracetic Acid

Peracetic acid (PAA) is a strong oxidant produced through a reaction between hydrogen peroxide and acetic acid (vinegar). Based on oxidation potential, PAA is a stronger oxidant compared to chlorine and chlorine dioxide. As a product, PAA is typically sold as a 5% or 15% active solution for disinfection/sterilization purposes, particularly in the food processing industry. While used for decades in the chemical and food processing industries, peracetic acid has more recently been considered as an oxidant for water treatment and a disinfectant for wastewater treatment. Specific proprietary solutions of PAA have been approved by the EPA for use in wastewater disinfection such as Proxitane® WW-12 which

is manufactured by Solvay Chemicals. For water treatment, possible uses of PAA in the future include pre-oxidation to remove taste and odor compounds as well as organics removal. For smaller systems, disinfection with PAA may be an alternative compared to chlorine based preoxidants and possibly as a disinfectant. However, no residual typically remains long enough to allow PAA to be a secondary disinfectant.

4.3.3 Conventional Filtration

NOM is composed of suspended, colloidal, and dissolved portions. Conventional rapid filtration is a process that removes suspended particulate material that would otherwise exert a load on the disinfection process and is commonly used for turbidity removal. Another goal of filtration is to remove microbiological contaminants. Coagulation and flocculation are required prior to the filtration process to remove the dissolved and colloidal NOM. Coagulation and flocculation are discussed later in this section.

Primarily, conventional filtration removes suspended NOM including matter that contains precursors of chlorinated DBPs, although some colloidal material will also be adsorbed to the filter media. The dissolved and colloidal portions of the NOM are the portions that contain the significant precursors to DBP formation. Filtration alone will remove a very small portion of unbound or dissolved NOM. The exceptions are where the operation of the filter allows the inadvertent growth of a biological layer on the filter media, or where the facility has been converted to a biologically active unit process. Therefore, conventional filtration alone is not a unit process considered by researchers as a means of removing DBPs (AWWA, 1999).

4.3.4 Biologically Active Filtration

In traditional water treatment practice, biological activity within a filter is discouraged. However, in many systems worldwide, particularly in Europe, biological activity is encouraged in filtration to remove biodegradable organic matter to control DBP precursors and make plant effluent water biologically stable. Studies have shown that Biological Activated Filtration and Ozone/Biological Fluidized Bed Treatment (FBT) effectively reduced DBP precursors; biodegradable dissolved organic carbon, and chlorine demand.

4.3.4.1 DBP Precursor Removal

Slow Sand Filtration

Many United Kingdom and European cities choose slow sand filtration as a water treatment method because of its simplicity, reliability, and economy. The historical use of slow sand filters in the United States is primarily in smaller communities with fewer than 10,000 people, 45 percent of which serve fewer than 1,000 people (Sims, et al., 1991).

Slow sand filtration differs from rapid sand filters not only in the flow rate, but also in the presence of the biologically active layer. Biological growth in the filter bed is discouraged in rapid sand filtration. However, the biological active layer or schmutzdecke, of the slow sand filter is the integral mechanism of the treatment process. The adsorption and biodegradation mechanisms provide removal of natural organic matter and accordingly a reduction in disinfectant demand and DBP precursors.

Field and pilot studies indicate the degree that slow sand filtration removes organic precursor materials is limited. A study of three municipal systems in New England (Collins, 1991) during the winter months, reported NOM removal ranging from 13 to 33 percent, 17 to 33 percent, and 9 to 27 percent, respectively. These results were based on non-purgable dissolved organic carbon (NPDOC), UV absorbance, and THMFP.

Collins (1989) evaluated slow sand filter modifications that could enhance the primary removal mechanisms of biodegradation and adsorption and that would not compromise the simplicity of the treatment process. These modifications included promoting biodegradation potential by increasing bacterial populations with filter depth and through incorporating a cleaning procedure that harrows the filter media thus minimizing biomass removal from mature sand.

Another slow sand filter modification studied by Collins (1996) was the addition of granular media amendments such as anionic exchange resin or granular activated carbon (GAC). Results showed that filters amended with anionic resin and granular activated carbon (GAC) achieved significantly higher (>75 percent) removals of organic carbon and THM formation potential during an 89 day filter run. The principal disadvantages associated with GAC or an anionic resin amended slow sand filter is the rapid head loss and the costs of cleaning and regeneration of the amendment. In addition, a potential problem with anionic resin in that they usually contain a quarternary amine functional groups that can become soluble as if resin degrades. These functional groups when exposed to chlorine can form a nitrogen disinfection byproduct, NDMA.

In addition, as previously discussed, preozonation can degrade NOM into smaller molecular weight organic compounds. The lower molecular weight NOM is more readily biodegraded by biologically active filtration. Therefore, preozonation before the slow sand filtration can increase TOC removal by as much as 35 percent (Rachwal, et al., 1988, Zabel, 1985).

As described above, slow sand filtration systems have multiple advantages. However, some disadvantages of slow sand filters include:

- Considerable land area and structures to prevent freezing are required due to much lower filtration rates compared to rapid filtration.
- Cleaning requires periodic manual removal and refilling of the sand.
- Cold water temperatures significantly reduce the treatment rate.

Granular Activated Carbon

GAC is often used as the support media for biologically active filtration. GAC has the advantage of both the biodegradation removal and adsorptive removal of NOM. Earlier researchers attributed the main difference in overall removal by GAC and non-adsorptive filter media to the adsorptive removal component of the GAC.

Later research, as reported in the EPA study of technologies, concluded that GAC media accumulated more biomass and that the increased biomass could also account for superior removal.

An AwwaRF study published in 2000 regarding filtration in biological filters concluded that sand filters with GAC provided better aldehyde removal at colder temperatures, established a biodegradable organic matter (BOM) removing biofilm more rapidly, provided an increase in protection against oxidant residuals in the filter influent, and permitted a faster reestablishment of BOM removal after periods the filter was out of service. In addition, better DOC and TOC removals were observed compared to sand filters with anthracite.

4.3.4.2 Process Interactions

Unlike the experience with slow sand systems, GAC biofiltration systems were less affected by chlorine, chloramine or chlorine dioxide levels in the feed water.

4.3.4.3 DBP Removal

Biodegradable organic matter (BOM) is formed during ozonation and to a lesser degree during the reaction of chlorine and chlorine dioxide with raw water NOM. BOM is also a DBP precursor since the organic matter can react with chlorine following ozonation if not removed.

Malley, et al., (1993) found that ozone pretreatment at practical ozone doses of 2 to 6 mg/L can dramatically improve THMFP removal via direct oxidation of precursors as well as enhance removal during biological filtration. Removals of 40 to 70 percent were observed, compared to 10 to 15 percent removals compared to conventional slow sand systems. On the other hand, ozone can cause dramatic increases in head loss through accumulation of biomass in the schmutzdecke. Using larger effective size sand media with greater biomass storage capacity and the use of biofilm support structures can reduce the rate of head loss development. While pre-ozonation does produce ozone byproducts

(formaldehyde, acetaldehyde, glyoxal), they are readily removed to below detection limits by biological filtration under proper design and operational (flowrates, etc.) On a cumulative-flow basis, ozone/slow sand is as effective as ozone/flocculation/sedimentation/GAC biological filtration in treating water for potable water production.

4.3.5 Coagulation/Flocculation

The coagulation/flocculation process has traditionally been used for turbidity and color removal. Coagulation/Flocculation treatment is used primarily for hard-to-settle suspended NOM particles, colloidal NOM particles and dissolved NOM to facilitate removal by sedimentation and filtration. In order for coagulation and flocculation followed by sedimentation to be most effective, the selection of the type and dose of coagulant is the primary and most important step in the process. The selection of the coagulant type and dose are dependent on the characteristics of the coagulant, the concentration and type of particulates to be removed (i.e. turbidity), the concentration and characteristics of NOM, the raw water temperature, and the raw water quality (alkalinity, pH, hardness, etc.).

4.3.5.1 DBP Precursor Removal

Coagulation/flocculation followed by sedimentation (either conventional or high rate), dissolved air flotation (DAF), or filtration can remove DBP precursors by the following mechanisms:

1. Colloidal destabilization (typically accomplished by charge neutralization or enmeshment).
2. Precipitation (conversion of dissolved particles to a solid).
3. Coprecipitation (typically by adsorption to surfaces).
4. Clarification followed by filtration.

At most surface water treatment plants using coagulation and flocculation treatment processes, the concentration and characteristics of NOM control the coagulant type and dose. A brief discussion regarding NOM, its characteristics and how it affects the coagulation/flocculation process is discussed below.

NOM Characteristics

The destabilization of the NOM is affected by a number of factors including the nature and concentration of NOM, raw water characteristics, pre-treatment processes, and the coagulant type and dosage.

NOM can be put into categories according to the molecular weight fractions or size ranges. The average molecular weight for aquatic humic NOM ranges from 800 to 3,000 Daltons. What is useful in the discussion of NOM and conventional treatment is the relative categorization of NOM as either high or low molecular weight material. The reactions during conventional treatment are not consistent for all molecular weights. Various researchers reported that larger molecules form more easily removed floc. Ozonation results in the reduction in the number of larger and medium size molecules and an increase in the smaller molecules. Accordingly, ozonation prior to coagulation can have a negative effect on NOM removal as discussed previously in this section.

The second NOM characteristic that affects the conventional treatment process is the charge of the molecules. Neutral molecules tend to remain in solution while negatively charged molecules are more readily destabilized.

Researcher (Edzwald, 1984) analyzed data from 17 operating treatment facilities and determined that the amount of NOM in the water affects the degree of removal. As the amount of NOM increases, the NOM removal also increases. This information is presented as Figure 4-1. As illustrated, the amount of NOM is represented by the total organic carbon (TOC) concentration. As illustrated by the three curves, TOC removals for alum dosages above 20-25 mg/L were not as dramatic as from 0-25 mg/L. Therefore, the minimal gain in TOC reduction may not be worth the additional alum costs.

Coagulant Dose

Removal of NOM as a function of coagulant dosage generally follows one of two behaviors. In Type 1 behavior, NOM removal increases sharply in response to the increasing coagulant dosage. In Type 2 behavior, NOM removal increases gradually with increasing amounts of coagulant. The removal characteristics are pH dependent. Water with a higher pH exhibits a predominant Type 2 removal curve, where coagulant addition alone without pH adjustment, is less likely to achieve high rates of NOM removal. Figure 4-2 presents the graphical results of an equation developed from data obtained from the 17 plants. These curves also show minimal increased NOM removal for pH 5.50, 7.15 or 8.04 beyond an alum dosage of 20-25 mg/L.

FIGURE 4-1
IMPACT OF RAW WATER TOC ON TOC REMOVAL BY ALUM COAGULATION/FILTRATION
(ADOPTED FROM TECHNOLOGIES AND COSTS FOR CONTROL OF DISINFECTION BYPRODUCTS, OCTOBER 1998)

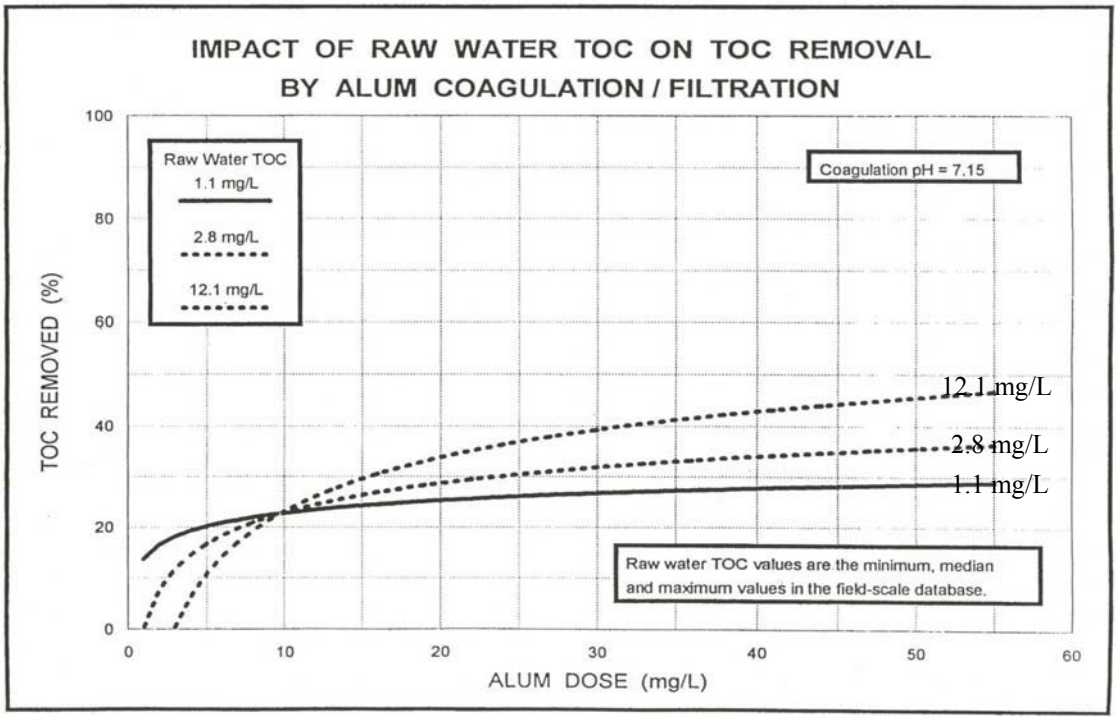
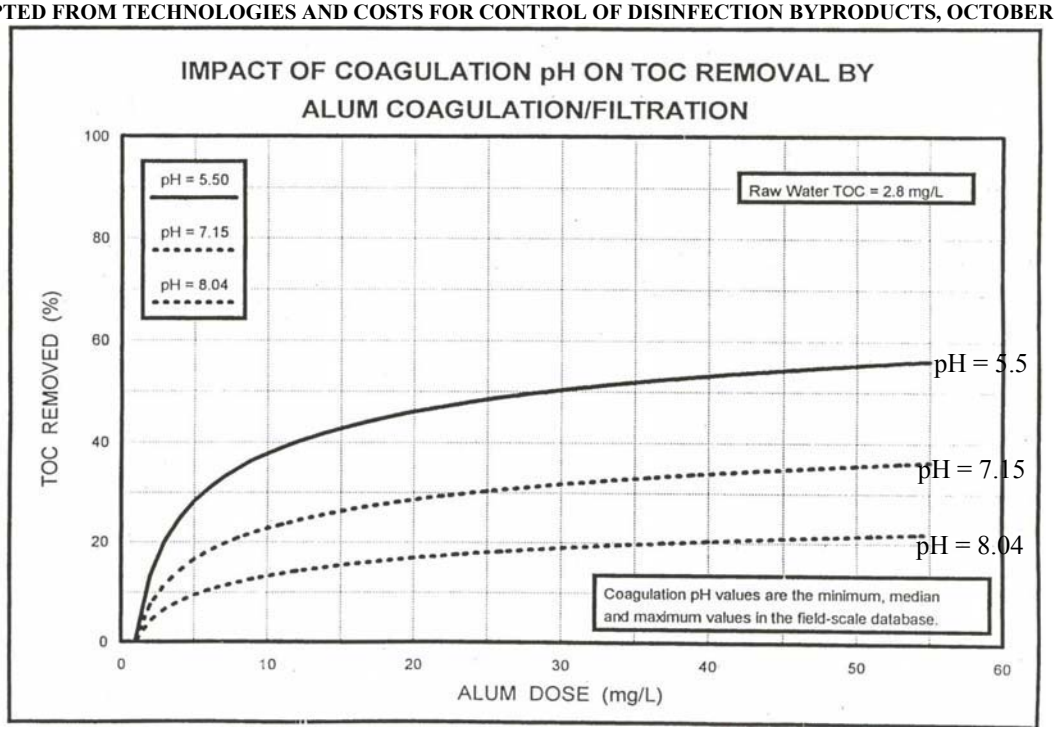


FIGURE 4-2
IMPACT OF COAGULATION pH ON TOC REMOVAL BY ALUM COAGULATION/FILTRATION
(ADOPTED FROM TECHNOLOGIES AND COSTS FOR CONTROL OF DISINFECTION BYPRODUCTS, OCTOBER 1998)



Coagulant Type

Although aluminum and iron salts are equally capable of reducing turbidity, their ability to remove NOM varies. The choice of coagulant will be site specific, dependent on influent water characteristics, chemical costs and handling requirements. Aluminum coagulants have been more studied and the general range of coagulant dose is provided below in Table 4-4 below. Iron coagulants have been less studied and the general range of dosage is also provided in Table 4-4. Iron coagulants tend to benefit from more acidity and coagulation at reduced pH values. Generally, iron coagulants are better suited for source waters with medium to high SUVA values and high alkalinity concentrations.

TABLE 4-4
GENERAL GUIDELINES ON COAGULANT DOSAGES

Estimates of Coagulant Dosages	Alum and PACls	Ferric Coagulants	Organic Cationic Polymers
• These estimates are very good for raw water supplies with SUVA values of 3-4 or higher. • For raw water supplies with SUVA of 2-3, estimated dosages should be reduced. • NOM does not control coagulation for raw water supplies with SUVA of 2 or less.	• pH 6 – 6.5: 0.7 mg as Al/mg DOC • pH 7 – 7.5: 1 mg as Al/mg DOC • Not recommended for pH values of 5.5 or less except for high TOC waters: 0.5 mg as Al/mg DOC • Dosages may be lower for some PACls. • Medium basicity PACls (40-50% aluminum) are well suited for cold waters with low turbidity. • Low basicity PACls (up to 20% aluminum) are well suited for raw waters high in color and TOC.	• pH 5 – 6: 2 mg as Fe/mg DOC • pH 7 – 7.5: 4 mg as Fe/mg DOC	• 0.65 – 1 mg active polymer/mg DOC (Multiply above by 100 divided by percent product to obtain product dosage).

Source: Edzwald, 2001.

The use of polymerized coagulants is relatively new in North America. Poly-aluminum chloride (PACl) has demonstrated better removal compared to aluminum and iron based coagulants (i.e. ferric chloride and alum) for low and moderate concentrations of humic acids and fulvic acids at pH values less than 5.5 and greater than 7.0.

PACl also consumes less alkalinity than traditional iron and aluminum coagulants. The alkalinity, water temperature, pH and SUVA of the raw water source will affect the effectiveness of polymerized coagulants. Note that both polymerized and traditional aluminum and iron based coagulants require jar testing for optimal performance. Advantages of using polymerized coagulants include the following:

- For source waters where the NOM does not dictate the coagulant dose, lower dosages are typically required compared to traditional coagulants.
- Consume less alkalinity and better suited for raw waters with low alkalinity.
- Flocs tend to be tougher and denser.
- Performance is less dependent on temperature as compared to alum.
- Polymerized coagulants are less sensitive to pH and can be used over a pH range of 4.5 to 9.5 (MWH, 2005).

A disadvantage to polymerized coagulants use is their cost. Typically, chemical costs for polymerized coagulants are higher than compared to traditional coagulants such as alum and ferric salts.

Guidelines on the nature of NOM and the expected removals by coagulation are summarized in Table 4-5.

TABLE 4-5
GUIDELINES ON NATURE OF NOM AND EXPECTED REMOVALS BY COAGULATION

SUVA	Composition	Coagulation	DOC Removals
4 or Greater	• Mostly aquatic humics. • High hydrophobicity. • High molecular weight organics.	• NOM controls. • Good DOC removals.	• > 50% for Alum and PACLs. • Slightly greater for ferric chloride.
2 - 4	• Mixture of aquatic humics and other NOM. • Mixture of hydrophobic and hydrophilic NOM. • Mixture of molecular weights.	• NOM influences. • DOC removals are fair to good.	• 25 – 50% for Alum and PACls. • Slightly greater for ferric chloride.
Less than 2	• Mostly non-humics. • Low hydrophobicity. • Low molecular weight organics.	• NOM has slight influence. • Poor DOC removals.	• < 25% for Alum and PACls. • Slightly greater for ferric chloride.
Note: EPA's Enhanced Coagulation Requirements set under Stage 1 DBPR are met for SUVA values < 2 for raw or treated waters.			

Source: Edzwald, 2001.

pH

Optimum pH ranges for the two most commonly used coagulants, alum and ferric chloride, are well known. The optimum pH ranges where conventional treatment and sedimentation removal of NOM are practiced are 5.0 to 6.5 for alum and 5.0 to 6.0 for ferric chloride, respectively.

Alkalinity

Alkalinity affects the conventional treatment reactions in two ways. A minimum amount of alkalinity is required for the reactions to proceed. In low alkalinity cases, a base is added to ensure that the raw water pH value is optimum for NOM removal and for the coagulation reaction to proceed. More significantly, alkalinity buffers the water and affects the reaction pH. High alkalinity can impede NOM removal by maintaining an elevated pH for the reaction. Adding excess alum to overcome excessive alkalinity can result in excess aluminum in the finished water. However, optimizing the reaction pH will minimize excess aluminum. In general, the optimum pH under excessive alkalinity conditions can

be best achieved through addition of an acid. Site specific cases have been recorded where this is not the case and alum addition proved to produce better results. Westerhoff, et al., (2000) investigated DBP precursor removal of a raw water source characterized with a TOC concentration of 3.2 mg/l and alkalinity of 121 mg/l (as $CaCO_3$). The study concluded that due to the high alkalinity concentration in the raw water, only 15 percent DOC removal was achieved using conventional treatment methods. On an operational note, acid addition raises the need for diligent workplace safety as strong acids can severally injure the operator if not handled properly.

Enhanced Coagulation/Flocculation

As mentioned earlier in this report (Section 3), the Stage 1 DBPR Rule requires Enhanced Coagulation/Enhanced Softening as an initial process step to improve removal of DBP precursors. The PWS must remove a certain percentage of TOC (based on raw water TOC and alkalinity concentrations) prior to the point of continuous disinfection. The Rule is based on 90 percent of the facilities complying with the removal requirements. The Rule also requires higher doses of coagulant than needed for removal of particles but does not require pH optimization.

Enhanced coagulation may improve the removal of DBP precursors in a conventional water treatment plant. The removal of TOC (a surrogate measure of NOM) by coagulation has been demonstrated in laboratory demonstrations and full-scale studies (Amy et al.). This removal of TOC can result in a decrease in TTHM and HAA5 formation. Public water systems can obtain a better idea of the removal they can obtain by using jar tests.

Enhanced coagulation can include increasing the coagulant dose, changing the coagulant, changing the pH or adding a polymer. The primary advantages of enhanced coagulation may include an improvement in disinfection effectiveness, a reduction in DBP precursor formation, a reduction in bromate formation, and can enhance arsenic and radionuclide removal (USEPA, 2007).

The primary disadvantages of enhanced coagulation include adverse effects on the filtration process (shorter filter run times, premature breakthrough), corrosion control in the distribution system, increased concentrations of inorganics (manganese, aluminum, sulfates, etc.) in the finished water, adverse impacts to residuals handling operations and higher chemical costs (USEPA, 2007).

4.3.5.2 Process Interactions

Coagulation will reduce the demand for chlorine by reducing NOM, which will improve disinfection efficiency[1]. A study performed by Summers et al., (USEPA 2007) indicated that enhanced coagulation, even with prechlorination, reduces DBP levels up to 20 percent as long as the point of pre-disinfection is no more than three minutes ahead of the rapid mix. The overall effect of pH reduction and reduced oxidant demand should improve disinfection efficiency. Over the typical pH operating range for water treatment plants (5.5 to 9.5), decreasing the pH improves the disinfection characteristics of chlorine and ozone, but decreases the effectiveness of chlorine dioxide.

In addition, at lower pH values, higher coagulant doses required for enhanced coagulation may result in the restabilization of particles and an increase in the settled water turbidity. Raw waters that are coagulated under charge neutralization conditions are particularly susceptible to this problem, because optimal coagulation for TOC and particle removal is different. In one study, Vrijenhoek, et al., (1998) observed that under optimized pH conditions, particle removal may not be adversely affected.

4.3.5.3 DBP Removal

Considerable research has been conducted on the removal of DBP precursors by conventional and enhanced coagulation. However, little has been conducted on the removal of DBPs by conventional and enhanced coagulation once formed. Since DBPs can form prior to the coagulation process by water treatment plants that use prechlorination or other preoxidation processes just upstream of the rapid

[1] Enhanced Coagulation is targeted at removing higher portions of NOM.

mix basins, moving the point of chlorination/oxidation to reduce DBP formation is a more typical treatment technique compared to removing DBPs with conventional or enhanced coagulation processes.

A review of studies based on water treatment plants that moved the point of chlorination from a pre-rapid mix location to a post-sedimentation location and used enhanced coagulation processes demonstrated an approximate 30 percent increase in TTHM removal. These results demonstrate the advantage of removing the DBP precursors rather than removing the DBPs.

4.3.6 Clarification

Clarification processes are not effective for NOM removal without pretreatment processes to form a removable floc.

4.3.6.1 Sedimentation

Sedimentation is a process used to remove easily settleable, heavier solids and is used to settle flocs to reduce the solids load conveyed to the filters. Sedimentation will remove some NOM by settling floc particles to which NOM is adsorbed. The amount of NOM removed is dependent on the amount of NOM that is adsorbed on the floc particles and the settling environment.

High rate sedimentation processes such as high-rate clarifiers using parallel plates or tube settlers, upflow clarifiers, sludge blanket clarifiers, and ballasted sedimentation provide higher overflow rates and more effective surface area for sedimentation compared to conventional sedimentation basins. Although these processes enhance sedimentation (reduced settling time, footprint) these processes do not remove any additional amounts of NOM as compared to conventional sedimentation.

4.3.6.2 Dissolved Air Flotation

Dissolved air flotation (DAF) is an effective alternative to sedimentation or other clarification processes. The process was first used for drinking water treatment in Sweden in 1960 and has been widely used in Scandinavia and the United Kingdom for more than 40 years. The first use in the United States was at New Castle, New York in 1993. The process is especially applicable when treating for algae, high color, cold water temperatures and low turbidity water.

Dissolved air flotation is a clarification process often used to remove low density particulate matter from raw water such as algae, for raw water supplies with DOC (natural color), and cold temperature raw water sources. The DAF process tends to achieve better removal of low-density floc produced from coagulation of TOC than sedimentation and has the ability to remove small microorganisms and turbidity (MWH, 2005).

The amount of NOM removed through DAF is dependent on both the amount of NOM bound in the floc particles and the efficiency with which the air can attach to suspended particles. The amount of NOM bound in the floc will depend on conditions that also affect conventional treatment processes. The successful attachment of air bubbles to floc will be a function of the destabilization of floc. If these destabilized floc particles (low particle charge and fairly hydrophobic) are not produced by the coagulation/flocculation process, then attachment to the air bubbles will most likely be poor (MWH, 2005).

Dosage and pH are critical conditions in preparing floc particles so that air bubbles stick to the floc particle in the contact zone of the flotation tank. Optimal coagulation conditions for flotation are similar to those for settling processes. However, the flocculation requirement is different. For the DAF process, flocculation has to be designed to create a large number of smaller floc particles for effective removal. Floc particles need be only tens of microns in size so that shorter flocculation times can be used. Minimal flocculation times of five minutes are feasible in DAF plants using high rate DAF and filtration loading rates (Edzwald, 1999).

Historically, DAF tanks have been designed with hydraulic loading rates of 4 to 8 gpm/ft^2 (10 to 20 m/h). However, as more research and full-scale DAF systems were installed, hydraulic loading rates have increased to as high as 17 gpm/ft^2 (42 m/h). Pilot scale research by Edzwald (1999) indicated that DAF loading rates as high as 12 to 16 gpm/ft^2 (30 to 40 m/h) can be used in an integrated design with a flocculation time of 5 minutes depending on water temperature (MWH, 2005).

4.3.6.3 Flottazone

Studies have shown that the process of combining ozone and flotation leads to a highly effective drinking water pretreatment stage, where the main objective is to reduce the load of clogging materials, especially algae going to the filters. This process is quite similar to DAF, however uses ozone gas rather than air. A pilot study was undertaken to prove the possibility of generating a very high number of ozonated microbubbles. The results obtained in this study led to the design and construction of a full-scale installation in Agen, France (Boisdon, et al., 1995). However, there have been few full scale installations due to operation complexities and operator safety associated with ozone.

4.3.7 Membrane Separation

Membranes have been used historically for desalination of brackish waters. The process uses hydraulic pressure to force the liquid through a semi-permeable membrane. There are four main classifications of pressure-driven membrane separation processes: reverse osmosis (RO), nanofiltration (NF), ultrafiltration (UF), and microfiltration (MF).

4.3.7.1 DBP Precursor Removal

Membrane separation technology offers potential advantages over conventional treatment because it provides a means for physical removal of NOM and it reduces the reliance upon chemical disinfection; therefore reducing DBPs. Although it will not eliminate the need for chemical disinfection, it may reduce the dosage needed to maintain a final disinfection barrier.

Membrane processes remove DBP precursors through filtration and adsorption of organics. If NOM molecules are larger than the membrane pores, NOM will be rejected and DBP formation potential will be reduced. However, the size of the NOM particles is only one factor that influences NOM rejection. Shape and chemical characteristics of organic compounds also play important roles in the permeation of NOM through a membrane (AwwaRF, 1996). Membranes may also remove NOM through adsorption of organics on the membrane surface. Adsorption depends on the chemical characteristics (particularly charge and hydrophobicity) of both the membrane material and the organic compounds. Unfortunately, organic adsorption is undesirable since it has proven to be a primary cause of irreversible fouling.

RO is a high-pressure separation process, operating within a typical pressure range of 100 to 4,605 psi. RO membranes are designed to concentrate low molecular weight organic materials and salts while allowing water and solvents to pass. It retains virtually all ions and passes water. In desalination applications, the pressure applied exceeds the osmotic pressure of the salt solution against a semi-permeable membrane, thus leaving the salt behind.

NF typically operates in pressure ranges between 50 pounds per square inch (psi) and 150 psi and typically allows passage of particles less than 1 nm (10-9m). It is similar to RO with the exception that monovalent ions and very low molecular weight organics will pass. NF significantly increases potable water quality by decreasing microbiological pathogen counts and concentrations of organic matter, resulting in more efficient post-disinfection and a decrease in THMFP. Because many biodegradable dissolved organic carbons have low apparent molecular weight, only RO and NF retain biodegradable organic matter. Studies showed that 80 to 95 percent rejection of NOM is possible with nanofiltration. A low concentration of organic matter easily assimilable by bacteria (AOM) might pass through NF membranes. NF systems are typically used to remove hardness and DBP precursors. Currently large scale use of NF members is used for brackish water treatment and is gaining momentum in surface water treatment to meet the Stage 2 DBPR regulations and considered a best available technology by USEPA.

Molecular weight cutoffs (MWCO) are a measure of the removal characteristic of a membrane in terms of atomic weight (or mass), as opposed to pore size and vary with the type of filter. A typical range for a NF is from 200 to 500 Daltons. The Dalton is a unit of mass used to express atomic and molecular masses and typically used as a unit of measure for the MWCO of a UF, NF, or RO membrane. It is equivalent to 1/12th the mass of a carbon-12 atom (i.e., one atomic mass unit (amu).

The use of NF for treatment of surface waters must incorporate adequate pretreatment to prevent fouling of the filters. Clarification is required to reduce particle content and size. Biodegradable organic matter removal prevents biological fouling and chemical processes (such as conventional treatment) prevent chemical fouling of the membranes. A study by Mulford, et al., (1999) demonstrated approximately 90 percent reduction of simulated distribution system trihalomethane formation potential and simulated distribution system haloaectic acid formation potential with a NF membrane system treating raw water with an average DOC level of 11.4 mg/L.

UF and MF are two alternative technologies that have attracted considerable attention in recent years. MF membranes have a typical pore size of 0.1 to 2.0 microns and are used to remove particulate material and microorganisms. UF membranes have a typical pore size of 0.01 to 0.1 microns and will concentrate high molecular weight species (MWCO between 1,000 and 500,000 Daltons) while allowing dissolved salts and lower molecular weight materials to pass.

These membrane systems can effectively remove all particulate matter and provide a physical barrier to microorganisms of concern. Due to the physical removal of microorganisms, less disinfection is required; therefore reducing DBP formation.

Without pretreatment, membrane processes remove NOM to varying degrees. Literature reports that MF and UF removals typically range between 5 and 30 percent. Karmi, et al., (2000) reported removals in the 5 to 15 percent range. Typically NF and RO removals are on the order of 50 to 99 percent. Membranes,

particularly those with MWCO in the 100 to 500 range such as NF, appear to be very effective for DBP precursor removal. TOC, THMFP, and total organic halide formation potential (TOXFP) removals of 70 to 95 percent are commonly achieved in systems using such membranes.

Over the past 15 years, much has been written regarding pretreatment for membrane filtration. There is evidence to suggest that pretreatment of the feed water to UF and MF systems may improve the removal of NOM. The two most common types of pretreatment are coagulation and the addition of an adsorbent such as powdered activated carbon (PAC).

A number of studies have demonstrated that pretreatment using coagulation can improve membrane performance. There are also studies that demonstrated an adverse effect on membrane performance and NOM removal. The use of coagulants and the resulting performance of MF and UF systems have been variable. The effect of coagulation on membrane fouling and NOM removal is site specific and due primarily to the feed water characteristics, type and dosage of coagulant, and the type of membrane materials (USEPA, 2007). A further study by Howe, 2001, reported that when the coagulant dose was adequate to remove a large fraction of NOM, membrane performance was improved due to the reduction of fouling. In lieu of the conflicting research conclusions, pilot studies are suggested to access the effect of coagulation on membrane performance.

Numerous studies have concluded the addition of PAC to the feed water upstream of MF and UF systems has been able to increase removal rates of NOM. In full-scale water treatment plants, a recirculating or submerged membrane filtration system provides an ideal reactor for the adsorption of NOM by PAC. These water treatment plants use PAC for the removal of dissolved natural or synthetic compounds and use the membranes to separate the PAC, as well as microorganisms and other particulate matter from the filtered water. The use of PAC as a pretreatment to MF and UF has been well documented with

TOC removals ranging from approximately 20 percent to 85 percent in a number of studies. In addition, the use of PAC to adsorb NOM has been observed to have little to no effect on fouling or membrane performance (flux rates) (AWWA, 1999 and MWH, 2005).

Research has also shown that extensive pretreatment of surface water sources may be required to reduce NF or RO fouling. Pretreatment processes commonly used are MF, UF, conventional treatment or bank filtration (Chellam, et al., 1998). Chowdhury, et al., (2000) reported the use of slow sand filtration, biologically optimized rapid sand filtration, and UF appeared adequate as pretreatment steps. Gullick, et al., (2000) studied pretreatment before NF to treat Mississippi River water characterized by high turbidity and particle levels. The study concluded that attempts to use MF or UF alone as pretreatment before NF resulted in excessive fouling of the MF and UF membranes and that conventional treatment provided effective pretreatment before the NF. Gullick, et al., (2000) also concluded that with pretreatment, the NF system provided 80 to 90 percent removal of NOM.

4.3.7.2 DBP Removal

DBP removal efficiencies documented in the literature demonstrated a general inverse relationship between DBP removal and recovery. A study of THM removals reported recovery rates for two RO systems at 52 percent and 67 percent, respectively. Reported THM removal rates for the 52 percent recovery RO system were between 15 and 31 percent. The reported THM removal rates for the 67 percent recovery RO system were between 67 and 79 percent. NF System performance reported THM removal for a NF system in excess of 80 percent at a recovery of 75 percent.

4.3.8 Activated Carbon Adsorption

Activated carbon, both in granular or powdered form has been used traditionally as an adsorbent in the removal of taste and odor and color. More recently, it has been used successfully for the removal of NOM in water, which are the main DBP precursor materials. Activated carbon has also been shown to be effective in removing DBPs after

formation. Powdered activated carbon (PAC) applied to the water is removed through filtration or membrane separation. Granular activated carbon (GAC) can be used as a filter medium in a filter/adsorber or in a separate contactor/adsorber.

4.3.8.1 DBP Precursor Removal

Granular Activated Carbon (GAC)

A number of factors affect the removal of NOM by GAC. These factors include raw water pH, pretreatment processes, GAC type, and empty bed contact time.

pH

The number of evaluations concerning the impact of pH on GAC adsorption is limited. However, Hopper, et al., (1996) established the relationship between TOC adsorption capacity and pH for 13 different source waters. The study concluded that between pH 5 and 10, a decrease of one pH unit resulted in a six percent increase in adsorption capacity. Pilot-scale treatment alternative studies for the City of San Diego and a bench-scale study at Minneapolis demonstrated the same trend. The Minneapolis study concluded that a decrease in pH from 7 to 6 increased the GAC bed life by 85 to 100 percent while achieving TOC removals between 50 and 70 percent. It is also known that at lower pH values, NOM is less soluble; therefore more absorbable.

Pretreatment Processes

Clogging of the GAC bed may be caused by suspended solids in the raw water, or 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 organics removal to reduce natural organic loading to the GAC bed.

Coagulation processes (as a pretreatment to a GAC contactor) improve performance by reducing the influent TOC levels and decreasing the influent pH. Studies have shown that coagulation using ferric chloride before GAC filtration lengthens the GAC bed life and increases TOC adsorption. This process

increases the total mass of TOC that a given mass of GAC removes. As concluded by Nowack, et al. (1999), as influent TOC concentration decreases GAC bed life increases rather linearly.

Preozonation has been the subject of several studies that concluded the impact of ozonation on adsorption of NOM can be highly variable. These changes are mainly due to the effect ozone has on the biodegradability of the NOM. If NOM is more biodegradable, increased removals by microbiological activity in the GAC contactor can be expected.

GAC Type

A number of studies reported a relationship between GAC pore size distribution and NOM molecular size distribution. These studies reported that removal of NOM molecules of low to moderate molecular weight fractions were favored. Owen, et al., (1998) reported contradictory results. The study concluded that low molecule size NOM broke through while moderate and large molecules continued to be removed. Another study by Faust and Aly (1998), concluded that molecular size of the adsorbate is a key factor with compounds too large to diffuse into the pore structure.

Empty Bed Contact Time (EBCT)

The empty bed contact time (EBCT) is the design factor that determines the bed depth at a given hydraulic loading rate. The EBCT is the time required for water to pass through the empty column. The choice of the EBCT is the primary parameter in determining the effectiveness of GAC adsorption. Studies have demonstrated the time it takes for the effluent to reach a given TOC concentration increases with increasing EBCT. However, the increase in run time is not directly proportional to the EBCT. Several studies (Miller and Hartman, 1982; Sontheimer, et al., 1988; USEPA 2007) concluded an EBCT between 7.5 and 20 minutes. The EBCT was demonstrated to have a definite effect in prolonging the life of the GAC contactor.

It has been demonstrated that a GAC bed, with a reactivation interval of 180 days, can remove 35 percent to 70 percent TOC (running average) with 15 minutes EBCT. The GAC bed also removed 55 percent to 85 percent TOC (running average) with 30 minutes EBCT.

Powdered Activated Carbon (PAC)

Removal of NOM by PAC is affected by the molecular weight distribution, pH, and the presence of competing compounds and oxidants. In addition, the PAC type, dose, location of PAC addition, and contact time are important. In typical water treatment applications, PAC is added at the rapid mix stage along with coagulants for the removal of taste and odor compounds, low concentrations of pesticides and other organic micropollutants.

The average dosages used for these purposes are typically between 5 and 10 mg/L, therefore removal of DBP precursors is generally minimal. In addition, micropollutants and lower molecular weight organics diffuse much faster than larger organic molecules. These smaller organics can diffuse into the smaller PAC pores and compete for adsorption sites. Therefore, pore blockage or competition for adsorption sites may reduce the adsorptive capacity of the PAC (AWWA 1999 and MWH, 2005).

In addition, THM precursors are generally slowly adsorbing compounds. Long PAC contact times are required to adsorb these precursors and for beneficial use of the carbon capacity. Therefore, the contact times typically observed in conventional settling basins (several hours) may not be long enough for effective DBP precursor removal (AWWA, 1999).

However, as previously mentioned, the application of PAC followed by ultrafiltration (PAC/UF) has shown significant potential for the reduction of DBP precursors and NOM removal. A hollow-fiber configuration is commonly used in the PAC/UF system to allow cross-flow filtration and recirculation. The PAC is retained in the recycle loop of the UF membrane long enough to allow the adsorption of organics. The process maximizes the organic loading on the PAC

and minimizes the carbon usage rate. The UF membranes are used for particle and microorganism removal and the PAC can easily be added to the process for NOM removal. As previously mentioned, the use of PAC may reduce membrane fouling by removing NOM. The disadvantages to the PAC/UF process is the high PAC dosage and long filtration times that may be required to effectively remove NOM. This can also lead to increased head-loss through the membrane and buildup of microorganisms (MWH, 2005).

A number of full-scale plants in France use the PAC/UF process with considerable success. The Vigneux-sur-Seine Plant utilizes the PAC/UF process and significant removal of TOC, biodegradable DOC (BDOC), and THMFP were reported. Based on plant data from 1998 (one year after installation), TOC was reduced by 69 percent, BDOC was reduced by 70 percent, and THMFP was reduced by 89 percent.

Jack and Clark (1998) also documented TOC and DOC removals based on PAC/UF studies. TOC removals between 13 percent and 85 percent and DOC removals between 13 percent and 76 percent were reported, respectively.

4.3.8.2 Process Interactions

Reactions with oxidative pretreatment chemicals such as chlorine, ozone, chlorine dioxide, and permanganate with the GAC surface or with NOM may adversely affect adsorption performance. Therefore, the application point for these oxidants should be moved to a point after the GAC contactor. In addition, pretreatment to prevent inorganic fouling of GAC systems may be required. The influent to the GAC contactor that is saturated with calcium carbonate, manganese dioxide, or iron hydroxide solids can lead to coating of the GAC media and blockage of the pores; thus reducing the GAC adsorptive capacity.

4.3.8.3 DBP Removal

GAC can remove minimal amounts of THMs. Studies conducted in the 1980's demonstrated that removal requires a long EBCT. This long EBCT leads to a shorter service life and earlier than expected GAC media replacement. THM removal rates in the 70 percent range were reported.

A case study at the City of Corpus Christi, Texas, (Tokuno, 1997) reported THM removals between 72 percent and 89 percent after GAC filtration. Symons, et al., (1981) investigated the performance at 11 pilot and full-scale GAC facilities. Two conclusions worth noting were that chlorinated THMs were more poorly adsorbed than brominated THMs and that the presence of background organics adversely affected the adsorption of the THMs.

Two studies reported by Malcolm Pirnie presented removals of brominated THMs. Kruithof (1986) reported 65 percent removal of dichlorobromomethane by GAC adsorbers in the Netherlands with an EBCT of 13 minutes. The Provincial Waterworks of North Holland in Andjek reported THM removals by two-stage GAC adsorption with EBCT of 20 to 30 minutes following conventional treatment. Removals of 76 percent and 96 percent were reported for dichlorobromomethane and dibromochloromethane, respectively.

A study by Tung et al. (2006) conducted adsorption isotherm and field tests to investigate the removal of HAAs in the initial stage of GAC filtration. Results from this study indicated that GAC adsorption plays only a minor role in overall HAA removal in GAC filters. In addition, HAAs (except trichloroacetic acid) had a much lower adsorption capacity than did THMs where the adsorption capacity for HAAs increased with increasing halogen number. Chlorinated HAA species had a lower adsorption capacity than did their brominated analogues. The study also suggested that HAA removal in aged GAC filters may be attributable to biodegradation.

Studies have also focused on removal of inorganic DBPs. The results of a study by Thompson and Ashe (1990) indicated that GAC may be a viable means of removing chlorite and chlorine dioxide. However the adsorptive capacity for chlorite was exhausted rapidly and the chlorite oxidized to chlorate on the GAC by free chlorine that was present. The study concluded that since chlorate is not removed, the practicality of GAC for chlorite removal is questionable.

Another study investigated GAC as a technology for chlorite removal. The research focused specifically on the interference between chlorite and organic matter removal on GAC active sites and the applicability of a "chemical regeneration" process to improve GAC removal efficiency for organic matter and chlorite. The results from the study demonstrated that GAC can be an efficient method for chlorite removal (Collivignarelli et al., 2006).

A study by Kirisits and Snoeyiak (1999) demonstrated that GAC may also remove small quantities of bromate by chemically reducing it to bromide. The controlled biological activity in the GAC contactor proved to be effective at removing bromate since the microorganisms donate electrons needed to reduce the bromate to bromide.

4.3.9 Precipitative Softening

Precipitative softening is a commonly used chemical precipitation process with lime and other chemicals to reduce water hardness (carbonate and noncarbonate hardness) and in some cases, to enhance clarification before filtration. The chemicals most commonly used to precipitate calcium and magnesium are lime and caustic soda. In addition, when raw water carbonate hardness is too low, sodium carbonate (soda ash) must be added to the process.

Historically, lime-soda ash softening has not been thought of as an organics removal process. However, since the same mechanisms required for softening also occur during coagulation and flocculation, the process has been considered for NOM removal. The softening of water involves the removal of calcium and magnesium hardness as calcium carbonate and magnesium hydroxide precipitates. When precipitated, magnesium hydroxide takes the form of light hydrous solids having a very high surface area and a positive charge. This high surface area and positive charge make magnesium hydroxide an excellent adsorbent for dissolved precursors and an effective coagulant for particles. However, calcium carbonate may also be capable of substantial NOM removal since it precipitates at a lower pH and requires a reduced lime dosage.

4.3.9.1 DBP Precursor removal

Removal of DBP precursors by precipitative softening depends on a number of factors. These factors include: the nature of the NOM, water quality characteristics including calcium hardness and magnesium hardness, pretreatment processes such as oxidation, and the type and dose of the chemical being used for hardness removal.

NOM Characteristics

Many of the same NOM characteristics that affect NOM removal during coagulation/flocculation affect NOM removal during softening. As previously mentioned, the removal of NOM is improved for higher molecular weight molecules, particularly those with molecular weights greater than 10,000 Daltons. Therefore, precipitative softening typically removes higher molecular weight and more hydrophobic DOC particles. DOC or DBP precursor removal is typically improved by increasing the lime dose (and pH), by the precipitation of magnesium as magnesium hydroxide, and by adding a coagulant. However, lime dosages higher than those required to remove magnesium typically produce modest increases in precursor removal.

In the softening process, calcium carbonated precipitates are negatively charged. Accordingly, acidic NOM molecules are poorly removed while the neutral and basic molecules are effectively removed. Whereas conventional treatment more effectively removes humic acids, lime-soda ash softening more effectively removes fulvic acids. The removal of humic substances during lime soda ash softening is increased with increasing pH.

Water Quality and Chemical Dose

Bench scale and full scale studies have demonstrated that adsorption onto calcium carbonate and magnesium hydroxide is the principle mechanism for NOM removal in lime-soda ash softening. As previously mentioned, it has been demonstrated that magnesium hydroxide adsorbs NOM to a stronger degree than does calcium carbonate due to its high surface area and positive charge.

A more recent study by Russell et al. (2009) evaluated the fundamental chemistry of precipitate characteristics and their effect on NOM removal. The research showed that NOM removal before magnesium hydroxide precipitation occurs from adsorption onto calcium carbonate (and mixed magnesium-calcium carbonate) precipitates as they form and possibly through direct precipitations as a magnesium fulvate or humate. The study also concluded that some water treatment facilities that practice softening may be able to operate at a slightly higher lime dose, avoiding magnesium hydroxide precipitation while achieving slightly greater removal of TOC. In addition, the study also reported that depending on source water characteristics, plants could enhance NOM removal by adding magnesium.

Enhanced Precipitative Softening

Enhanced precipitative softening by increasing the lime dose (and pH), the precipitation of magnesium as magnesium hydroxide, and/or adding a coagulant can significantly remove higher amounts of NOM and reduce DBP precursor formation as compared to conventional softening. The success of enhanced softening for NOM removal may prove beneficial to water treatment plants that already use conventional softening to meet the Stage 1 DBPR requirements for TOC removal.

A study conducted by Roalson et at. (2003), on Lake Austin water (Austin, Texas) reported that enhanced softening (by increasing lime dosages) improved NOM removal by as much as 43 percent (as measured by reductions in DOC). The study also reported that by increasing the lime dosage to the point just below the dose needed to precipitate magnesium hydroxide, may be a good operating point for water treatment plants to improve the removal of NOM without significantly affecting the overall operation of the plant (i.e. increased solids handling, solids carry-over from clarifiers to filters, and reduced filter runs).

In addition, the results of the study also reported that bromine-substituted DBPs can preferentially form more rapidly as compared to chlorine-substituted DBPs for surface waters with high bromide concentrations. The study concluded that because bromine-substituted DBPs have a higher molecular weight than chlorine-

substituted DBPs, surface water sources with high bromide concentrations may produce a higher mass concentration of DBPs. This may affect the water treatment plants ability to meet DBP regulations under the Stage 1 DBPR.

Similar to past research, the study also concluded that magnesium hydroxide was more effective than calcium carbonate at removing NOM, although the difference was not substantial.

4.3.9.2 Process Interactions

The recycling of sludge is a practice with lime-soda ash softening plants in order to increase the efficiency and effectiveness of hardness removal. This practice of promoting rapid formation of large calcium carbonate particles inhibits NOM removal by decreasing the adsorptive capacity. Therefore, the practice of recycling sludge may be detrimental to NOM removal.

Liao and Randtke (1985) demonstrated that optimized NOM removal during the softening process occurred under opposite carbonate conditions than for enhanced hardness removal. Hardness removal is optimum when carbonate is in excess whereas NOM removal was enhanced when calcium was in excess with respect to carbonate. It was suggested during the EPA review of the Stage 1 DBPR Rule, that a two step approach to effectively remove both NOM and hardness be considered. The first step would optimize NOM removal by adding excess lime to elevate pH and calcium levels and then optimize hardness removal by adding carbonate alkalinity and recycling sludge. It was further suggested that soda ash addition in a single stage process be delayed for several minutes after lime addition.

Enhanced softening will reduce the demand for chlorine by reducing NOM, which in turn will improve disinfection efficiency. But the combined effects of increased pH and reduced oxidant demand may result in a decrease in disinfection effectiveness. The pH can be lowered by recarbonation or acid addition; however, this should be balanced with the softening and final pH requirements of the process. As stated previously, for a typical water treatment plant pH operating range (5.5 to 9.5), decreasing the pH may improve the

disinfection characteristics of chlorine and ozone. However, decreasing the pH may negatively affect the effectiveness of chlorine dioxide. In addition, at a pH below 7.5, lower chloramine concentrations are anticipated. The reverse is true for increasing pH values.

When enhanced softening is practiced prior to ozonation, the pH of the water should be lowered prior to the ozonation. However, this will increase the carbon dioxide dose required during recarbonation.

4.3.9.3 DBP Removal

Lime-soda ash softening has not been considered a common process for DBP removal. Facilities that soften with lime-soda ash normally use potassium permanganate as a pre-oxidant.

4.3.10 Ion Exchange (MIEX)

Ion exchange processes are widely used in water treatment to remove ionic contaminants. In municipal applications, this is typically done using the MIEX process. The processes involve the exchange of ions from one phase to another. The exchange of ions occurs between the solid phase of the exchanger and influent water. Since ion exchange processes work only with ions, substances that do not ionize in water are not removed by the process.

Color, TOC, and THM precursors are all surrogate measures of NOM. As previously noted, NOM is the result of natural decay of vegetable matter, primarily leaves, grasses, roots, and wood. The resulting soluble or colloidal dispersions of semi soluble matter are a mixture of fulvic and humic acids, tannins, and lignins. These are primarily aromatic hydrocarbon polymers and carboxylic groups that behave as anions, which can be removed by anionic ion exchange processes.

In general, 80 percent of NOM measured as TOC have molecular weights under 10,000 Daltons. This is larger than inorganic ions but still small enough to enter the gel phase of ion exchange resins. Rook and Evans (1979) found that the use of a weak base anion exchange resin resulted in 50 to 60 percent removals of THM precursors. Brattebo, et al., (1987) have also documented removal of up to 80 percent DOC in highly colored surface water.

With these resins, however, carboxylic acids occurring in NOM are weakly ionized and are kinetically slower and bulkier than inorganic ions. The ion exchange and regenerant delivery systems must be designed for lower flow rates and larger regenerant contact times. Disposal of the regenerate may also be a problem.

The magnetic ion exchange process MIEX® is a process developed in Australia that reportedly overcomes the practical limitations with NOM removal resins. The MIEX® process utilizes 180 µm beads in a suspension contactor, allowing for greater diffusion without the headloss associated with packed columns. The resin is then recovered by sedimentation assisted by a magnetic component in the resin beads. Testing of the process occurred in a lime softening plant at St. Paul, Minnesota and at a conventional treatment plant at Perth, Australia. The process demonstrated an increase in the TOC removal at the softening plant by 39 percent. The results at the conventional treatment plant demonstrated greater than 75 percent DOC removal could be achieved when the MIEX® process was applied in combination with the conventional treatment process while utilizing 70 percent less coagulant.

A study conducted by Hsu and Singer, (2009) reported that operational data from a Florida utility that had implemented a full-scale MIEX® process upstream of its lime-softening facilities demonstrated improved calcium removal after implementation of this treatment. However, the effects of NOM in the lime-softening process were not well understood. The study also reported that when model water and real waters were pretreated with anion exchange resin, DOC concentrations were reduced and the result was improved calcium removal by lime-softening.

4.3.11 Summary

Table 4-6 summarizes the effectiveness of the treatment technologies reviewed. Based on these treatment technologies, conventional treatment, membrane filtration, activated carbon adsorption and ion exchange present the best physical and chemical treatment processes for DBP reduction. Table 4-7 summarizes the effectiveness of treatment technologies for removal and control of DBPs.

TABLE 4-6
TREATMENT TECHNOLOGIES SUMMARY FOR DBP REDUCTION

Treatment Technology	General Effectiveness
Preoxidation	
Prechlorination	• Creates chlorinated organic DBPs.
Preozonation	• Creates brominated organic DBPs and bromate. • Creates biodegradable organics which may increase DBP formation in the distribution system. Therefore, should be followed by biologically active filtration.
Advanced Oxidation Processes	• Slight DBP formation reduction at pH greater than 8.5. • Further research required.
Chlorine Dioxide	• Can create inorganic DBPs.
Potassium Permanganate	• Slight THMFP reduction. • No DBPs created.
Hydrogen Peroxide	• Effective at oxidizing sulfides and iron. • No significant DBP formation reduction when used alone. • Increased organic removal and DBP formation reduction when used with ozone or UV light.
Conventional Filtration	• Little DBP precursor or DBP reduction without coagulation pretreatment. • Provides some protozoan removal.
Biologically Active Filtration	• DBP precursor removal between 15% and 25%. • Cold water temperature may affect performance. • Large filtration units may be required. • Effective for removal of biodegradable organics created by ozonation. • Biologically activated carbon and ozone can reduce THMFP by 40 to 70%.
Conventional Treatment	• DBP precursor removal between 30 and 50%. • Commonly used in water treatment industry. • Process well understood. • Process can be optimized for DBP precursor removal. • Provides protozoan removal.
Clarification	• Not effective without coagulation. • DAF preferred over sedimentation for NOM removal.
Flottazone	• Additional research required.
Membrane Filtration	• Pretreatment required for most surface waters. • Nanofiltration most appropriate. • 80 to 90% DBP precursor removal. • Improved removals with PAC. • Provides good microorganism removal.
Activated Carbon Adsorption	• Pretreatment should be provided for GAC. • 35 to 70% TOC removal. • 70 to 90% THM removal. • 65% BDCM removal.
Lime-Soda Ash Softening	• DBP precursor removal of 30 to 50%. • Two stage process may improve TOC removal. • Best results at pH 10.8 or greater • Enhanced softening effective for NOM removal.
Ion Exchange (MIEX)	• 50 to 60% THM precursor removal possible.

4.4 DISINFECTION PROCESSES AND INJECTION LOCATIONS

4.4.1 Introduction

Free chlorine is the most widely used primary disinfectant in the drinking water treatment industry in North America. Free chlorine may exist as either hypochlorous acid or hypochlorite ion, based on pH. The effectiveness of free chlorine in bulk water is dependent upon pH. In addition, numerous other chlorinated species such as organic chloramine, dichloramine and/or trichloramine are typically present during typical free chlorination practices. Chlorine is very cost effective and provides the water utility with a reliable disinfection method. However, when NOM is present in the source water supply, the use of chlorine results in the formation of halogenated DBPs.

The type and amount of DBPs produced during water treatment depends largely on the disinfectant type, source water quality (TOC, DOC, bromide concentrations, etc.), treatment sequences, contact time, and environmental factors such as temperature and pH. In general, THM formation rate increases with time, temperature and pH. Also, an increase in the concentration of humic and fulvic acids may cause a proportional increase in THM formation.

Modifications to a treatment train utilizing chlorine can be effective in reducing DBP formation. Modifications may include moving the point of chlorination, lowering the pH, enhanced coagulation or softening, or switching disinfectants. For water utilities that move the point of chlorination, consideration should be given to changes that may be required in order to achieve total contact time (CT), such as enlarging the size of the chlorine contact basins.

Because of the possible long-term health effects associated with DBPs, and to comply with current drinking water regulations such as the Stage 1 and Stage 2 DBPR, LT2ESWTR, LCR, and other rules, PWSs are looking to alternative disinfection technologies to provide clean, safe drinking water to their consumers.

For surface water treatment systems, treatment plant performance should be optimized for DBP precursor removal and filtered water turbidity. Disinfectants should be applied for sufficient time and at sufficient concentrations for inactivation of microbial pathogens. In addition, all water systems should actively manage their distribution systems to meet water demand and provide consistently good water quality.

This section discusses the characteristics of several alternative disinfectants to free chlorine, the associated DBP formation potential, process interactions, and alternative application points. A brief summary of the disinfectants along with possible treatment strategies to combine disinfectants to provide a "multiple barrier" approach to pathogen removal is also discussed. In addition, the effectiveness of several physical chemical processes to remove pathogens is discussed along with how treatment and disinfection strategies will affect the water quality in the distribution system.

4.4.2 Ozone

Ozone is used in water treatment for disinfection and oxidation. Ozone was first used for drinking water disinfection and oxidation in Europe. Early application of ozone in the United States was primarily for non-disinfection purposes such as color removal or taste and odor control. However, since the implementation of the Stage 2 DBPR and the LT2ESWTR, ozone usage for primary disinfection has increased in the United States.

For example, some of the largest water utilities in North America to utilize ozone, the Los Angeles Aqueduct Filtration Plant and the Massachusetts Water Resources Authority (MWRA), treat up to 600 MGD and 405 MGD of drinking water, respectively. In 2002, approximately 330 water treatment plants were using ozone in the United States.

4.4.2.1 Disinfecting Characteristics

Ozone is a powerful chemical disinfectant and oxidizer. It is an unstable gas that is generated on-site, using either air or liquid oxygen through a corona electric discharge. Ozone is very effective at disinfecting many microbes and as a pre-oxidant. However, ozone can react with bromide in the raw water source and form bromate; a DBP regulated by the Stage 1 DBPR. Ozone can also oxidize organic matter into smaller molecules, which can provide a more easily degradable food source for microorganisms in the distribution system. Because

of its instability in water, ozone cannot be used to provide a disinfectant residual in the distribution system. Also, in some instances ozonation of organic matter will form aldehydes, ketones, and other compounds that can impart taste and odors to the water.

Ozone is a powerful disinfectant but has a very short half-life (<4 minutes). Therefore, ozone cannot be used as a secondary disinfectant. A secondary disinfectant such as free chlorine or chloramine is typically used. The use of ozone generally requires higher doses to inactivate *Cryptosporidium* than those necessary for *Giardia* and viruses (USEPA, 2007).

Bacterial Inactivation

Ozone is a highly effective disinfectant because of its high oxidation potential. Various studies on bacterial inactivation have demonstrated that ozone is very effective in inactivating bacteria. A 4-log reduction of *E. Coli* in less than 1 minute with an ozone residual of 9 mg/L at a temperature of 12°C was reported by Wuhrmann and Meyrath as early as 1955. Domingue, et al., (1988) reported greater than 2-log reduction of *Legionella pneumophila* within 5 minutes contact time at an ozone concentration of 0.21 mg/L (USEPA 1999).

Virus Inactivation

Ozone is most effective against viruses. As with most chemical disinfectants, the degree of microbial inactivation is temperature dependent. Inactivation is greater at higher temperatures. Table 4-7 illustrates the required CT values for 4-log inactivation of viruses by ozone.

TABLE 4-7
CT VALUES FOR 4.0-LOG INACTIVATION OF VIRUSES BY OZONE

pH	≤ 1°C	5°C	10°C	15°C	20°C	25°C
6-9	1.8	1.2	1.0	0.6	0.5	0.3

Source: MWH, (2005), USEPA (2003). CT values reported in min x mg/L.

Protozoa Inactivation

Protozoan cysts are more resistant than bacteria or viruses to inactivation by ozone and other disinfectants. *Giardia lamblia* requires approximately six times the inactivation time as compared to *E. coli*. One study by Zafer, et al., (2000) reported on a pilot-scale investigation at the City of Austin. The study found that *Cryptosporidium* oocysts were approximately 10 times more resistant to ozone than *Giardia lamblia*. The study also found that with raw water at ambient pH or softened water at pH 9.5, *Cryptosporidium* inactivation CT targets could not be met (ozone doses less that 10 mg/L).

Table 4-8 summarizes the required CT values for 3-log inactivation of *Giardia*. Table 4-9 illustrates the required CT values for inactivation of *Cryptosporidium* by ozone at various log inactivations and water temperatures.

TABLE 4-8
CT VALUES FOR 3.0-LOG INACTIVATION OF *GIARDIA* CYSTS BY OZONE

pH	≤ 1°C	5°C	10°C	15°C	20°C	25°C
6-9	2.9	1.9	1.4	0.95	0.72	0.48

Source: MWH, (2005), USEPA (2003). CT values reported in min x mg/L.

TABLE 4-9
CT VALUES FOR INACTIVATION OF *CRYPTOSPORIDIUM* BY OZONE

Temperature	Log Inactivation					
	0.5	1.0	1.5	2.0	2.5	3.0
≤ 0.5°C	12	24	36	48	60	72
1°C	12	23	35	46	58	69
5°C	7.9	16	24	32	40	47
10°C	4.9	9.9	15	20	25	30
15°C	3.1	6.2	9.3	12	16	19
20°C	2.0	3.9	5.9	7.8	9.8	12
25°C	1.2	2.5	3.7	4.9	6.2	7.4

Source: MWH, (2005), USEPA (2003). CT values reported in min x mg/L.
pH values between 6 and 9.

4.4.2.2 DBP Formation

Oxidation of NOM with ozone does not involve halogen substitution and therefore does not form halogenated DBPs (TTHMs or HAA5). However, if the bromide ion is present in the raw water source, halogenated DBPs may be formed. These brominated DBPs appear to pose a greater health risk than non-brominated DBPs.

Ozonation of NOM does form a variety of organic and inorganic byproducts as discussed previously in Section 4. Some more common disinfection byproducts are: acetoniterilles, aldo-acids, keto-acids, carboxylic acids, peroxides, and cyanogen bromides. Specific byproducts which forms are depending the source water quality. Aldehydes and low-molecular weight organic acid groups have been shown to be compounds formed by ozonation of NOM and easily biodegradable. Reported total aldehyde concentration in drinking water disinfected with ozone ranged from less than 5 to 300 μg/L, depending on the TOC concentration and the applied ozone to organic carbon ratio (USEPA 1999).

In the presence of bromide, the major inorganic DBP of concern is bromate. The EPA established MCL for bromate is 10 μg/L; although this value may be revised to a higher value based on new research sponsored by the International Ozone Association. The reactions with bromide are variable depending on the characteristics of the raw water source (i.e. bromide concentration). Four principal reactions with bromide may occur; therefore producing ozone byproducts. These four reactions are summarized below.

1. Ozone reacting with NOM can form aldehydes and oxidized organics.
2. Ozone reacting with bromide will form hypobromite ions which can further react with ozone and hydroxyl radicals to form bromate.
3. Ozone reacting with bromide can also form hypobromous acid, which can further react with ammonia in the water to form bromomine.
4. Hypobromous acid may also react with NOM and form brominated organic byproducts such as bromoform.

Ozonation of source water containing bromide ion can produce brominated byproducts, the brominated analogues of the chlorinated DBPs. Song et al. (1997) found that bromate ion formation is an important consideration for waters containing more than 0.10 mg/L bromide ion.

4.4.2.3 Process Interactions

Oxidation of NOM with ozone produces biodegradable compounds that may cause re-growth problems in the distribution system if not removed. Escobar, et al., (2000) reported a 127 percent increase of assimilable organic carbon (AOC) and a 49 percent increase of biological dissolved organic carbon as compared to before ozonation. As discussed previously in section 4, increases in the biodegradable fraction of organic carbon may occur as a result of moderate to high levels of ozonation. These ozone doses are typical of the doses commonly applied for disinfection. Note that biologically active filters will remove AOC and biodegradable DBPs.

In addition, ozone can increase the ultraviolet light transmittance (UVT) for water treatment plants that utilize UV disinfection by degrading NOM, reducing soluble material, and precipitating metals. However, ozone is also a strong absorber of UV light, and will decrease the UVT if an ozone residual is present in significant concentrations in the water passing through a UV reactor. Therefore, the ozone residual must be quenched from the feed water to the UV reactors. Quenching agents that do not absorb UV light (such as sodium bisulfite) can be used to destroy the ozone residual upstream of the UV reactors.

4.4.2.4 Application Point

Ozone is normally applied as a preoxidation process. Moving the point of ozonation after coagulation, flocculation, and sedimentation reduces the formation of ozonated DBPs since NOM is removed as part of the coagulation/flocculation process. As previously mentioned, biodegradable organic matter that forms may be removed by biological active filters.

The only practical ozone feed point for high quality water with direct filtration is the raw water. For water characterized by low ozone demand and high turbidity, (which indicates the presence of inorganic material such as clay or silt particles), ozone should be added after either pre-sedimentation or conventional treatment. For raw water with low turbidity and high ozone demand, (which indicates the presence of dissolved constituents like bromide ion, iron, manganese, color, or organics), ozone can be added to either the raw water or after sedimentation. If the water contains organic constituents that become more biodegradable by ozonation, a biological treatment step is required. For raw water with high turbidity and high ozone demand, (which indicates presence of high concentrations of organic material and inorganic particles), the most effective use of ozone is after sedimentation and possibly after filtration. Ozonation after filtration requires a biological treatment step to remove the biodegradable organics to prevent the formation of DBPs by secondary chlorination.

Ozonation requires capital investment since ozone must be produced on-site that requires a high level of maintenance and substantial operator training.

4.4.3 Ultraviolet Light

Ultraviolet light disinfection is relatively new to the drinking water industry. Interest in using ultraviolet (UV) light to disinfect drinking water has grown among PWSs due to its ability to inactivate pathogenic microorganisms without forming regulated DBPs. UV light has proven effective against some pathogens, such as *Cryptosporidium*, that are resistant to commonly used disinfectants like chlorine.

As of the year 2000, there were more than 400 UV disinfection facilities worldwide. These UV facilities typically treat flows of 1 million gallons per day (mgd) or less (USEPA, 2006). Since 2000, several large UV installations across the United States have been constructed or are currently under design. A 180 mgd facility is in operation in Seattle, Washington, and a 2,200-mgd facility is under design for the New York City Department of Environmental Protection. The number of PWSs using UV disinfection is expected to increase significantly over the next decade due in part to meeting regulations under the LT2ESWTR for *Cryptosporidium* (USEPA, 2006).

UV light is in the region of the electromagnetic spectrum that lies between X-rays and visible light. The UV spectrum is divided into four regions: vacuum UV [100 to 200 nanometers (nm)]; UV-C (200 to 280 nm); UV-B (280 to 315 nm); and UV-A (315 to 400 nm). UV disinfection primarily occurs due to the germicidal action of UV-B and UV-C light on microorganisms (200 to 300 nm) with the most effective wavelength being 260 nm.

Unlike other disinfectants such as chlorine, ozone or chloramine, UV disinfection is not a chemical disinfectant. Electromagnetic radiation emitted by the UV reactors with wavelengths ranging from 240 to 280 nm effectively inactivates microorganisms by damaging their Deoxyribonucleic Acid (DNA). This affects the microbe's ability to replicate and infect the host organism.

4.4.3.1 Disinfection Characteristics

The use of UV light disinfection is an EPA approved technology for primary disinfection of potable water. UV can effectively inactivate pathogens such as *Giardia* and *Cryptosporidium* without the need for chemicals, which could otherwise contribute to the formation of DBPs. *Giardia* and *Cryptosporidium* are chlorine-resistant pathogens. Therefore UV disinfection may be an effective treatment alternative for water systems that require additional *Cryptosporidium* treatment under the LT2ESWTR while meeting requirements for DBPs under Stage 1 and 2 DBPR.

Ultraviolet light disinfects drinking water by altering the genetic material (DNA) in the cells of microorganisms inhibiting further transcription of the cell's genetic code and preventing the microorganisms to reproduce. A microorganism that can not replicate can not infect a host. UV light is produced by germicidal lamps submerged in the filtered water. As water flows past the UV lamps, microorganisms are exposed to the required UV dose for inactivation. The UV calculated dose is the product of the intensity of the UV light and the time of exposure. UV contact time is measured in seconds. The UV dose is expressed in units of milliwatt-seconds per centimeter squared (mWs/cm^2) or more typically, millijoules per centimeter squared (mJ/cm^2).

A typical UV disinfection system is comprised of an enclosed chamber with an inner sleeve of high purity quartz, which contains one or more UV lamps. Common UV lamp types include low pressure, low pressure-high output, and medium pressure. Low pressure and low pressure-high output lamps operate at a lower temperatures; therefore consume less power. However, more lamps are required to deliver the appropriate UV dose. Medium pressure lamps operate at a much higher temperature and consume more power. Less medium pressure lamps are needed to deliver the required UV dose; therefore take up a smaller footprint. UV reactors are typically installed in filtered water pipe runs, for a completely closed system. Common UV lamps contain mercury. Proper design, construction, and operation will prevent lamp breakages and subsequent release of mercury to the water.

The effectiveness of UV disinfection (measured by a dose-microbial response curve) is not temperature or pH dependent. However, because UV disinfection relies on UV light interacting with the microorganism's DNA to be effective, any substance that either absorbs or refracts the germicidal UV light can interfere with disinfection. A common measure of the portion of germicidal light (i.e., light specifically with a wavelength of 254 nanometers) transmitted through a material is UV Transmittance (UVT). The higher the UVT, the better UV light can be transmitted through the water and the more effective the treatment. Compounds in source waters that can absorb or refract UV light and reduce UVT include humic and fulvic acids, phenols, metals (e.g., iron and manganese), and anions (e.g., nitrates). In addition to absorbing UV light and decreasing UVT, compounds in the water can foul the external surfaces of the UV lamp sleeves. The rate of fouling depends on water quality characteristics such as hardness, alkalinity, ion concentration, and pH.

Bacteria, Protozoa and Virus Inactivation

Among the pathogens of interest in drinking water, viruses are most resistant to UV disinfection followed by bacteria, *Cryptosporidium* oocysts, and *Giardia* cysts. The doses to inactivate bacteria by UV disinfection are small compared to

viruses. The International Ultraviolet Association (IUVA) reported approximate UV doses required for 3-log inactivation of *E. coli* and *Salmonella* between 5-10 mJ/cm^2 and 5-20 mJ/cm^2, respectively. The current UV dose required for 3-log inactivation of Adenovirus is between 75-150 mJ/cm^2.

Under the LT2ESTWR, the EPA developed UV dose requirements for PWSs in order to receive credit for inactivation of *Cryptosporidium*, *Giardia*, and viruses. The UV dose values are applicable to post-filter applications of UV disinfection in filtered and unfiltered water systems. These dose requirements are illustrated in Table 4-10.

TABLE 4-10
UV DOSE REQUIREMENTS UNDER THE LT2ESWTR (MJ/CM^2)

Target Pathogen	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	12	15	22
Giardia	1.5	2.1	3.0	5.2	7.7	11	15	22
Virus (Adenovirus)	39	58	79	100	121	143	163	186

Source: Adapted from the USEPA UVDGM, November 2006

Prior to installation and operation, UV reactors are required by the EPA to undergo experimental testing known as validation. This validation process is used to determine the operating conditions under which the UV reactors deliver the dose required for the inactivation credits illustrated in Table 4-10. At the conclusion of the validation process, the validated dose is compared to the dose requirements in Table 4-10 for compliance and proper inactivation credit.

As observed in Table 4-10, a disadvantage of UV disinfection for the inactivation of viruses is the large dose required for 3 to 4-log inactivation. However, recent studies and research is focused on demonstrating inactivation of viruses, specifically adenovirus since this virus is used as the benchmark for inactivation credit. A recent study by Linden et al. (2009) demonstrated 4-log inactivation of adenovirus by a medium pressure (MP) UV reactor at much lower dosages than required by the EPA. The study developed a protocol for field-scale testing and

validation of adenovirus disinfection by MP UV light and used the protocol to perform a live adenovirus challenge test showing the level of adenovirus inactivation possible in a field-scale UV disinfection system. The study demonstrated that MP UV irradiation technology can achieve greater than 4-log reduction at economically viable UV doses lower than 100 mJ/cm^2.

Many microorganisms have enzyme systems that repair damage caused by UV light. Repair mechanisms are classified as either photorepair or dark repair. Even though microbial repair can occur, neither photorepair nor dark repair is anticipated to affect the performance of drinking water UV disinfection. Photorepair of UV irradiated bacteria can be prevented by keeping the UV disinfected water in the dark for at least two hours before exposure to room light or sunlight. Treated water typically remains in the dark in the piping, reservoirs, and distribution system after UV disinfection. Most facilities also use chemical disinfection to provide further inactivation of bacteria and virus and protection of the distribution system. Both of these common practices make photorepair unlikely to be an issue for PWSs. Dark repair is also not a concern for PWSs because the required UV doses shown in Table 4-9 are derived from data that are assumed to account for dark repair. A research study by Linden et al. (2002) did not observe dark repair of *Giardia* at UV doses typical for UV disinfection applications (16 and 40 mJ/cm^2) (USEPA, 2006).

4.4.3.2 DBP Formation

In drinking water, research on potential byproducts of UV disinfection has focused on the effect of UV light on the formation of halogenated DBPs after subsequent chlorination, the transformation of organic material to more degradable components, and on the potential formation of other DBPs (e.g., biodegradable compounds, nitrite, and other byproducts).

Research has shown that UV light doses less than 400 mJ/cm^2 do not significantly affect the formation of THMs or HAAs upon subsequent chlorination (USEPA, 2006).

Several studies have shown low-level formation of non-regulated DBPs (e.g. aldehydes) as a result of applying UV light at doses greater than 400 mJ/cm^2 to wastewater and raw drinking water sources. However, at the doses typical for UV disinfection in drinking water (< 140 mJ/cm^2), no significant change was observed. In addition, UV disinfection has not been found to significantly increase the assimilable organic carbon (AOC) of drinking water at UV doses ranging from 18 – 250 mJ/cm^2 (USEPA, 2006).

Since UV disinfection does not produce any regulated DBPs, it is an attractive alternative to chlorine disinfection. Select surface water systems and systems using GWUDI of surface water may meet Stage 2 DBPR requirements by switching to UV disinfection and lowering their primary disinfection chemical dosage. However, as previously noted, secondary chemical disinfectant is required to maintain a residual in the distribution system. This secondary disinfection dose, which is relatively fixed due to chlorine demand in the distribution system and requirements to maintain a residual to meet the TCR, may be high enough to interact with the finished water TOC levels to form high levels of chloraminated DBPs.

4.4.3.3 Process Interactions

Unit processes and chemical addition upstream of UV reactors can significantly affect UV reactor performance because they can change the particle content and UVT of the water. Additionally, when UV disinfection is used in combination with another disinfectant, synergistic disinfection potentially may occur (i.e., the combination of disinfectants may be more effective than either disinfectant acting alone). Water treatment processes upstream of the UV reactors can be operated to maximize UVT (thus reducing the potential for lamp sleeve fouling), thereby optimizing the design and costs of the UV reactor. Pre-oxidation and enhanced coagulation are potential treatments that can be used for this purpose (USEPA, 2006).

Adding oxidants, such as ozone and chlorine, can increase UVT by degrading NOM, reducing soluble material, and precipitating metals. Ozone, if used for taste and odor control, will typically be added before the filters. However, if an

ozone residual is present in the water before it enters the UV reactor, the ozone must be quenched. Quenching agents that do not absorb UV light (such as sodium bisulfite) can be used to destroy the ozone residual upstream of the UV reactors. The ozone residual should not be quenched with thiosulfate, as thiosulfate also absorbs UV.

In addition, if chlorine dioxide is being used, it should be added after the UV reactor. If corrosion inhibitors that contain UV-absorbing compounds are used, they should be added after water has passed through the UV reactor (USEPA, 2007).

When UV disinfection is applied to water with a free or total chlorine residual, some reduction of the residual may occur. The reduction in free chlorine residual is proportional to the delivered dose and the independent of flow rate (USEPA, 2007). The reduction in chlorine residual further depends on the chlorine species, UV light source, and water quality characteristics. A free chlorine residual loss of approximately 0.3 mg/L was observed in a WTP at a UV dose between 80 and 120 mJ/cm^2. A total chlorine residual loss of approximately 0.2 mg/L was observed in bench-scale testing at UV doses up to 40 mJ/cm^2 (USEPA, 2007).

Other chemicals used in water treatment such as ferric iron and permanganate also absorb UV light and can decrease UVT. However, conventional treatment processes (coagulation/flocculation, filtration, etc.) should reduce or remove these chemicals below concentrations that may affect the UVT prior to reaching the UV reactors.

4.4.3.4 Application Points

The three most common installation locations for UV disinfection are downstream of the combined filter effluent (upstream of the clearwell), on the individual filter effluent piping (upstream of the clearwell), and downstream of the clearwell (either upstream or downstream of the high-service pumps, if present). Each one of these application points has advantages and disadvantages.

The most common point of application is downstream of the combined filter effluent and upstream of the clearwell. This application point typically provides the water treatment plant with the most flexibility for design and operation, and prevents surge and pressure fluctuations unless membrane filtration, pressure filters, or booster pumps are used (USEPA, 2006).

4.4.4 Chlorine Dioxide

Chlorine dioxide is highly soluble in water and remains in solution as a dissolved gas. Chlorine dioxide is a powerful oxidant and disinfectant. Generally, it is a more effective disinfectant than chlorine, but less effective compared to ozone. Chlorine dioxide can also reduce the formation of TTHMs and HAAs by oxidizing precursors prior to chlorine disinfection, as discussed previously in Section 4. Hence historically it has been used as a pre-oxidant in place of chlorine.

Traditionally, chlorine dioxide has been generated on site by reaction between a chlorite source, a chlorine source, and sometimes an acid. Although effective in producing chlorine dioxide, the process requires the transportation, storage and mixing of these hazardous chemicals. These safety issues have been one of the reasons for the sporadic use of chlorine dioxide.

Recently however, patented processes to generate chlorine dioxide on-site using sodium chlorite salt, water, and electricity have been developed as shown below.

$$NaC10_2 + H_20 \xrightarrow{\text{Electricity}} 2C10_2 + 2NaOH + H_2$$

These processes eliminate the need for storage and handling of more hazardous chemicals like hydrochloric acid and gaseous chlorine. One process developed by PureLine, uses sodium chloride, water and electrochemical generators (as opposed to traditional chemical generators) to provide 99.5 percent pure, chlorine dioxide solution with no chlorine.

Chlorine dioxide can be explosive under pressure and historically has not been allowed by the Federal Department of Transportation to be stored or shipped. This is another reason the sporadic use of chlorine dioxide. More recent generations of concentrated ready-to-use chlorine dioxide aqueous solutions are now available which eliminates the need for on-site production and the corresponding equipment. The aqueous solution requires no mixing or activation, which simplifies the application of chlorine dioxide for small PWSs. The solutions are dosed in a similar fashion as bleach. The concentrated solution is typically delivered in a 250-300 gallon tote for ease of storage.

As previously mentioned earlier in this section, chlorite and chlorate can form as a result of chlorine dioxide generation or the decay of chlorine dioxide after it is applied to the water, (i.e. oxidation of NOM or iron and manganese). Researchers have estimated that 50 to 70 percent (by mass) of chlorine dioxide applied during drinking water treatment is converted to chlorite (MWH 2005). This means that if the oxidant demand of the raw water is greater than approximately 1.4 mg/L, chlorine dioxide may not be used as a disinfectant because the chlorite/chlorate ions might exceed the maximum level allowed, unless chlorite removal technologies are implemented downstream (USEPA, 1999).

4.4.4.1 Disinfection Characteristics

Chlorine dioxide is a strong oxidant and can effectively inactivate a wide range of microbes. Generally, chloride dioxide is a more effective disinfectant than chlorine but is less effective than ozone. Chlorine dioxide is slightly less effective than chlorine against viruses and bacteria, but is more effective against *Giardia* and *Cryptosporidium* (USEPA, 2007). Chlorine dioxide can achieve some inactivation of *Cryptosporidium* oocysts. Required CT levels for *Cryptosporidium* inactivation are relatively high, so achieving more than a half log inactivation is unlikely given restrictions on dose. Chlorine dioxide can, however, be a relatively low cost alternative for systems that require a 0.5 log *Cryptosporidium* inactivation to comply with the LT2ESWTR.

The effects of pH, temperature and suspended matter or the aggregation of pathogens on the disinfection efficiency of chlorine dioxide have been studied.

As compared to chlorine, studies have shown that pH has a reduced effect on pathogen inactivation for viruses and cysts with chlorine dioxide than with chlorine in the pH range between 6 and 8.5. Studies on chlorine dioxide have shown the degree of inactivation of poliovirus 1 and *Naegleria gruberi* cysts increase as the pH increases. A recent study on *Cryptosporidium* found that inactivation of oocysts using chlorine dioxide occurred more rapidly at a pH of 8.0 than 6.0. Another study found that chlorine dioxide efficacy increases for *Giardia* inactivation at higher pH levels (USEPA, 1999).

Temperature changes do have an adverse effect on microbial inactivation with chlorine dioxide. Studies have shown the CT product for *Naegleria gruberi* cysts at 5°C are approximately twice that at 20°C. LeChevallier, et al., (1997) found that reducing the temperature from 20 to 10°C reduced *Cryptosporidium* inactivation by 40 percent. Therefore, water treatment systems that regularly experience near freezing temperatures should probably investigate other disinfection techniques (USEPA, 2007).

Suspended matter and pathogen aggregation affect the disinfection efficiency of chlorine dioxide. Laboratory studies of poliovirus 1 preparations containing mostly viral aggregates took approximately 3 times longer to inactivate with chlorine dioxide than single state viruses. Chen et al. (1984) also found that clumps of *Naegleria gruberi* cysts were more resistant to chlorine dioxide than unclumped cysts or clumps of smaller size (USEPA, 1999).

Bacteria Inactivation

As mentioned, chlorine dioxide is slightly less effective than chlorine against bacteria. A study on chlorine and chlorine dioxide inactivation of total coliform and the dose-time relationship was conducted by Roberts et al., (1980). The study was performed using secondary effluents from three different wastewater treatment plants. One of the objectives was to determine the relationships between dosages and contact times and bactericidal efficiency. Chlorine dioxide and chlorine dosages were compared for 2, 5, and 10 mg/L. The contact times selected were 5, 15 and 30 minutes. The results of the study demonstrated that

chlorine dioxide had a more rapid coliform inactivation than chlorine at the shortest contact time of 5 minutes and at the higher concentrations (10 mg/L). However, after 30 minutes of contact time, chlorine dioxide was equal or slightly less efficient than chlorine (USEPA, 1999).

Virus Inactivation

Chlorine dioxide is an effective disinfectant of viruses. However, as mentioned chlorine dioxide is slightly less effective than chlorine and significantly less effective than ozone against viruses. Table 4-11 summarizes the CT requirements for 4-log inactivation of viruses by chlorine dioxide, chlorine and ozone.

TABLE 4-11
CT VALUES FOR 4.0-LOG INACTIVATION OF VIRUSES BY CHLORINE DIOXIDE

Inactivation by	Temperature °C					
	≤1	5	10	15	20	25
Chlorine Dioxide	50.1	33.4	25.1	16.7	12.5	8.4
Chlorine	12	8	6	4	3	2
Ozone	1.8	1.2	1.0	0.6	0.5	0.3

Source: MWH (2005), USEPA (2003).
CT values reported in min x mg/L. pH values between 6 and 9.

Protozoa Inactivation

As previously mentioned, the disinfection efficiency of chlorine dioxide has been shown to be equal to or greater than chlorine for *Giardia* inactivation. Table 4-12 illustrates the CT requirements for 3-log inactivation of *Giardia* cysts by chlorine dioxide, chlorine and ozone.

TABLE 4-12
CT VALUES FOR 3.0-LOG INACTIVATION OF *GIARDIA* CYSTS BY CHLORINE DIOXIDE

Inactivation by	Temperature °C					
	≤1	5	10	15	20	25
Chlorine Dioxide	63	26	23	19	15	11
Chlorine*	N/A	149	112	75	56	37
Ozone	2.9	1.90	1.43	0.95	0.72	0.48

Source: MWH (2005), USEPA (2003).
CT values reported in min x mg/L. pH values between 6 and 9 for chlorine dioxide and ozone.
*Based on a free chlorine dose of 1.0 mg/L at pH 7.0.

4.4.4.2 Disinfection Byproduct Formation

Chlorine dioxide does not form significant amounts of TTHMs or HAA5s. Historically, chlorine dioxide generators do produce some chlorine as a byproduct, so some TTHM and HAA5 may be formed (depending on the system efficiency) but generally much less than as would be dosed by chlorine alone. Chlorine dioxide provides a good alternative to chlorine for systems that wish to lower the formation of TTHM or HAA5 (USEPA, 2007).

One of the biggest disadvantages of using chlorine dioxide as a disinfectant is that it forms chlorite as a byproduct of generation, and by oxidation of NOM or iron and manganese. High oxidant demand and high pH lead to higher chlorite production. The MCL for chlorite was set at 1.0 mg/L by the Stage 1 DBPR. As previously mentioned, as much as 70 percent of the chlorine dioxide dose added to water can decay and form chlorite. This limits the dose of chlorine dioxide that can be used for disinfection and the amount of inactivation achieved. This especially limits *Cryptosporidium* inactivation, since the required CT values for Cryptosporidium are much higher than for other microbes (USEPA, 2007).

4.4.4.3 Process Interactions

Once formed, chlorate is stable in finished drinking water. No known treatment exists for removing chlorate once it is formed. Chlorite can be removed by adding reduced-sulfur compounds such as sulfur dioxide and sodium sulfite, utilizing either GAC or PAC, or adding reduced iron salts, such as ferrous chloride and ferrous sulfate.

The use of sulfur dioxide and other sulfur-based reducing agents forms chlorate when reacting with chlorite. For this reason sulfur-based processes are not recommended (USEPA, 1999).

Also, as mentioned previously if chlorine dioxide is being used as a residual disinfectant, it should be added after the UV reactor. Chlorine dioxide is sensitive to UV light and will degrade to form chlorate when exposed to UV light. This will reduce chlorine dioxide residuals and therefore lower inactivation (USEPA, 2007).

4.4.4.4 Application Points

When used as an oxidant or primary disinfectant, chlorine dioxide is normally added in the raw water prior to filtration. However, due to its volatile nature, it can be easily removed by turbulent aeration in rapid mix tanks and no residual generally is expected in the filtered water.

As a secondary disinfectant, chlorine dioxide may be added after UV reactors and before the clearwell. Chlorine dioxide also is one of three oxidants which carry a sufficient residual and approved by US EPA as a disinfectant for the distribution system. Chlorine and chloramine are the other two approved chemicals. Due to its short half life, it is likely to be more effective for small distribution systems.

It is likely that a utility would be required to test the system full scale. This would require that the liquid chlorine dioxide be purchased or rented and tested over several months. It is likely to take some time for the system demand to be satisfied such that a chlorine dioxide residual can be detected. It is also possible that for systems with unlined cast iron pipe could have a demand which is too large to satisfy. Testing is recommended to be performed in the colder months to determine feasibility followed by a summer test.

4.4.5 Chloramine

Chloramine is formed by the addition of chlorine and ammonia to water. Most utilities use a feed ratio of 3:1 to 5:1 chlorine to ammonia as N by weight. Chloramination has typically been used where it is difficult to maintain a chlorine residual, such as larger water distribution systems with consecutive systems characterized by long residence times. In addition, more water treatment utilities are considering switching to chloramine to minimize the formation of chlorine DBPs (THMs and HAA5s).

Chloramine reacts significantly slower than free chlorine and as a result, they form fewer DBPs and provide a more persistent residual in the distribution system compared to chlorine. Studies have shown that chloramine compounds can penetrate and disinfect biofilms more effectively than free chlorine as well. Biofilms are defined as the

biological layer located on the interior surface of supply pipes, where they proliferate, they can shelter potential microorganisms of concern. Monochloramine (NH_2Cl) is generally considered the preferred species for disinfection purposes because of its biocidal properties, relative stability, and infrequent taste and odor problems (USEPA, 2007). Monochloramine is a weaker disinfectant than free chlorine; therefore it is more frequently used as a residual disinfectant in the distribution system. However, like all treatment processes, if the chlorine to ammonia ratio is not properly controlled, the use of chloramine can lead to nitrification episodes in the distribution system and may cause taste and odor issues (increased concentrations of dichloramines ($NHCl_2$)) and trichloramines (NCl_3) loss of disinfectant residual, and other issues such as toxicity to dialysis patients similarities are done with chlorine) and to fish. Nitrification is the conversion of free ammonia to nitrates and is caused in part by excess ammonia used during the chloramination process. The excess ammonia leads to the growth of nitrifying bacteria in the distribution system which can lower the pH of the water and consequently hinder lead and copper control. Nitrification may also lead to a loss of chlorine residual which allows an increase in HPC bacteria.

4.4.5.1 Disinfection Characteristics

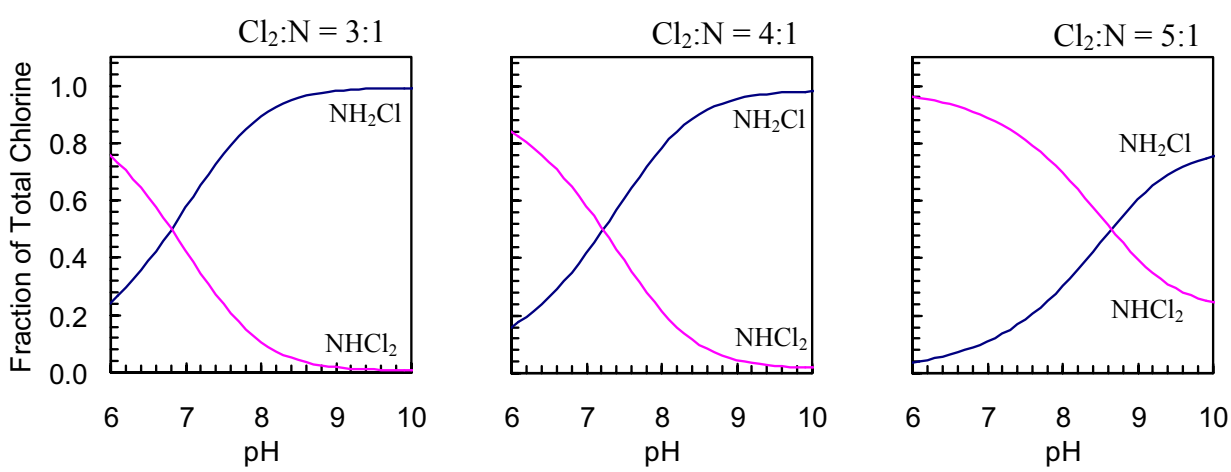
Chloramine is comparatively weak disinfectants for virus and protozoa inactivation. As a result, it is very difficult to meet the SWTR CT criteria for primary disinfection of *Giardia* and viruses using chloramine due to the very long detention times. However, chloramine provide a more stable residual than chlorine. Therefore, the primary use of chloramine is for secondary disinfection protection against microbial growth in distribution system and reduced production of chlorinated DBPs.

As previously mentioned, monochloramine is the preferred species of the three species of chloramines. Monochloramine can be formed by first adding ammonia to the water and then chlorine or vice versa. Ammonia may be added first where formation of objectionable taste and odor compounds caused by the reaction of chlorine and organic matter is a concern. However, the contact time will be much longer to achieve similar disinfection as chlorine. In the specific case of chloramine as a distribution (secondary residual), chloramine

performed better than chlorine. Bacteria imbedded within the distribution system biofilim are contacted by monochloramine for a long period of time, as the monochloramine residual penetrates the biofilm. Conversely, a free chlorine residual does not exist at the pipe wall interface due to faster reactions with source constituents like iron, and is not continuously in contact with biofilm bacteria.

Several studies have been performed to determine the effect of pH, chlorine to nitrogen ratios, temperature, and organic and inorganic compounds on the disinfection effectiveness of chloramine. At typical operating conditions, monochloramine is the predominant chloramine species formed. However, the dominant species formed is a function of the chlorine to nitrogen mass ratio and the pH. Typically as the pH values decrease below 7 and the chlorine to ammonia-nitrogen ratio increases (from 3:1 to 7:1), the production of dichloramine is favored (AWWA, 1999). Therefore, chlorine to nitrogen ratio between 3:1 and 5:1 should be maintained with a pH between 7 and 8.5. These chlorine to nitrogen ratios and the chemistry of chloramine is illustrated in Figure 4-3 for a batch system containing water, ammonia, and chlorine. Note that actual values for real systems will vary depending on system characteristics.

FIGURE 4-3
CHEMISTRY OF CHLORAMINES
(Courtesy of Dr. Greg Harrington University of Wisconsin-Madison)



Note: NH_2C1 = monochloramines
$NHC1_2$ = dichloramines

Similar to most of the disinfectants discussed in this report, the bactericidal and viral inactivation efficiency of chloramine increase with increasing temperature. Furthermore, the efficiency dramatically decreases under conditions of high pH and low temperature (USEPA, 1999).

In addition to ammonia, free chlorine reacts with organic nitrogen compounds to form a variety of organic chloramine. There are over 600 known DBPs of free chlorine, and the number of nitrogen based DBPs is much smaller. The health effects of these other byproducts are largely unknown. These organic chloramines are undesirable byproducts because they exhibit little or no microbiocidal activity. Several other reactions may occur which divert chlorine from the formation of chloramine. These reactions can include oxidation of iron, manganese, and other inorganics such as hydrogen sulfide (USEPA, 1999).

Bacteria Inactivation

Studies on monochloramine as a disinfectant conclude that for a 2.0-log reduction of *E. coli,* monochloramine requires approximately 100 times the CT as compared to free chlorine. (WHO, unknown date; Ward, et.al., 1984).

Virus Inactivation

Chloramine is relatively ineffective against viruses. Experiments show that contact times between 2 and 8 hours are required at concentrations between 0.67 and 1.0 mg/L to achieve greater that 2-log reduction of viruses. Table 4-13 summarizes the CT requirements for 4-log inactivation of viruses.

TABLE 4-13
CT VALUES FOR 4.0-LOG INACTIVATION OF VIRUSES BY CHLORAMINE

pH	≤ 1°C	5°C	10°C	15°C	20°C	25°C
6-9	2883	1988	1491	994	746	497

Source: MWH (2005), USEPA (2003). CT values reported in min x mg/L.

Protozoa Inactivation

Chloramine is a poor agent for protozoa inactivation. Table 4-14 summarizes the CT requirements for 4-log inactivation of *Giardia*. Considering the guideline values for a total chloramine dose of 3.0 mg/L and a CT value of 2,200 at 5°C, a contact time of 12 hours would be required.

TABLE 4-14
CT VALUES FOR 3.0-LOG INACTIVATION OF
***GIARDIA* CYSTS BY CHLORAMINE**

pH	≤ 1°C	5°C	10°C	15°C	20°C	25°C
6-9	3800	2200	1850	1500	1100	750

Source: MWH (2005), USEPA (2003). CT values reported in min x mg/L.

4.4.5.2 Process Interaction

Biological nitrification is the oxidation of ammonia to nitrite and then eventually to nitrate by bacteria and other organisms and is a disadvantage of chloramine. Nitrification adversely impacts the effectiveness of chloramine by increasing the chloramine demand, depleting chloramine residuals and thus allowing bacterial regrowth. A loss of disinfectant residual in the distribution system can result in a violation of the SWTR, and may lead to increased vulnerability to contamination. Nitrification resulting from the use of chloramine can lower the alkalinity and the pH of the water in the distribution system (USEPA, 2007). This can lead to an increase in corrosion and metal release which will have negative effects on lead and copper control.

Ozone use prior to chloramination increases the assimilable organic carbon (AOC) concentration and could destabilize the chloramine residual. This may lead to problems with chloramine residual concentrations at the ends of the distribution system (USEPA, 2007).

Taste and odor problems have been reported from utilities using chloramination, while other water treatment systems have not experienced any problems. Chlorine-based disinfectants typically have some associated taste and odor impacts. Taste and odor problems can also arise from nitrification episodes due

to excess ammonia. Monochloramine has fewer taste and odor problems and variations in residual concentrations produce less noticeable odors than free chlorine. However, dichloramine and trichloramine have much stronger odors than either monochloramine or free chlorine. The production of monochloramine under favorable conditions (pH 7 to 8.5, chlorine to nitrogen ratio of 3:1 to 5:1) will minimize taste and odor concerns. Note the formation of monochloramine is fastest at a pH of 8.3. The odor thresholds for chloramine and chlorine are summarized in Table 4-15.

TABLE 4-15
ODOR THRESHOLDS FOR CHLORAMINE AND CHLORINE

Substance	Formula	Odor Threshold (mg/L as Cl_2)
Monochloramine	NH_2Cl	0.65
Dichloramine	$NHCl_2$	0.15
Trichloramine	NCl_3	0.02
Chlorine*	Cl_2	0.31

Source: Krasner and Barrett, 1985. * Source: USEPA, 2000.

Prior to considering chloramine as a disinfectant, the bromide levels in the water should be reviewed. Higher bromide concentrations may increase the rate of monochloramine decay and chlorinated DBP formation. Bromide levels of 0.1 mg/L are not considered high while levels of 0.5 mg/L or above would likely generate DBPs above the guidelines.

4.4.5.3 DBP Formation

Studies have demonstrated that DBP production by chloramination involves the same compounds (THMs, HAAs, haloacetonitriles (HANs) and cyanogen chloride), but at lower levels than those produced by free chlorine. Factors that affect the DBP formation potential using chloramine include: levels of TOC, bromide ion concentration, pH, chlorine to ammonia ratio, ratio of dichloramine to monochloramine formed, chloramine dosage, order of addition of chlorine and ammonia, and intensity of mixing.

If chlorine is added first, then ammonia added downstream, the THM generation potential increases. If added together, the THM potential is minimized. Although not as sensitive to mixing, THM generation is sensitive to pH and the chlorine to nitrogen ratio. DBP formation is increased with lower pH water. As the ratio of chlorine to ammonia is increased, the DBP formation potential increases. Chlorination breakpoint is near 7.6:1 chlorine to ammonia ratio. If the ratio is 3:1 or lower, this may result in free ammonia in the distribution system and growth of nitrifying bacteria (USEPA, 1999). As an example, nitrification has been observed in tanks with chlorine to free ammonia ratios of 0.3 to 1.4 mg Cl_2/mg N; whereas no nitrification was observed for ratios between 2 and 5 mg Cl_2/mg N.

Alkalinity also has an effect on DBP formation. As alkalinity increases in the source water, the rate of DBP formation increases with monochloramine reacting with the halogen substance organics (USEPA, 1999).

The formation of nitrogenous DBPs (N-DBPs) such as NDMA, have been linked to the use of chlorine and chloramine for the disinfection of drinking water.

Under normal conditions of water chlorination, monochloramine is the principal end product and disinfectant. Chloramine has been approved for use by the EPA. However, data gaps remain about the occurrence of nitrogenous DBPs (N-DBPs) such as N-Nitrosodimethylamine (NDMA). NDMA belongs to a class of compounds known as nitrosamines. The EPA identifies NDMA and other nitrosamines as “probable human carcinogens”. Cancer risks have been calculated for nitrosamines in drinking water by the State of California Department of Public Health (CDPH).

These risks are summarized in Table 4-16. Note the risk levels are expressed in nanograms per liter (ng/L). For example, drinking water that contains NDMA concentrations equal to or greater than 3 ng/L may increase the risk of acquiring cancer by a 1 in 1 million for lifetime exposure to NDMA in drinking water. The

typical risk level used to establish MCLs of lifetime excess cancer cases to be between 1 in 10,000 and 1 in 1,000,000 at the regulated limit. This risk exposure method takes into account several other exposure routes in addition to NDMA in water.

TABLE 4-16
CANCER RISKS FOR NITROSAMINES BY THE STATE OF CALIFORNIA DEPARTMENT OF PUBLIC HEALTH

Nitrosamine	10^{-6} Risk Level (ng/L)	Notification Level[1] (ng/L)	Response Level[2] (ng/L)
N-Nitrosodiethylamine (NDEA)	1	10	100
N-Nitrosodimethylamine (NDMA)	3	10	200
N-Nitrosodi-n-propylamine (NDPA)	5	10	500
N-Nitrosodi-n-butylamine (NDBA)	3	-	-
N-Nitrosomethylethylamine (NMEA)	1.5	-	-
N-Nitrosopiperidine (NPIP)	3.5	-	-
N-Nitrosopyrrolidine (NYPR)	15	-	-

1. Notification levels for NDEA, NDMA, and NDPA are established at 10 ng/L. These limits are above the *de minimis* level to take into account the very low detection limits and their potential presence in association with drinking water treatment.
2. Response levels are levels at which the CDPH recommends removing the source from service. Theycorrespond to a 10^{-4} risk, 100 times the *de minimis* (10^{-6}) value.

The United States Department of Health and Human Services (DHH) also published cancer risk factors for nitrosamines in 2005. Table 4-17 summarizes these risks. Note that risks for NDEA, NDMA and NPIP are more stringent than those set by the CDPH.

TABLE 4-17
CANCER RISK LEVELS FOR NITROSAMINES SET BY THE U.S. DHH

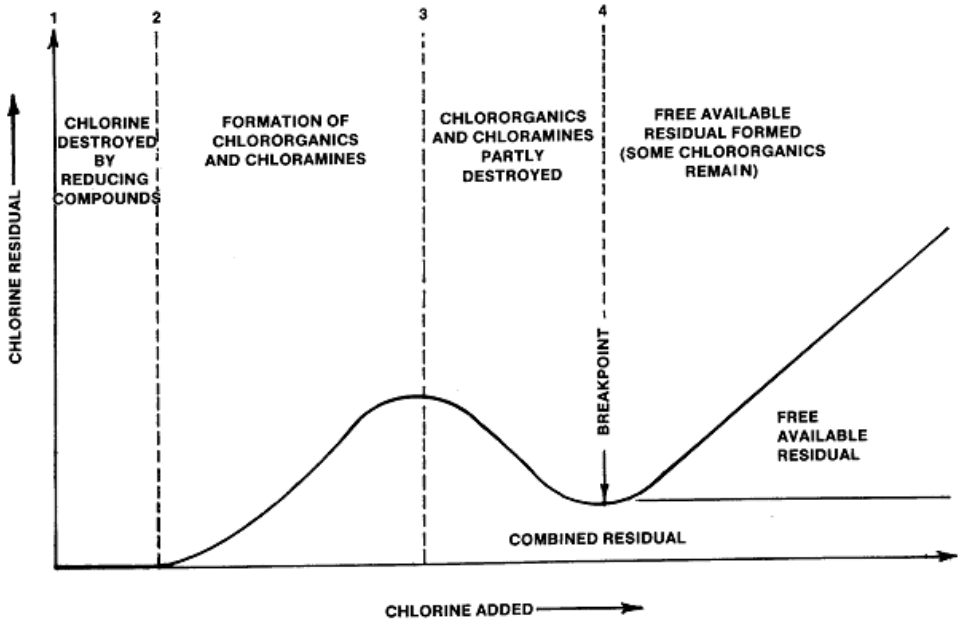
Nitrosamine	10^{-6} Risk Level (ng/L)
N-Nitrosodiethylamine (NDEA)	0.2
N-Nitrosodimethylamine (NDMA)	0.7
N-Nitrosodi-n-propylamine (NDPA)	5
N-Nitrosodi-n-butylamine (NDBA)	6
N-Nitrosomethylethylamine (NMEA)	2
N-Nitrosopiperidine (NPIP)	0.8
N-Nitrosopyrrolidine (NYPR)	16

Source: AwwaRF, 2008.

Recent research has focused on how these N-DBPs are formed, their precursors, and how the use of chlorine and chloramine for the disinfection of drinking water may affect their formation. Based on this research, results have shown that chloramination or chlorination, long contact times for disinfection, the presence of cationic polymers, and surface waters impacted by wastewater effluent discharge may enhance the formation of N-DBPs, especially NDMA. Therefore, much of the research has focused on the formation of NDMA.

Much of the discussion about chloramine has focused on NDMA as it tends to be detected more frequently compared to the other N-DBPs. NDMA can be a byproduct of chloramination or chlorination, but drinking water is not a major source of exposure to NDMA. AECOM's recent analysis of data from US EPA's Unregulated Contaminant Monitoring Regulation 2 (UMCR2) supports this finding. The analysis indicates that NDMA was detected in 1.4% of 71,351 samples (detection limit. = 0.002 ng/L) and was the most frequently detected of the nitrosamines sampled comprising of 96.5% of the nitrosamines detections. NDMA was detected both in chloraminated treated waters and chlorinated treated waters with a slight edge toward chloraminating systems (61% of the detections). Surface water sources tended to have higher detections than groundwater sources. The biggest exogenous sources of human exposure to NDMA are tobacco smoke, chewing tobacco, bacon and other cured meats, beer, fish, cheese, toiletries, shampoos, cleansers, interior air of cars, legumes and household pesticides.

FIGURE 4-4
CHLORINE BREAKPOINT CURVE



The chemistry of nitrosamine formation is complex and many reactions can occur simultaneously. The N-DBPs formed are a result of reaction conditions including concentrations of reactants (i.e. monochloramine, free chlorine, and N-DBP precursors), catalysts and inhibitors, and competing reactions. Based on current research findings, NDMA and other nitrosamines are thought to form through three major pathways. Two of these pathways are significant, since these pathways involve reactions between monochloramine and N-DBP precursors such as organic nitrogen containing compounds (i.e. dimethylamine, dissolved organic nitrogen, etc.). Many of these organic nitrogen containing compounds can be found in wastewater effluent, surface waters, and may be found in raw waters used for drinking water. Research also concluded that water utilities that use surface water under the influence of wastewater effluent are at a higher risk for nitrosamine formation during chloramination compared to those utilities that use non-impacted raw water sources (AwwaRF, 2008). This higher risk is primarily due to increased levels of dissolved organic nitrogen and other nitrogen containing compounds in the wastewater effluent.

Based on the results from laboratory scale experiments and full-scale tests at several water utilities, several options exist for minimizing the formation of NDMA and other nitrosamines. The principal strategies for minimizing nitrosamine formation during disinfection of drinking waters include the following:

1. Utilize raw water sources not significantly impacted by industrial or municipal wastewater effluent discharge.
2. Optimize operational parameters of the disinfection process (i.e. decrease chloramine dose, lower pH).
3. Avoid the use of polymeric materials with nitrogen functional groups during the flocculation process.
4. Removal of nitrosamine precursors.
5. Select polymers with low NDMA precursor material as verified by quality assurance testing.

If no other raw water sources exist besides those impacted by industrial or municipal wastewater effluent, water utilities may consider removal of nitrosamine precursors. Precursor removal strategies are further discussed below.

Optimization of the chloramination process may not be possible as operational changes may affect the microbiological safety of the disinfected water as well as create corrosion issues in the distribution system. In addition, laboratory testing concluded the rate of NDMA formation from the reaction of dichloramine with organic nitrogen containing precursors is significantly faster than reactions with monochloramine. Minimizing the concentration of dichloramine in the production of chloramine may be an effective strategy to reduce NDMA formation (i.e. by addition of free chlorine before ammonia) (AwwaRF, 2008).

However, as mentioned above, THM generation potential increases by adding free chlorine before ammonia. Therefore, water utilities must consider the effects of any operational changes to the chloramination process prior to implementing those changes.

Research has also shown that the use of polyelectrolytes used in water treatment as primary coagulants or flocculant aids may be sources of NDMA precursors. Specifically, the use of polyDADMAC and epi-DMA in the coagulation/flocculation/sedimentation process has been shown to cause significant NDMA formation during chlorination and chloramination. Therefore, the use of these polymer materials should be optimized to limit the polymer breakthrough and subsequent inaction with chlorine or chloramine as well as choosing polymers that limit NDMA precursor material (AwwaRF, 2008).

Options exist for the removal of nitrosamine precursors and removal of nitrosamines formed during chloramination and chlorination. However, the options for nitrosamine removal are limited as NDMA and other N-DBPs are not easily removed by conventional treatment such as coagulation/flocculation/-sedimentation, ozonation or GAC filtration. Therefore, efforts should be focused on the removal of nitrosamine precursors, similar to the removal of other DBP precursors (i.e. NOM) (AwwaRF, 2008).

Some of the options for the removal of nitrosamine precursors include the following:

1. Microbial degradation: Slow sand filtration, artificial groundwater recharge, or riverbank filtration.
2. Pre-oxidation: Bench-scale and full-scale tests at several water utilities demonstrated that pre-chlorination or pre-ozonation are efficient treatment processes for removal of nitrosamine precursors. Observed reduction rates for NDMA formation after pre-ozonation and pre-chlorination were greater than 90 percent. Chlorine dioxide may also be used for pre-oxidation (AwwaRF, 2008)

3. Adsorption: Adsorption onto activated carbon may be a promising alternative for partial removal of N-DBP precursors formed during chloramination. However, results from bench-scale experiments could not be confirmed by full-scale testing. Therefore, further research is required to assess the efficiency of full-scale GAC filtration for precursor removal (AwwaRF, 2008).

   In comparison, NDMA formation as a result of chloramination or chlorination is produced at significantly lower concentrations than THM and HAA concentrations produced by free chlorine. However, NDMA is currently estimated to have a significantly higher cancer risk than the THMs and HAAs.

4.4.5.4 Application Point

Chloramine is rarely used as a primary disinfectant because of the relatively weak disinfecting properties for inactivation of viruses and protozoa pathogens. Therefore, the main use of chloramine is as a secondary disinfectant to maintain residuals in the distribution system.

4.4.6 Peroxone (Ozone/Hydrogen Peroxide)

Peroxone is an advanced oxidation process where hydrogen peroxide is added to ozonated water to increase production of hydroxyl radicals. Peroxone is primarily used for taste and odor control. The process provides comparable disinfection to ozone alone. However, interest in Peroxone for drinking water applications has diminished due in part to costs, operational complexities, and issues related to bromate formation (Malley, 2006).

Disinfection Technology Summary

A summary of disinfection characteristics of the five most commonly used disinfection technologies in water treatment is illustrated in Table 4-18.

TABLE 4-18
SUMMARY OF DISINFECTION TECHNOLOGIES

Parameter	Chlorine	Chloramine	Chlorine Dioxide	Ozone	UV
Bacteria inactivation	Good	Fair	Good	Very good	Good
Virus inactivation	Very Good	Fair	Good	Very good	Fair
Protozoa inactivation	Fair	Poor	Good	Very good	Very good
DBP formation	High	Low	High	Moderate	Low
Residual stability	Moderate	Long	Moderate	None	None
Contact time required	Moderate	Long	Moderate	Short	Short
pH dependant	Yes	Yes	Yes	No	No
Reliability	Excellent	Excellent	Very good	Good	Good
Complexity of equipment	Simple	Moderate	Moderate	Complex	Moderate
Process Controls	Well devel.	Well devel.	Developing	Developing	Developing
Safety Concerns	High (gas)	High	High	Moderate	Minimal
Typical size of plant	All	All	Small to Medium	Medium to Large	All
Relative Cost	Low	Moderate	Moderate	High	Moderate

Source: AWWA (1999)

4.4.7 Combinations of Disinfectants

The above sections on treatment technologies and disinfectants discussed the impacts on DBPs, microbial inactivation, and DBP control strategies. The information gathered on the DBP issues has prompted the use of multiple disinfectants as a means of controlling DBPs. The practice of using multiple disinfectants to produce a synergistic effect by either simultaneous or sequential application to achieve more effective pathogen inactivation is referred to as interactive disinfection by the EPA.

The use of primary disinfectants and secondary disinfectants in water treatment allows each to be optimized independently. Primary disinfection refers to the inactivation of microorganisms to meet the bacteriological reduction requirements. Secondary disinfection refers to the application of a disinfectant to meet requirements for maintaining the microbiological quality within the distribution system.

4.4.7.1 Primary Disinfectants

An effective primary disinfectant inactivates the target organism. Table 4-19 lists the potential primary disinfectants for the four groups of target organisms with or without filtration. Interactive disinfection is acceptable in all cases; however any interactive disinfection that uses ozone without biologically active filtration is strongly discouraged.

TABLE 4-19
POTENTIAL PRIMARY DISINFECTANTS

Target Organism	Potential Primary Disinfectants	
	With Filtration[1]	Without Filtration
Coliform Bacteria	Chlorine Chloramine Chlorine dioxide Ozone UV Interactive disinfection	Chlorine Chlorine dioxide Interactive disinfection[3]
Giardia cysts	Chlorine[2] Chlorine dioxide[2] Ozone[2] UV Interactive disinfection	Chlorine[2] Chlorine dioxide[2] Interactive disinfection[3] UV
Viruses	Chlorine[2] Chlorine dioxide[2] Ozone[2] UV[2] Interactive disinfection	Chlorine[2] Chlorine dioxide[2] UV[2] Interactive disinfection[3]
Cryptosporidium oocysts	Chlorine dioxide Ozone Interactive disinfection UV	Chlorine dioxide Interactive disinfection[3] UV

Source: Adopted from Alternative Disinfectants and Oxidants Manual, USEPA 1999
[1]Natural or treatment filtration reduces disinfection inactivation requirements.
[2]Inactivation credit established in SWTR and its amendments.
[3]Any interactive disinfection that uses ozone without filtration is strongly discouraged.

4.4.7.2 Secondary Disinfectants

The choice of a secondary disinfection is limited to those disinfectants that remain stable in the distribution system and have been approved by the EPA. In order of decreasing stability, the secondary disinfectants are chloramine, chlorine, and chlorine dioxide. Other disinfectants such as ozone and UV do not produce a residual.

4.4.7.3 Disinfectant Combinations

Various combinations of primary and secondary disinfectants are possible. Table 4-21 lists the combinations of disinfectants and their typical applications in water treatment. These combinations consider DBP formation and comments are highlighted on the degree of microbiological inactivation.

4.4.7.4 DBP Formation and Disinfectant Combinations

The type and amount of DBPs formed by any disinfectant combination depends on the water quality, agent used for primary disinfection, the agent used for secondary disinfection and what, if any, preoxidant is used.

A specific combination that is appropriate for one raw water source may in fact cause an increase in DBP when applied to another. There is no "silver bullet" that is applicable for all raw water sources.

As a preoxidation step, chlorination of raw water with high NOM combined with chlorine as a secondary disinfectant produces the highest levels of chlorinated DBPs (CDBPs). Use of an alternate preoxidant that does not produce CDBPs and moving the point of chlorination downstream of NOM removal processes will reduce the overall formation of CDBPs.

The choice of preoxidants that limit CDBP production include ozone and chlorine dioxide. When considering preoxidants, the possible formation of other DBPs should be considered. The choice of ozone is based on the bromide levels and the cost of providing biologically active filtration. In raw waters with low pH and high bromide concentrations, brominated organic byproducts are produced. In raw waters with high pH and high bromide concentrations, bromate is produced. The addition of chlorine dioxide will produce chlorite and chlorate and may form some oxygenated DBPs (e.g., maleic acids).

In addition to Table 4-20, a table of the potential benefits and simultaneous compliance issues of the various combinations of primary and residual disinfectants is summarized in Table 4-21. Table 4-22 summarizes the potential DBPs formed by various combinations of disinfectants.

TABLE 4-20
PRIMARY/SECONDARY DISINFECTANT COMBINATIONS AND TYPICAL APPLICATIONS IN WATER TREATMENT

Primary / Secondary	Typical Application*	Comment
Chlorine/Chlorine	• Low THMFP raw water. • Low TOC. • Conventional treatment with optimal coagulation.	• Most commonly used disinfection scheme. • Effective system.
Chlorine/Chloramine	• High THM production. • Typical with conventional treatment.	• Chlorine to provide disinfection. • Monochloramine to provide residual and limit DBP formation.
Chlorine dioxide/ Chlorine dioxide	• High DBP production. • Requires filter process to remove Cryptosporidium. • Low chlorine dioxide demand in treated water.	• Primary and secondary usage requires a limit on chlorine dioxide dose to reduce residual chlorate/chlorite.
Chlorine dioxide/ Chloramine	• High DBP production. • Requires filtration to remove Cryptosporidium.	• Primary chlorine dioxide dose limited to residual chlorate/chlorite. • Stable, low reactive secondary disinfectant.
Ozone/Chlorine	• Moderate DBP formation. • Direct or no filtration. • Low THMFP.	• Highly effective disinfection to achieve high log inactivation. • Low THMFP to accept free chlorine. Requires post GAC filtration.
Ozone/Chloramine	• Moderate DBP formation. • Direct or no filtration. • Higher THMFP.	• Highly effective disinfection to achieve high log inactivation. • Low THMFP. • Requires post GAC filtration.
UV/Chlorine	• UV effective for *Giardia* and *Cryptosporidium* inactivation. • Chlorine for residual. • Low THMFP and HAA5 potential.	• High UV doses required for virus inactivation. • May require use of chlorine for virus inactivation and residual. • May reduce clearwell size.
UV/Chlorine/ Chloramine	• Moderate to Hi DBP formatting • Obtain virus inactivation • UV for Giardia and *Cryptosporidium* inactivation.	• May not be suitable for system with large on site storage due to DBP formation.
UV/Chloramine	• UV effective for *Giardia* and *Cryptosporidium* inactivation. • Chloramine for residual. • Low THMFP and HAA5 potential.	• High UV doses required for virus inactivation, if required due to small on site storage. • May require alternative disinfectant for virus inactivation; typically chlorine.

Source: Alternative Disinfectants and Oxidants Manual, USEPA 1999

Notes:

* Low DBP formation is defined as producing less than the Stage 1 and 2 DBPR Standard (less than 0.080 mg/L TTHM; less than 0.060 mg/L HAA5). High DBP formation is defined as producing more than the Stage 1 and 2 DBPR Standard (greater than 0.080 mg/L TTHM; greater than 0.060 mg/L HAA5).

**Chlorine refers to free chlorine.

TABLE 4-21
SUMMARY OF POTENTIAL BENEFITS AND ADVERSE EFFECTS ASSOCIATED WITH DIFFERENT COMBINATIONS OF PRIMARY AND SECONDARY DISINFECTANTS

Disinfection Switch Primary / Secondary (from → to)	Potential Benefits	Potential Adverse Effects	Drinking Water Regulation(s) Impacted
Chlorine/Chlorine → Chlorine/Chloramine	• Improved ability to maintain a disinfectant residual. • Lower TTHM and HAA5. • Possible improved biofilm control. • Improved taste and odor control.	• Excess ammonia can cause nitrification. • Possible elevated nitrite/nitrate levels. • Possible corrosions concerns. • Concerns for dialysis patients, fish owners, and other industrial customers.	• Stage 1 DBPR. • Stage 2 DBPR. • SWTR. • TCR. • LCR. • ESWTR.
Chlorine/Chlorine → Ozone/Chlorine	• Lower TTHM and HAA5. • *Cryptosporidium* inactivation. • Improved taste and odor control.	• Bromate MCL concerns. • Additional bromate monitoring required. • May increase brominated DBPs. • Increased AOC may enhance biofilm growth.	• Stage 1 DBPR. • Stage 2 DBPR. • ESWTR. • TCR.
Chlorine/Chlorine → Ozone/Chloramine	• Lower TTHM and HAA5. • *Cryptosporidium* inactivation. • Improved ability to maintain a disinfectant residual. • May improve taste and odor control.	• Nitrification may increase. • Possible elevated nitrite/nitrate levels. • Possible corrosions concerns. • Bromate MCL concerns. • Additional bromate monitoring required. • Increased AOC may enhance biofilm growth. • Concerns for dialysis patients, fish owners, and other industrial customers.	• Stage 1 DBPR. • Stage 2 DBPR. • SWTR. • ESWTR. • TCR. • LCR.
Chlorine/Chloramine → Chlorine Dioxide/Chloramine	• Lower TTHM and HAA5. • *Cryptosporidium* inactivation. • *Giardia* and virus inactivation. • Can control iron and manganese. • Chlorite from chlorine dioxide may control nitrification.	• Additional chlorine dioxide and chlorite monitoring required. • Chlorite MCL concerns. • Chlorine dioxide MRDL concerns.	• Stage 1 DBPR. • Stage 2 DBPR. • ESWTR. • LCR.
Chlorine/Chloramines → Ozone/Chloramine	• Lower TTHM and HAA5. • *Cryptosporidium* inactivation. • *Giardia* and virus inactivation. • Improved taste and odor control.	• Increased AOC may encourage nitrification and biofilm growth. • Bromate MCL concerns. • Additional bromate monitoring required. • Potential for ozone taste and odor issues • May create brominated DBPs.	• Stage 1 DBPR. • Stage 2 DBPR. • ESWTR. • TCR. • LCR.

Disinfection Switch Primary / Secondary (from → to)	**Potential Benefits**	**Potential Adverse Effects**	**Drinking Water Regulation(s) Impacted**
Ozone/Chlorine → Ozone/Chloramine	• Lower TTHM and HAA5. • Improved ability to maintain a disinfectant residual. • *Giardia* and virus inactivation. • Improved taste and odor control.	• Increased AOC may encourage nitrification. • Concerns for dialysis patients, fish owners, and other industrial customers. • Possible corrosions concerns.	• Stage 2 DBPR. • SWTR. • TCR. • LCR.
Chlorine/Chlorine or Chlorine/Chloramine → UV/Chlorine or UV/Chloramine	• Lower TTHM and HAA5. • *Cryptosporidium* inactivation. • *Giardia* and virus inactivation.	• UV less effective at inactivating viruses compared to chlorine. • UV is not an oxidant. • Reduced taste and odor control.	• Stage 2 DBPR. • SWTR. • ESWTR.
Ozone/Chlorine → Ozone/UV/Chlorine	• Additional *Cryptosporidium* inactivation. • *Giardia* and virus inactivation. • Increased taste and odor control.	• Ozone may lower UVT unless quenched.	• ESWTR.

Source: Adopted from the Simultaneous Compliance Guidance Manual for the LT2ESWTR and Stage 2 DBPR, USEPA 2007.

TABLE 4-22
DBPS ASSOCIATED WITH VARIOUS COMBINED OXIDATION/DISINFECTION PROCESSES

Disinfection Alternative			Potential DBPs	Remarks
Preoxidation	**Primary Disinfection**	**Secondary Disinfection**		
Chlorine	Chlorine	Chlorine	Halogenated DBPs (XDBPs)	Maximum XDBP formation compared to all other strategies. Principal components are TTHMs and HAAs.
			Aldehydes	Formed at relatively low levels.
Chlorine	Chlorine	Chloramine	XDBPs Cyanogen chloride Cyanogen bromide	Formation of XDBPs (specifically TTHMs and HAA5s) significantly reduced compared to chlorine/chlorine/ chlorine.
			Aldehydes	Formed at relatively low levels.
Chlorine dioxide	Chlorine dioxide	Chlorine	XDBPs	Formation of XDBPs may be decreased by delaying the point of chlorine addition.
			Aldehydes, carboxylic acids, maleic acids	Formed at relatively low levels.
			Chlorate Chlorite	Chlorite is a significant product of chlorine dioxide decay.
Chlorine dioxide	Chlorine dioxide	Chloramine	XDBPs	Formation of XDBPs (especially TTHMs and HAA5s) minimized by avoiding use of free chlorine.
			Aldehydes, carboxylic acids, maleic acid	Formed at relatively low levels.
			Chlorate Chlorite	Chlorite is a significant product of chlorine dioxide decay.
Potassium permanganate	Chlorine	Chlorine	XDBPs	Formation of XDBPs may be decreased by delaying the point of chlorine addition.
			Aldehydes	Formed at relatively low levels.
Potassium permanganate	Chlorine	Chloramine	XDBPs Cyanogen chloride Cyanogen bromide	Formation of XDBPs may further be decreased compared to potassium permanganate/chlorine/chlorine.
			Aldehydes	Formed at relatively low levels.

Disinfection Alternative			Potential DBPs	Remarks
Preoxidation	**Primary Disinfection**	**Secondary Disinfection**		
Ozone	Ozone	Chlorine	XDBPs	Formation of certain XDBPs may increase or decrease compared to chlorine/chlorine/chlorine. Brominated byproducts may be of concern when bromides are present in the raw water.
			Bromate, Aldehydes, carboxylic acids	Although formed at relatively high levels, significant amounts of this BOM can be removed through biological filtration.
Ozone	Ozone	Chloramine	XDBPs Cyanogen chloride Cyanogen bromide	Formation of XDBPs (especially TTHMs) minimized by avoiding use of free chlorine.
			Bromate, Aldehydes, carboxylic acids	Although formed at relatively high levels significant amounts of this BOM can be removed through biological filtration.
Chlorine	UV	Chloramine	XDBPs Cyanogen chloride Cyanogen bromide	May form XDBP from pre-oxidation.
			Aldehydes	Low levels.
			Aldehydes, carboxylic acids	Very low, if any, due to less reactive oxidants.

Source: Adopted from the Alternative Disinfectants and Oxidants Manual, USEPA 1999

4.4.8 Physical Chemical Processes

The following paragraphs discuss the ability of various filtration processes to remove bacteria, viruses, and protozoa; which determine the amount of disinfection required. Typically these technologies are used within a multi-barrier approach for maximum removal or inactivation of the microbiological agents of concern. The EPA has allowed a log credit for inactivation under the SWTR for various processes or combinations of processes in order to meet their goal of 3-log inactivation of *Giardia* and 4-log inactivation of viruses. These provisions have been carried over in the IESWTR. The IESWTR also added requirements for 2-log reduction of *Cryptosporidium* which were increased to a maximum of 3-log reduction under the LT2ESWTR.

The filtration process provides a physical barrier for removal of particles of known size (or larger). This is an effective method for removing bacteria, viruses and protozoa. However, there are implications of contamination from the filter backwash. To mitigate these risks, the EPA published the Filter Backwash Rule (FBRR) in 2001. Under the FBRR, PWSs are required to return specific recycle flows through all processes of the system's existing conventional or direct filtration system or at an alternate location approved by the state.

4.4.8.1 Conventional Treatment

Conventional systems involve chemical addition, coagulation, flocculation, clarification or sedimentation and rapid filtration. Dissolved air flotation (DAF) could also be used in connection with the above noted processes. Table 4-24 provides the removal capabilities for conventional systems.

4.4.8.2 Direct Filtration

Direct filtration involves chemical addition followed by flocculation and filtration. A modification of this is “in-line” filtration, which is simply chemical addition and filtration. These processes, in particular in-line filtration, are limited to waters with low turbidities. The filtration step is typically rapid filtration or slow sand filtration. Table 4-24 summarizes the removal capabilities of these filtration technologies.

4.4.8.3 Slow Sand and Diatomaceous Earth Filtration

Slow sand and diatomaceous earth filtration systems have also demonstrated capabilities to remove pathogens. Table 4-23 provides the removal capabilities for slow sand filters.

TABLE 4-23
REMOVAL CAPABILITIES OF FILTRATION PROCESSES

Process	Viruses	*Giardia*	*Cryptosporidium*
Conventional Treatment	1- to 3-log	2- to 6-log	2-to 6-log
Direct Filtration	1- to 2-log	3- to 4-log	2- to 3-log
Slow Sand Filtration	1- to 3-log	>3-log	>3-log
Diatomaceous Earth Filtration	1- to 2-log	>3-log	>3-log

Source: National Primary Drinking Water Regulations: Interim Enhanced Surface Water Treatment Rule Notice of Data Availability, November 3, 1997

4.4.8.4 Membranes

Membrane technology is increasingly being targeted for installation at water treatment plants for particle and pathogen removal. This technology is typically referred to as being one of four general types as follows:

- Microfiltration: capable of removing particles of approximately 0.05 to 2 µm.
- Ultrafiltration: capable of removing particles of approximately 0.005 to 0.75 µm.
- Nanofiltration: capable of removing particles of approximately 0.001 to 0.025 µm.
- Reverse Osmosis: capable of removing particles of approximately 0.0001 to 0.0075 µm.

The City of Los Angeles pilot tested four membrane systems (2 MF systems and 2 UF systems) from 1997 to 1999 for treatment of water from reservoirs that receive treated surface water. Karimi et al., (2000) reported 3- to 4-log reduction of particles in the size range of *Giardia* and *Cryptosporidium.* The Yorkshire

Water Services (a private water purveyor in the UK) tested UF systems from six suppliers for microbiological control (Franklin et al., 2000). The membranes tested for *Cryptosporidium* removal used MS2 phage as an indicator organism. Log removals were between 1.13 and 8.6. Gullick et al., (2000) found similar *Cryptosporidium* reductions (4.1 to 4.8-log reduction) for two composite thin walled NF systems and a cellulosic ester UF system.

The SWTR primarily considers conventional filtration plants and to a lesser extent, other media filtration technologies such as direct, slow sand, and diatomaceous earth filtration. All other filtration technologies are considered alternative technologies (AFTs). Under the existing SWTR, log removal credits for AFTs, such as membrane processes and bag and cartridge filters, were not explicitly addressed, but instead covered under a special State primacy requirement. For compliance with the SWTR and ESWTRs, many States grant removal credits to membrane processes based on the guidelines for AFTs in the *Guidance Manual for Compliance With the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources* (commonly called the SWTR Guidance Manual) (USEPA, 2005). However, there is significant variability in the manner in which States regulate membrane processes. A survey was conducted and published in 2001 of the 29 state primacy agencies that were or did implement regulatory requirements for MF/UF systems. Table 4-24 summarizes the log removal credits given by the majority of the states agencies surveyed to MF/UF systems based on *Giardia*, *Cryptosporidium* and viruses.

TABLE 4-24
REMOVAL CAPABILITIES OF MF AND UF MEMBRANE FILTRATION PROCESSES

Organism	Size (μm)	Ultrafiltration*	Microfiltration*
Giardia	7 to 14	3.0	3.0
Cryptosporidium	3 to 5	2.0	2.0
Viruses	0.0025 to 0.1	0	0

* Log removal credit based on majority of 29 state agencies reporting as a result of survey (USEPA, 2001).

The LT2ESWTR builds on the previous SWTRs by requiring additional treatment for those systems with elevated influent *Cryptosporidium* levels. The rule identifies a number of "toolbox" technologies that may be employed to achieve additional *Cryptosporidium* treatment requirements. The range of removal or inactivation credits allocated to each of the various toolbox options under the rule varies based on the capabilities of the particular treatment technology. Because the various types of membrane filtration processes represent one of these toolbox alternatives, utilities have the option of using membrane filtration for compliance with the rule requirements as a distinct technology rather than simply as a general AFT. Consequently, the EPA developed the *Membrane Filtration Guidance Manual* to provide specific regulatory requirements and associated guidance, for membrane filtration processes used for compliance with the LT2ESWTR (USEPA, 2005).

It should be noted the regulatory framework established under the LT2ESWTR applies only to membrane filtration processes used to achieve *Cryptosporidium* removal for rule compliance. Thus, the LT2ESWTR does not supersede or conflict with any prior SWTR, including the allowance of State primacy for regulating membrane filtration processes as AFTs for *Giardia* and virus removal. The LT2ESWTR regulatory framework could be employed for other applications of membrane filtration (e.g., for the removal of *Giardia*, viruses, or other pathogens), albeit solely at the discretion of the State (USEPA, 2005).

In order to receive removal credit for *Cryptosporidium* under the LT2ESWTR, a membrane filtration system must meet the following three criteria:

1. The process must comply with the definition of membrane filtration as stipulated by the rule.
2. The removal efficiency of a membrane filtration process must be established through a product-specific challenge test and direct integrity testing.
3. The membrane filtration system must undergo periodic direct integrity testing and continuous indirect integrity monitoring during operation.

The rule does not prescribe a specific removal credit for membrane filtration processes. Instead, removal credit is based on system performance as determined by challenge testing and verified by direct integrity testing. Thus, the maximum removal credit that a membrane filtration process may receive is the lower value of either the removal efficiency demonstrated during challenge testing or the maximum log removal value that can be verified by the direct integrity test used to monitor the membrane filtration process (USEPA, 2005).

For the purposes of compliance with the LT2ESWTR, membrane filtration is defined as a pressure- or vacuum-driven separation process in which particulate matter larger than 1 μm is rejected by an engineered barrier primarily through a size exclusion mechanism and which has a measurable removal efficiency of a target organism that can be verified through the application of a direct integrity test (USEPA, 2005).

A direct integrity test is defined as a physical test applied to a membrane unit in order to identify and isolate integrity breaches. The LT2ESWTR does not mandate the use of a specific type of direct integrity test, but rather performance criteria that any direct integrity test must meet. The three criteria include the following:

1. Resolution: The direct integrity test must be applied in a manner such that a 3 mm breach contributes to the response from the test.
2. Sensitivity: The direct integrity test must be capable of verifying the log removal value awarded to the membrane process by the State.
3. Frequency: The direct integrity test must be applied at a frequency of at least once per day, although less frequent testing may be permitted by the State at its discretion if appropriate safety factors are incorporated (USEPA, 2005).

Because currently available direct integrity testing methods require the membrane unit to be temporarily taken out of service, or are either too costly or infeasible to apply continuously, direct testing is only conducted periodically.

Thus, in the absence of a continuous direct integrity test that meets the resolution and sensitivity requirements of the LT2ESWTR; continuous indirect integrity monitoring is required. Although the indirect monitoring methods are typically not as sensitive as direct tests for detecting a loss of membrane integrity, the indirect methods do provide some measure of performance assessment between applications of direct testing. For the purposes of the LT2ESWTR, indirect integrity monitoring is defined as monitoring some filtrate water parameter that is indicative of the removal of particulate matter, and "continuous" is defined as monitoring at a frequency of no less than once every 15 minutes. Although turbidity monitoring is specified as the default method of continuous indirect integrity monitoring under the rule, other methods, such as particle counting or particle monitoring, may be used in lieu of turbidity monitoring at the discretion of the State. For any indirect method used, a control limit must be established that is indicative of acceptable performance. Monitoring results exceeding the control limit for a period of more than 15 minutes must trigger immediate direct integrity testing (USEPA, 2005).

4.4.8.5 Miscellaneous Filtration Technologies

Point Of Use Filters

This refers to "under the sink" type filters which are capable of removing particles in down to 0.5 µm. While possibly effective, this type of treatment is not considered acceptable by the EPA as the responsibility of water treatment and filter maintenance is now on the home owner, rather than the water utility.

4.5 STORAGE AND DISTRIBUTION

4.5.1 Introduction

The storage and distribution system is defined as those components of the water supply system that extend from the water treatment plant or point of disinfection, to the end user. Balancing storage is considered to be an integral part of the distribution system.

The objective of this section is to:

1. Provide an overview of the physical, chemical and biological processes that occur in the distribution system;
2. Describe characteristics of the distribution system;

3. Review the effectiveness of various treatment and disinfection methods and their effect on distribution system water quality;
4. Describe current strategies for water quality improvements in the distribution system; and
5. Identify new development and research underway to improve water quality in the distribution system.

4.5.2 Storage and Distribution System Characteristics

Water distribution systems are designed to meet multiple purposes including delivering drinking water at the required flow and pressure to individual consumers (residences), institutions (schools, hospitals, etc.), agriculture (irrigation), and to commercial and industrial establishments. The proper function of a water distribution system is critical to providing sufficient drinking water to consumers, as well as for fire protection and emergency situations (Mays, 2000).

Most water distribution systems consist of distribution piping, water storage (tanks or reservoirs), and pumping stations. Water distribution storage is provided to ensure supply (peak demand, fire protection, emergency conditions, etc.), maintain pressure, equalize pumping and treatment rates, minimize the size of the transmission mains, and improve operational flexibility (Mays, 2000). Water storage tanks are normally covered and operate with a water surface open to air. Hydraulically, the distribution system is designed to meet the peak water demand conditions.

The water quality in the distribution system can be influenced by the finished water characteristics (pH, alkalinity, temperature, hardness, pathogen concentrations, etc.), the type of disinfectant used, the material and condition of the pipes, valves and storage tanks, and the amount of time the water is kept in the system. Water systems with long residence times provide conditions that may maximize the production of DBPs, increase the loss of residual disinfectant, and allow for the formation of biofilms. Measures to improve water quality or prevent its deterioration in the distribution system include water treatment process changes (i.e. enhanced NOM removal, switching disinfectants, corrosion control, etc.) pipe repair, relining or replacement, and modifications to the distribution system operation such as keeping less water in storage, hydrant flushing or looping dead end sections (Mays, 2000).

In recent years, there has been an increased interest by PWSs in understanding what is happening to water quality once the finished water leaves the treatment plant and enters the distribution system. This is due primarily to the publication of the Stage 2 DBPR and the LT2ESWTR. As previously mentioned, the Stage 2 DBPR requires PWSs to monitor multiple sites within the distribution system to report locational running average concentrations for TTHMs and HAAs. The LT2ESWTR may require some PWSs to provide additional treatment or enhance existing treatment techniques based on *Cryptosporidium* concentrations.

In addition, due to aging water infrastructure throughout the United States, many PWSs and utilities are examining whether the replacement, repair or relining of piping will improve water quality in the distribution system, help with compliance under existing regulations (LCR, TCR, DBPRs, etc.) and to ensure consistent supply and pressure to all of their consumers.

4.5.3 Effect of Treatment and Disinfection on Distribution System Water Quality

The transformation and deterioration in water quality can be physical, chemical, or microbiological in nature and occurs either in the bulk water phase or through interaction with the pipe walls (Mays, 2000). Water treatment processes and disinfection techniques have a direct impact on the finished water quality in the distribution system.

Water Treatment Processes

The transformation processes in the distribution system that may result from water treatment processes include, but are not limited to the following:

1. Loss of disinfectant residual.
2. Growth of DBPs.
3. Internal corrosion of pipe wall or lining.
4. Growth of biofilm.

These four transformation processes are described briefly below.

Loss of disinfection residual: The most common chemicals used to provide secondary disinfection, or distribution system residuals, are chlorine and chloramine, followed by chlorine dioxide. The rate of disinfectant decay in the distribution system can occur in the bulk phase (reaction with organic compounds in the water) or as the disinfectant reacts with materials on or near the pipe wall such as iron or with a biofilm. At typical bulk decay of residual oxidants based on the decay equations in the US EPA Water Treatment Plant model are provided for comparison purposes.

TABLE 4-25
TYPICAL DISINFECTANT RESIDUAL COMPARISON BASED ON BULK DECAY RATE

Disinfectant	Residual After 18 hours
Chlorine	0.1
Chloramine	0.9
Chlorine Dioxide	0.6

Input Boundary Conditions:
TOC – 2.0 mgC/L, UV254 = 0.03 1/cm, Dose = 1.0 mg/L, pH 7.0, Temp. 10 C.

Typical half-lives for chlorine in the bulk phase range from several days to several weeks, depending on water temperature and the concentration and reactivity of the NOM (expressed as DOC) in the water (Mays, 2000). The half-life of chlorine and chlorine dioxide is significantly reduced for reactions on or near the pipe wall, especially in older, unlined cast iron or steel pipes. The reaction rates are much higher compared to those in the bulk phase; therefore the half-life can be several hours as compared to several days or weeks.

Chloramine is much more stable than chlorine; therefore provide a longer lasting residual. The rate of decay is much slower than that of chlorine. However, as previously mentioned, under certain pH conditions and higher chlorine to ammonia ratios, nitrification can occur which may lead to increased corrosion.

Similar to chlorine and chloramine, the rate of decay for chlorine dioxide is affected by the pH, water temperature, the concentration of chlorine dioxide used for disinfection, and the concentration of NOM and other inorganic substances (i.e. manganese, iron, sulfides, etc.). When applied to finished water with low oxidant-demand, chlorine dioxide residuals may last several days.

Besides switching to a more stable secondary disinfectant (i.e. chloramine), a water utility can implement the following strategies to mitigate the loss of disinfectant residual.

1. Develop a pipe replacement or relining program to replace older and corroded pipes.
2. Institute a uni-flushing program: flushing of localized hydrants for removal of sediments, biofilms and/or organic matter that may degrade residuals found in the distribution system. The frequency of flushing will be specific to the water quality characteristics observed in the distribution system. Typically, flushing is instituted in the fall and spring of each year.
3. Reduce the amount of time the water spends in the distribution system by keeping less water in storage, exercising storage tanks more frequently, valve off pipes, or looping dead-end sections.
4. Make changes to beginning stages in the treatment process to remove greater amounts of NOM, such as enhanced coagulation or GAC filtration.
5. Booster disinfection: the practice of adding disinfectant at points in the distribution system (beyond the entrance to the system) where there is large disinfectant demands such as dead ends or at distant points in the distribution system.

Growth of disinfection byproducts (DBPs): As previously discussed in this Section, chlorine, chloramine, chlorine dioxide, and ozone all react with NOM to produce DBPs. Although production rates of DBPs are less in the distribution system as compared to the initial application point at the treatment plant (primary disinfection), DBPs will continue to form in a distribution system using a chlorine residual as long as there is enough chlorine residual and reactable precursor compounds in the water. It is not unrealistic to see 50 percent or more of the TTHMs produced to occur in the distribution system (Mays, 2000).

The control of DBPs and the possible treatment technologies available to reduce DBP formation has been previously summarized in this section and is not repeated here.

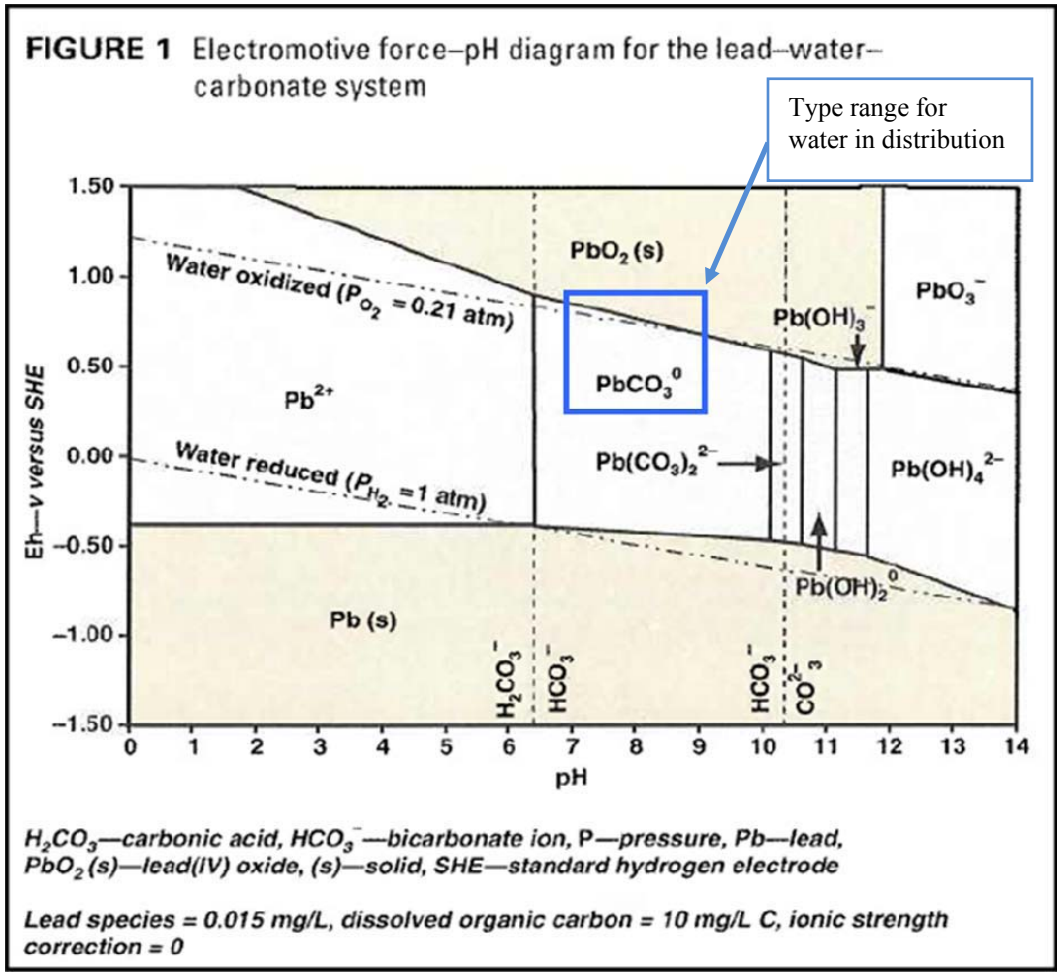
Internal corrosion: Internal corrosion, or the deterioration of the inside wall or lining of a pipe caused by reactions with water, can result in the dissolution of metals (i.e. lead and copper), impart a taste to the water, cause staining of plumbing fixtures, provide an attachment location for pathogenic microorganisms, reduce a pipes hydraulic capacity, and may result in leaks or breaks (Mays, 2000). Flow velocity, pH, alkalinity, dissolved oxygen concentration, total dissolved solids concentration, hardness, and bacteria concentrations are all factors affecting the rate and amount of corrosion in pipes. The general approach to controlling corrosion in the distribution system involves modifying the water quality to make the finished water less corrosive (i.e. pH adjustment), adding a protecting lining between the water and the pipe (addition of lime and soda ash or corrosion control inhibitors such as orthophosphates), or replacing pipe materials that are less prone to corrosion (i.e. PVC) (Mays, 2000).

Unintended consequences can occur when making treatment changes at the WTF or treatment of the distribution water. One example of this is that some facilities in the southern part of the United States have experience severe lead corrosion issues when the primary coagulant was changed from alum to polyaluminum chloride as the main coagulant (Edwards and Triantafyllidou, 2007). Using bench scale testing, the authors suggested that the increased levels of chloride made the finished water more aggressive towards lead solder and caused galvanic corrosion between the solder and copper or brass connections. The authors went on the suggest that if the chloride to sulfate mass ratio (CSMR) was maintained below 0.5, then the finished water tended to be less favorable for lead leaching due to galvanic corrosion.

Another example of this is the switch from chlorine to chloramine as the distribution disinfectant. DC Water and Sewer Authority (WASA) recently made this switch over and later found that the lead scales which had formed on the service lines was incompatible with the change. Previously WASA had been maintaining a high chlorine residual in the distribution system which formed lead dioxide ($PbO_{2(s)}$) (Figure 4-5).

When the switch was made to chloramine, the oxidization-reduction potential (ORP) dropped and the lead dioxide become unstable and dissolved. The result was a spike in lead levels in the distribution system.

FIGURE 4-5
LEAD SCALES LIKELY TO FORM ON LEAD CONTAINING SERVICES AND OTHER LEAD CONTAINING APPURTENANCES (SOURCE: LYTLE AND SCHOCK, JAWWA NOVEMBER, 2005)



Regrowth/Biofilms: Biofilms are defined as a complex mixture of microbes, organic, and inorganic material accumulated amidst a microbially-produced organic polymer matrix attached to the inner surface of the distribution system (USEPA, 2006). This slime layer anchors firmly to the pipe surface and provides a protective environment for

microorganisms and many pathogens to adhere to and flourish. Biofilm formation is dependent on a number of factors including organic matter present in the distribution system, temperature, disinfectant residual concentration and the pipe material.

Biofilms attach and grow in areas of corrosion or slow flow where sediments accumulate and water velocity is not fast enough to disrupt the biofilm development. Cement lined pipe and PVC pipe materials are less receptive to biofilm growth, as compared to iron pipe. In addition, water storage tanks and reservoirs may also accumulate biofilm deposits on side walls and in sediments on the bottom of the tank. These deposits may cause taste and odor issues due to the static nature of the water and the anaerobic growth of various organisms in the biofilm (Mays, 2000).

The growth of biofilms in the distribution system is significant. The release or sloughing of biofilm organisms into the water in the pipe network and storage locations within the distribution system may pose a health concern since some of these organisms are known to be pathogenic. It is also known that some organisms can interfere with the detection of coliforms and other bacteria. In addition, concerns have arisen that biofilm occurrences may be hiding fecal contamination either from inadequate treatment or contamination of the distribution system (Mays, 2000).

Elevated levels of disinfectants (booster disinfection), pipe flushing, storage tank cleaning, adjusting pH and alkalinity of the finished water, as well as adding corrosion inhibitors and replacing old leaking pipes, are methods that may control biofilm growth within the distribution system (Mays, 2000).

Disinfection Techniques

The effects of primary disinfection with ozone, chloramine, chlorine dioxide and UV light on DBP formation, various process interactions, and the water quality in the distribution system have been discussed previously in this section. Therefore, these effects are not repeated here.

However, the effect of secondary disinfection (i.e. chlorine residual) on the water quality in the distribution system is a growing concern among water utilities. Water utilities face new challenges and concerns as they try to simultaneously comply with multiple regulations regarding improved water quality. Compliance with the LCR, TCR, and the Stage 1 and Stage 2 DBPRs have many water utilities using and others considering using chloramine in place of free chlorine. As mentioned, chloramine provide greater stability (longer lasting residual), and produce less DBPs than chlorine. Therefore, more and more water utilities are switching to chloramine for better compliance with the TCR and DBPRs. However, new information and research about the effects of switching from free chlorine to chloramine on the oxidation reduction potential (ORP), lead chemistry, scale formation, and scale destabilization involving NOM in the distribution system have explained why some water utilities to fall out of compliance with the LCR. Most notably, in Washington D.C. the transition from chlorine to chloramine led to a significant increase in lead release from piping in their distribution system (AwwaRF, 2006).

Part 1 of a study published by the AWWA Research Foundation in 2006 focused on the effect of changing disinfectants on distribution system lead and copper release. This literature review aimed at building upon the water industry's limited understanding of the potential consequences of changing disinfectants and how it would impact lead and copper release (AwwaRF, 2006). A significant result of this study concluded that formation and stability of lead (IV) oxides are also important to control lead solubility. Part 2 of the study, which should be available in 2010, will update the literature review from part 1 and summarize laboratory studies, pipe loop testing, and field sampling results.

4.5.4 Strategies to Improve Water Quality in Distribution Systems

As previously mentioned, the water quality in the distribution system can be influenced by the finished water characteristics (pH, alkalinity, temperature, hardness, pathogen concentrations, etc.), the type of disinfectant used, the material and condition of the pipes, valves and storage tanks, and the amount of time the water is kept in the system. Strategies to improve water quality and reduce the formation of DBPs in the distribution system may include but are not limited to the following:

- Changes to water treatment processes such as:
  - Enhanced coagulation/softening for NOM removal.
  - Switching disinfectants.
  - Adjust finished water pH and alkalinity.
  - Maintaining a stable residual.
  - Addition of corrosion control
- Pipe repair, relining or replacement.
- Modifications to the distribution system operation such as:
  - Improving mixing in storage facilities.
  - Minimizing water residence time in storage facilities and transmission mains by looping or flushing
- Implement a water flushing program.
- Regular finished water storage inspection and cleaning.
- Local treatment within the distribution to remove DBPs or TOC:
  - Stripping TTHMs.
  - Nanofiltration to remove TOC or DBPs.
  - GAC.
- Booster chlorination.

Booster chlorination can be installed to improve residual disinfectant concentrations in the far reaches distribution system, thus reducing the dose at the water treatment plant and thereby providing some reduction in DBPs. Typical booster locations are located near storage tanks and dosed when water is leaving the storage tanks or reservoirs. The actual location should be based on hydraulic modeling to obtain the best results.

Sampling and Monitoring Requirements

Minimum sampling and monitoring requirements are established by the EPA and either enforced by the EPA or by the states that have primacy. Individual states may institute more stringent requirements if they believe it is necessary. Current sampling and monitoring requirements for various parameters in finished water such as disinfectant residual, coliform bacteria, TTHMs, HAA5s, lead, copper, etc. are summarized as part of the applicable rule or regulation. For example, monitoring and sampling requirements for TTHMs and HAA5s are summarized in the Stage 1 and Stage 2 DBPR.

A routine sampling and monitoring program should be implemented by the water utility to not only meet existing regulatory requirements, but to collect additional water quality information that may be useful to the water utility to improve the operation of the water treatment plant. Water quality parameters such as alkalinity, aluminum, ammonia, iron, pH, temperature, turbidity and taste and odor are among some of the additional parameters that should be considered as part of a water utilities routine sampling and monitoring program.

In addition, representative areas of the distribution system should be monitored for disinfection residual, total coliforms, and heterotrophic plate count (HPC) bacteria to assess whether a biological regrowth problem exists in the system which may trigger a TCR violation. AECOM's experience in conjunction with information found in literature suggests the following guidelines:

1. HPC counts less than 50 cfu/ml in 90% of samples. (USEPA standard is 500 cfu/ml)
2. Background counts determined during total coliform testing (filtration method) of less than two (2).
3. Disinfection residual detectable throughout the distribution system.
    a. Free and combined residuals within 0.2 mg Cl_2/L (for free chlorine only as distribution residual).
4. Adenosine triphosphate (ATP) measurements.
    a. < 0.5 pg/ml indicates stable water.
    b. 0.5 to 10 pg/ml indicates potential problem and action steps required..
    c. > 10 pg/ml indicates a need to increase disinfection.

These suggested guidelines are not representative for all systems and will vary depending on water quality characteristics observed in the distribution system. However, these guidelines provide a good starting point for establishing specific parameters and to provide safe drinking water. Further information concerning monitoring parameters and the evaluation and control of biological regrowth in the distribution system can be found in the AwwaRF study entitled, *Evaluating Biological Regrowth in Distribution Systems* published in 2000.

Modeling of Water Quality in Distribution System

A routine sampling and monitoring program alone can not provide a clear understanding of what is happening on a daily basis within the distribution system. Thousands of miles of piping, variable flow paths and travel times, different pump schedules, and varying fill and draw cycles for storage facilities make it difficult to exclusively rely on system-wide sampling and monitoring programs. For these reasons, modeling of water-quality in the distribution system is an excellent supplement to monitoring. Generally there are four types of mathematical models for the analysis of drinking water distribution systems:

1. Hydraulic models;
2. Models to calculate the evolution of non-reactive contaminants;
3. Models to calculate the evolution of disinfectants; and
4. Models of microbiological quality.

Distribution system models have allowed utility managers to analyze the water hydraulics, pressures and flows in a distribution system with some accuracy for the last 20 years. The models' capabilities have continued to expand in this time. Over the last 7 years, with more data available from water use patterns and from daily flow records, extended period simulations are now provided with more regularity. With modeling over time, water age and the decay of constituents in the water can be predicted for several scenarios.

Modeling of contaminant/constituents in a distribution system are modeled as:

$$C_{t+\Delta t} = C_t \times e^{-(\Delta t \times k)}$$

Where C_t = concentration at time zero, in mg/L

$C_{t+\Delta t}$ = concentration at time Δ, in mg/L

K = decay coefficient, in time $^{-1}$

As an example, recent studies have shown that in older water systems using ferrous type piping, there is considerable chlorine residual loss due to reactions with the pipe wall. For plastic or cement mortar lined pipe, chlorine residual decay is more associated with biofilm uptake.

Models can be used to identify where flushing of water mains may be necessary. The model hydraulics can identify which pipes have low velocities and therefore most susceptible to deposition of particulate matter. Models offer the water utility a cost effective way to study the hydraulics and water quality characteristics of the distribution system.

4.5.5 Development/Research in Distribution System Water Quality

A significant amount of research is ongoing, primarily through the Water Research Foundation (WRF, formerly the AwwaRF) and the EPA in conjunction with water suppliers. The following is a brief summary of the ongoing research regarding water quality and the distribution system.

***Switching Disinfectants**:* As mentioned previously, there is a continuing research effort into the dynamics of chloramine reactions within the distribution system due to issues with nitrification and the corrosion effects of switching from free chlorine to chloramine disinfection. There are WRF studies to be released in 2010 that will continue to build on the existing knowledge and quantify chloramine decay mechanisms and their relationship to supporting nitrification in distribution systems and the effect a change in disinfectant can have on lead and copper release.

Biofilms: A WRF study due to be published in 2010 regarding biofilms proposes to develop utility-friendly guidance on assessing and managing distribution system biofilms and present current state-of-the-science knowledge on biofilms in a practical format that is helpful to utility staff and other end users. Another WRF study to be published in 2012 focuses on the driving factors of when and how biofilms promote the persistence of pathogens introduced to distribution systems. The study proposes to clarify the mechanisms for better biofilm control to counteract their promotion of introduced pathogens and to develop a more practical, utility-oriented understanding of biofilms and the remaining biofilm information gaps.

Other Developments: Other prominent WRF studies to be released in the next two to three years that focus on the distribution system include:

1. The Fate of Non-Regulated DBPs in Distribution Systems.

2. The Relationship between Biodegradable Organic Matter (BOM) and Pathogen Concentrations in Premise Plumbing. Note premise plumbing includes that portion of the distribution system associated with schools, hospitals, public and private housing, and other buildings. It is connected to the main distribution system via the service line.
3. A study focused on developing a tool to help the water industry proactively identify management techniques that will be critical in addressing future regulations and in reducing the public health risks arising from distribution systems.
4. Influences of water chemistry on copper pitting and the corrosion of brass in premise plumbing systems.

4.6 POTENTIAL GROUNDWATER SOURCES

4.6.1 Introduction

One way of reducing DBPs if a system were to remain on chlorine as a secondary disinfectant, is to rely on sources of supply that have low DBP precursors (NOM levels) and/or do not require disinfection. Groundwater supplies typically require limited or no disinfection because the earth is capable of filtering out most bacteria. Even if a groundwater source requires chlorine disinfection, the resulting DBP levels tend to be low because the TOC levels in groundwater are low and less prone to DBP formation compared to surface water supplies.

Groundwater quality in Vermont may require treatment for other constituents such as iron, manganese, hydrogen sulfide, etc. Treatment typically would consist of catalytic media filtration, stripping and other similar treatment methods.

AECOM has evaluated all 15 water systems for potential groundwater supply sources. The results of our evaluation are summarized on Table 4-26, Groundwater Favorability in Sand-and-Gravel Aquifers, and Table 4-27, Groundwater Favorability in Fractured-Bedrock Aquifers. In this chapter we discuss the sources of data that were used to evaluate groundwater favorability, and an explanation of Tables 4-26 and 4-27. Finally, we discuss groundwater favorability in each of the 15 individual water systems along with accompanying maps depicting areas favorable for groundwater development. Our evaluation addresses favorability strictly from the standpoint of well yield. We have not

considered land use nor groundwater quality, especially groundwater contamination from man-made sources. These factors would have to be considered separately in future stages of groundwater planning, testing and development.

4.6.2 Sources of Data on Groundwater Favorability

Our evaluation involved a review of mapping available electronically through the Vermont Geological Survey (VGS). The State of Vermont has been active in recent years in preparing and posting geologic and groundwater resources mapping on-line. Below AECOM lists the types of maps that were reviewed together with comments on their value and limitations:

1. Groundwater Favorability Maps prepared by A.L. Hodges in the mid 1960s. These maps were done at the drainage-basin scale (e.g., Winooski River, Lamoille River, etc.) and cover the glacial (sand-and-gravel) aquifers only. Hodges evidently compiled these maps from limited data, i.e., available well/boring logs, and perhaps field reconnaissance and map interpretation. Hodges developed three categories of groundwater favorability: “excellent”, “low to moderate”, and “low”. Only those areas mapped in the “excellent” category are suitable for municipal well development. Even though these maps are highly generalized, it was the only mapping available to us that addresses groundwater favorability directly. In addition, the “Water Well” listings give a clue – anecdotal as they might be – into actual well yields.

GROUNDWATER FAVORABILITY SUMMARY
SAND-AND-GRAVEL AQUIFERS

Existing Mapping – Sand & Gravel Aquifers	Summary of Groundwater Favorability – Sand & Gravel Aquifers
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, oi River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – no potential for municipal supply wells. 2) VCGI – isolated bodies of marine-sand and esker deposits may have some groundwater potential, especially w contact with surface water, such as the esker that meets Ransoms Bay.
es, A.L., Jr., 1966, *Groundwater Favorability Map, Batten Kill, Walloomsac d Hoosic River Basins, Vermont,* VT Dept. Water Resources and U.S. Geological ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – excellent potential extending from South Stream valley north along Walloomsac River valley to Para gpm well, 121 feet deep, reported in South Stream Valley. 2) VCGI – best potential in glaciofluvial deposits along Walloomsac River valley, Paran Creek valley; also poten South Stream.
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Winooski sin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer. nt, S. et al., 2009, *Surficial Geologic Map of the Burlington, Vermont 7.5-minute ngle*, Vermont Geol. Surv. Open File Report VG09-1. nt, S., 2009, *Surficial Geologic Map of the Colchester, Vermont 7.5-minute ngle*, Vermont Geol. Surv. Open File Report VG09-2.	1) Hodges – excellent potential in spots along Winooski River, and along shore of Lake Champlain; 450 gpm wel 2) VCGI – best potential in pebbly marine sand, widespread along Winooski River 3) Wright et al. – best potential along south side of Winooski River floodplain west of Winooski; also possible al near Lakeview Cemetery. 4) Wright – possible along south side of Winooski River floodplain beneath alluvium.
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Winooski sin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges et al. – excellent potential along Winooski River valley. 2) VCGI – best potential in sand and gravel deposits along Winooski River; less potential in outwash in Mill Broo
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Winooski sin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Lamoille sin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer. nt, S. et al., 2009, *Surficial Geologic Map of the Burlington, Vermont 7.5-minute ngle*, Vermont Geol. Surv. Open File Report VG09-1. nt, S., 2009, *Surficial Geologic Map of the Colchester, Vermont 7.5-minute ngle*, Vermont Geol. Surv. Open File Report VG09-2.	<u>Shelburne:</u> Hodges (1) – excellent potential along La Platte River to Shelburne Bay. Wright et al. – Possible groundwater supplies in sandy soils along La Platte River and Munroe Brook. VCGI – best potential in delta sands at mouth of La Platte River at Shelburne Bay. <u>S. Burlington:</u> Hodges (1) – excellent potential in floodplain of Winooski River. Wright et al. – Possible groundwater supplies in sandy soils along Potash Brook and Winooski River valleys. VCGI – pebbly marine sands widespread; best potential along Winooski River. Some potential in delta sands al Champlain. <u>Williston:</u> Hodges (1) – excellent potential in floodplain of Winooski River west of Route 2A. Wright et al. – Possible groundwater supplies in sandy soils at confluence of Muddy Brook and Winooski River. Williston. VCGI – best potential in pebbly marine sands in northwestern part of town along Winooski River, and delta sand northeastern part of town along Winooski River; less potential in kame terrace along tributary to Sucker Brook in <u>Winooski:</u> Hodges (1) – excellent potential in floodplain of Winooski River. One 450-gpm well, 31 feet deep reported. Wright et al. – best potential in sandy delta soils, widespread along Winooski River. VCGI – pebbly marine sands nearly ubiquitous; best potential along Winooski River. <u>Essex:</u> Hodges (1) – excellent potential in floodplain of Winooski River west of Route 2A. Wright et al. – best potential in sandy delta soils, widespread along Winooski River. Map incomplete for Essex. VCGI – best potential in pebbly marine sand widespread in southwest, delta sands widespread in southeast; other along Alder Brook valley north of Route 15. <u>Colchester:</u> Hodges (1) – excellent potential in floodplain of Winooski River west of Winooski. Hodges (2) – no potential in Lamoille River Basin.

Sand & Gravel Aquifers	Sand & Gravel Aquifers
	Milton: Hodges (2) – small area of low to moderate potential near Checkerberry Village. Wright – Possible groundwater supplies east of Checkerberry Village along Route 7, along Allan Brook south of Lamoille River. Map incomplete for Milton. VCGI – best potential in pebbly marine sands widespread in the center third of Milton, especially along the Lamo deposits on northern shore of Lake Champlain may also have potential Jericho: Hodges (1) – excellent potential along Winooski River valley. VCGI – best potential in sand and gravel deposits (lake sands, delta sands, and outwash) widespread along the W southwest, but also along Browns River, Lee River and Mill Brook.
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Lamoille sin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – no potential for municipal supply wells. 2) VCGI – isolated bodies of marine sand and beach gravel near southern boundary of Grand Isle may have mino
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, oi River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – no potential for municipal supply wells. 2) VCGI – no apparent potential for municipal supply wells.
es, A.L., Jr., 1967, *Groundwater Favorability Map, Otter Creek Basin, Vermont,* . Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – excellent potential along Otter Creek valley. One 280 gpm town well noted. 2) VCGI – best potential in glaciofluvial deposits east of Otter Creek near center of town, also along much of the
es, A.L., Jr. and Butterfield, D., 1968, *Groundwater Favorability Map, West- d River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – low to moderate potential along Deerfield River downstream of village. 2) VCGI – best potential from glaciofluvial deposit along Deerfield River downstream of village. Deposit could
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, oi River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – excellent potential along Missisquoi River east and south of town, and along tributary stream north c 2) VCGI – best potential in glaciofluvial deposits along Missisquoi River east of village, and North Branch Missi
es, A.L., Jr., 1967, *Groundwater Favorability Map, Otter Creek Basin, Vermont,* . Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer. Hoesen, J, 2009, Open File Report VG-09-7, *Surficial Geologic Map of Rutland,* , Vt Geol. Survey	1) Hodges – excellent potential along Otter Creek and south of Tenney Brook. 2) VCGI – best potential in glaciofluvial deposits, northeast part of Rutland along Mendon Brook and East Creek Creek, west and south of Rutland City. 3) Van Hoesen – best potential in kame terrace deposits, northeast part of Rutland along Mendon Brook, East Cre Reservoir.
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, Nulhegan- sic River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological gston, G. and Haselton, G., 1999, *Surficial Geologic Map of the Eastern Portion Johnsbury Quadrangle, VT,* VGS Open-File Report VG99-5, Vermont cal Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – excellent potential along Passumpsic River. One 800 gpm town well in St. Johnsbury noted. 2) Springston and Haselton – best potential in subaqueous glacial sand-and-gravel outwash, widespread along Pas 3) VCGI – best potential in esker deposits (10 segments mapped) along Passumpsic River, extending from the no Johnsbury.
es, A.L., Jr. and Butterfield, D., 1967, *Groundwater Favorability Map, oi River Basin, Vermont,* VT Dept. Water Resources and U.S. Geological ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – excellent potential along Missisquoi River south of town. 2) VCGI – best potential in large area of pebbly marine sand mapped north and south of town along Missisquoi R esker and kame-terrace deposits may have some potential.

Sand & Gravel Aquifers	Sand & Gravel Aquifers
es, A.L., Jr., 1967, *Groundwater Favorability Map, Otter Creek Basin, Vermont,* t. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – no potential for municipal supply wells. 2) VCGI – One body of glacial lake sand on the shore of Lake Champlain along the border between Bridport and potential.
es, A.L., Jr., 1967, *Groundwater Favorability Map, Otter Creek Basin, Vermont,* t. Water Resources and U.S. Geological Survey. ont Center for Geographic Information (VCGI), Surficial Geology Layer.	1) Hodges – no potential for municipal supply wells. 2) VCGI – no potential for municipal supply wells.

TABLE 4-27
GROUNDWATER FAVORABILITY SUMMARY
FRACTURED BEDROCK AQUIFERS

Water System	Mean Yield of Wells (mostly in bedrock)	Geographic Area of Town (approx. percent of total area)	Total Per Cent Area Underlain by Bedrock Wells Yielding 15-19 GPM and 20-35 GPM	General Favorability for Developing High Capacity Wells in Fractured Bedrock	Highest Yielding Well Noted by Hodges (1966, 1967, or 1968) in Gallons per Minute (GPM)
Alburgh Village	15 – 19 gpm 20 – 35 gpm	0% 0%	0%	Low	20
Bennington	15 – 19 gpm 20 – 35 gpm	60% 20%	80%	Moderate to High	150
Burlington	15 – 19 gpm 20 – 35 gpm	80% 0%	80%	Moderate to High	
Catamount-Bolton	15 – 19 gpm 20 – 35 gpm	0% 0%	0%	Low	
Champlain WD:					
Shelburne	15 - 19 gpm 20 - 35 gpm	50% 10%	60%	Moderate	
S. Burlington	15 - 19 gpm 20 - 35 gpm	60% 30%	90%	Moderate to High	
Williston	15 - 19 gpm 20 - 35 gpm	10% 15%	25%	Low	
Winooski	15-19 gpm 20 - 35 gpm	100% 0%	100%	High	
Essex	15 - 19 gpm 20 - 35 gpm	20% 0%	20%	Low	345
Colchester	15 - 19 gpm 20 - 35 gpm	85% 0%	85%	Moderate to High	
Milton	15 - 19 gpm 20 - 35 gpm	80% 0%	80%	Moderate to High	
Jericho	15 - 19 gpm 20 - 35 gpm	0% 0%	0%	Low	

TABLE 4-27 (Continued)
GROUNDWATER FAVORABILITY SUMMARY
FRACTURED BEDROCK AQUIFERS

Water System	Mean Yield of Wells (mostly in bedrock)	Geographic Area of Town (approx. percent of total area)	Total Per Cent Area Underlain by Bedrock Wells Yielding 15-19 GPM and 20-35 GPM	General Favorability for Developing High Capacity Wells in Fractured Bedrock	Highest Yielding Well Noted by Hodges (1966, 1967, or 1968) in Gallons per Minute (GPM)
Grand Isle CWD	15 - 19 gpm 20 - 35 gpm	10% 0%	10%	Low	
North Hero	15 - 19 gpm 20 - 35 gpm	0% 0%	0%	Low	
Proctor	15 - 19 gpm 20 - 35 gpm	50% 0%	50%	Moderate	
Readsboro Village	15 - 19 gpm 20 - 35 gpm	0% 0%	0%	Low	
Richford	15 - 19 gpm 20 - 35 gpm	<5% 0%	<5%	Low	
Rutland City	15 - 19 gpm 20 - 35 gpm	100% 0%	100%	High	
St. Johnsbury	15 - 19 gpm 20 - 35 gpm	60% 0%	60%	Moderate	
Swanton	15 - 19 gpm 20 - 35 gpm	50% 0%	50%	Moderate	
Tri Town WD:					
Addison	15 - 19 gpm 20 - 35 gpm	10% 10%	20%	Low	
Bridport	15 - 19 gpm 20 - 35 gpm	10% 5%	15%	Low	
Shoreham	15 - 19 gpm 20 - 35 gpm	30% 10%	40$	Moderate	

TABLE 4-27 (Continued)
GROUNDWATER FAVORABILITY SUMMARY
FRACTURED BEDROCK AQUIFERS

Water System	Mean Yield of Wells (mostly in bedrock)	Geographic Area of Town (approx. percent of total area)	Total Per Cent Area Underlain by Bedrock Wells Yielding 15-19 GPM and 20-35 GPM	General Favorability for Developing High Capacity Wells in Fractured Bedrock	Highest Yielding Well Noted by Hodges (1966, 1967, or 1968) in Gallons per Minute (GPM)
Vergennes-Panton:					
Vergennes	15 - 19 gpm 20 - 35 gpm	30% 15%	45%	Moderate	
Panton	15 - 19 gpm 20 - 35 gpm	20% <5%	<25%	Low	

2. Surficial Geologic Quadrangle maps. These maps were available only for Rutland, the Burlington and Colchester 7.5-minute quadrangles, and the eastern portion of the St. Johnsbury quadrangle. The surficial maps represent modern, detailed mapping of the Pleistocene glacial deposits and the post-Pleistocene deposits. AECOM was most interested in identifying coarse-grained (i.e., sand and gravel) glaciofluvial deposits from these maps, as these soils are the most likely to be highly water-bearing. Those areas mapped as glaciofluvial soils in direct contact with large surface water features are considered to be most favorable for groundwater development. Lake Champlain, the Winooski River, the Walloomsac River and the Lamoille River are examples of large surface-water features that would enhance well recharge. The main drawback of the surficial geologic mapping is that they offer only clues into groundwater favorability. Surficial mapping, by its nature, represents only the uppermost soils, perhaps only the upper ten feet. An aquifer thickness of at least 30 feet is normally required to support a high-yielding well, i.e., one in the range of several hundred gallons per minute, for municipal use. Without the "thickness component", the surficial geologic maps are somewhat limited in their value in identifying groundwater favorability.

3. Surficial Geology (Vermont Center for Geographic Information) – The VCGI maps cover the entire state, and are most useful for areas that have not been mapped at the quadrangle scale. The mapping originates from the Surficial Geologic Map of Vermont (Doll et al., 1970). These maps identify areas of till and bedrock, as well as other deposits by their origin, (e.g., glacio-lacustrine, glacio-fluvial and glacio-marine), by landform (e.g., kame terrace, delta, and esker) and by grain size (clay, silt, sand, and gravel). From the standpoint of evaluating groundwater favorability, the main drawback of these maps – like the quadrangle maps – is the absence of any reference to thickness of the surficial deposits. Despite this drawback, we can conclude that coarse-grained deposits are most favorable for groundwater development, especially where they are in contact with large surface-water features.

4. Well Yield Maps Compiled by Bedrock Unit (Gale et al., 2009) – These preliminary maps depict mean well yields by bedrock unit, compiled from over 92,000 wells in the state. AECOM used these maps as an indicator of groundwater favorability in fractured-bedrock aquifers. In that regard, however, the mapping is somewhat imperfect. Of the 92,000 water wells compiled, approximately 88,000 are bedrock wells, and the remaining 4,000 are wells in glacial sand-and-gravel aquifers. Therefore, the mean well yields could be slightly biased by a few high yielding wells in sand-and-gravel. Despite this imperfection, AECOM believes the maps do give a broad idea of bedrock-aquifer favorability.

4.6.3 Groundwater Favorability Tables

Table 4-26 summarizes the mapping reviewed and the groundwater favorability for each water system. For the Hodges mapping, AECOM stayed with his favorability terminology, for "excellent potential" and "low to moderate potential". We have assumed that "excellent potential" refers to areas that are capable of yielding several hundred gallons per minute (gpm) or more to a well, and are therefore suitable for municipal purposes. Those areas described by Hodges as having "low to moderate potential" for "domestic, commercial and industrial use", we have assumed are capable of yielding less than 100 gpm to a well, and could perhaps meet the demands of small water systems. Areas mapped by Hodges as "low potential", we have assumed have no potential for municipal use.

From the other mapping, groundwater favorability is described in terms such as "best potential", and "possible groundwater supplies". These terms reflect our general optimism about developing groundwater supplies for municipal use (i.e., several hundred gpm or more) in a particular area, based on the mapped deposits and their proximity to surface-water features. For example, the entire valley along the Winooski River, especially in its floodplain, is likely to contain at least some coarse-grained soils, i.e., sand and gravel. We conclude this not only from available mapping, but also from a limited number of water-well records, and knowledge of glacial-depositional processes. Furthermore, the Winooski River drains over 1,000 square miles of territory; its lowest recent flow (October 5, 2007) was 180 million gallons per day (MGD). So, the Winooski

River represents an enormous recharge source. This is why for the communities in the Champlain Water District, AECOM will frequently use the phrase "best potential…along the Winooski River" on Table 4-26.

Table 4-27 summarizes well yields in fractured bedrock for each of the 15 public water systems, as a preliminary expression of groundwater favorability. This table displays the geographic percent of each community where the average bedrock-well yield is either 15 to 19 gpm, or 20 to 35 gpm. The geographic percentages are approximations from inspection of VGS maps. The average yields apply to specific rock units identified by VGS. For example, approximately 60% of the geographic area of Bennington is underlain by a group of rock units which, on a state-wide basis, yield between 15 and 19 gpm to wells on average. And, 20% of the geographic area of Bennington is underlain by rocks which, on a state-wide basis, yield between and 20 and 35 gpm to wells on average. Though even 35 gpm would not be sufficient for most towns, clearly there are wells in that rock unit that yield more – perhaps substantially more. In this regard, we offer a note of caution about using well yields that are averaged. Bedrock well yields of 5 to 10 gpm are normally considered sufficient for domestic purposes. Therefore, in drilling a domestic well, the well driller will stop drilling once this yield has been achieved, even though a higher yield may be achieved by drilling deeper. So, when wells are compiled and the yields are averaged, the domestic wells tend to lower the average. This approach tends to under-represent the true yield of a fractured bedrock aquifer.

For each water system, we describe the favorability, or "potential", for high-capacity bedrock well development as "low", "moderate", "moderate to high" or "high". The "low" potential category includes those water systems whose percentages on Table 4-28 (15 – 19 gpm, plus 20 – 35 gpm) total less than 30%; the "moderate" category includes those systems whose percentages total 30 to 70%; "moderate to high" potential is for percentages ranging from 70 to 99%; and "high" potential is reserved only for those communities whose percentage total is 100%. These categories are somewhat arbitrary, but they do give a general idea of relative favorability. A detailed well inventory, fracture-trace analysis and bedrock test-well drilling program would be needed to fully evaluate the potential for high-capacity bedrock wells in any given community.

4.6.4 Alburgh Village - Summary of Groundwater Favorability

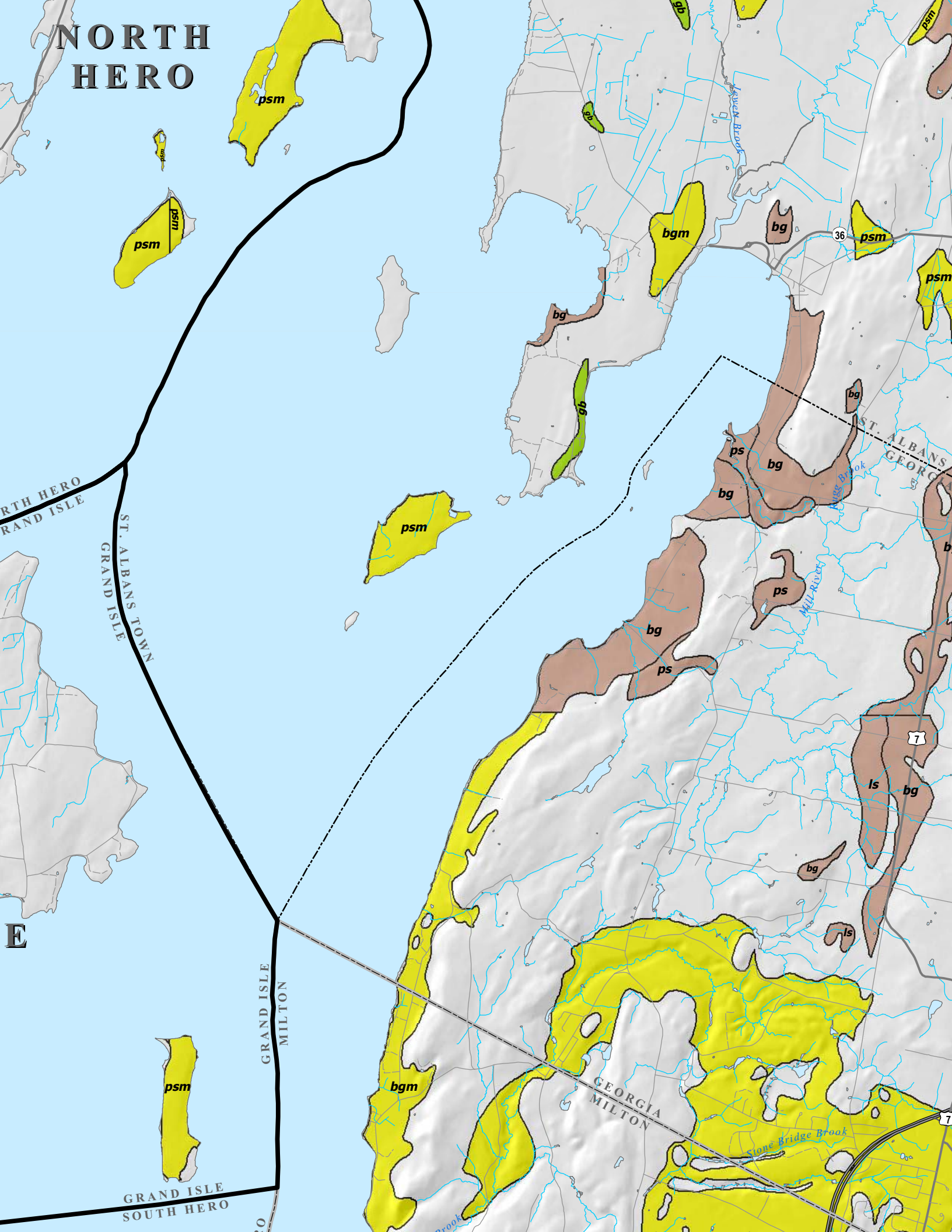
Much of the land area in Alburgh is underlain by soils that are unsuitable for developing municipal groundwater supply (see Figure 4-6). However, small areas of marine sand in contact with Lake Champlain and an esker mapped along Ransoms Bay may have moderate groundwater potential, if these deposits are sufficiently thick, coarse-grained and hydraulically connected to the lake. Despite the meager sand-and-gravel aquifers in Alburgh, they may be sufficient to meet Alburgh's recent maximum day demand of about 0.11 MGD (80 gpm). Test-well drilling would be necessary to more fully evaluate the groundwater potential in sand-and-gravel aquifers in Alburgh. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled.

Based on Table 4-27, Alburgh has low potential for groundwater in fractured bedrock aquifers. It is our understanding that wells in "Alburgh have objectionable water quality.

4.6.5 Bennington - Summary of Groundwater Favorability

While much of Bennington's surface is covered with glacial till and bedrock, significant glaciofluvial deposits are present along the Walloomsac River, Roaring Branch Walloomsac Brook, Paran Creek and a tributary of South Stream (see Figure 4-7). Glaciofluvial deposits are typically composed of water-bearing sand and gravel. If these deposits are sufficiently thick and coarse-grained, they could represent an important groundwater resource, capable of supplementing or replacing the existing surface-water supplies in Bennington. A 121-foot deep well near South Stream reportedly (Hodges, 1966) yields 225 gpm. Test-well drilling would be necessary to more fully evaluate the groundwater potential in sand-and-gravel aquifers in Bennington. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled.

Table 4-28 suggests moderate to high potential for high-capacity bedrock wells in Bennington. Hodges (1966) reported one bedrock well that yields 150 gpm. A well inventory and a detailed fracture-trace analysis would be needed before planning any bedrock well-drilling program. This approach would improve the chances of identifying a suitable bedrock well supply.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Lewen Brook
Mill River
Stone Bridge Brook
36
7
7
ST. ALBANS
GEORGIA
RTH HERO
RAND ISLE
ST. ALBANS TOWN
GRAND ISLE
E
GRAND ISLE
MILTON
GEORGIA
MILTON
GRAND ISLE
SOUTH HERO

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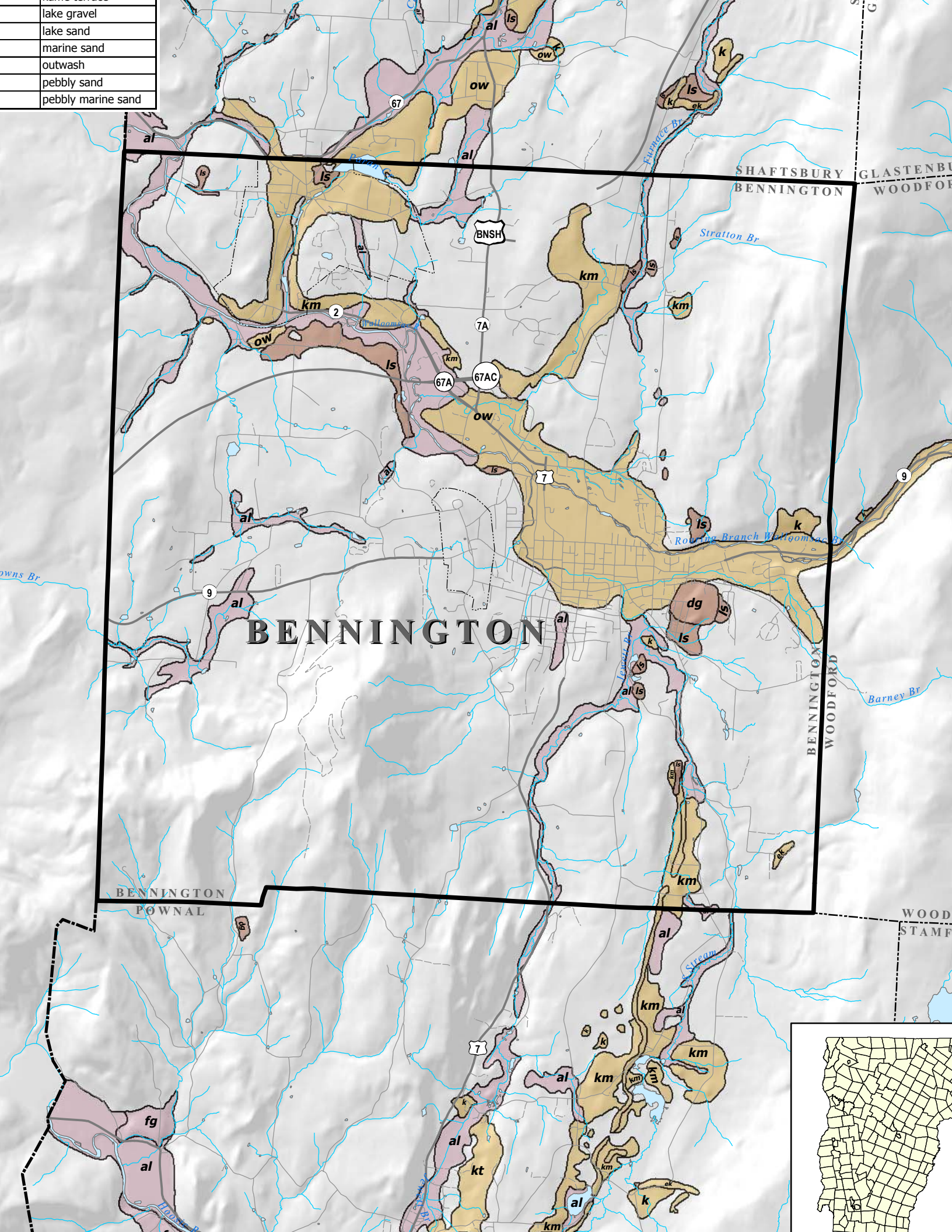
	lake sand
	marine sand
	outwash
	pebbly sand
	pebbly marine sand



4.6.6 Burlington - Summary of Groundwater Favorability

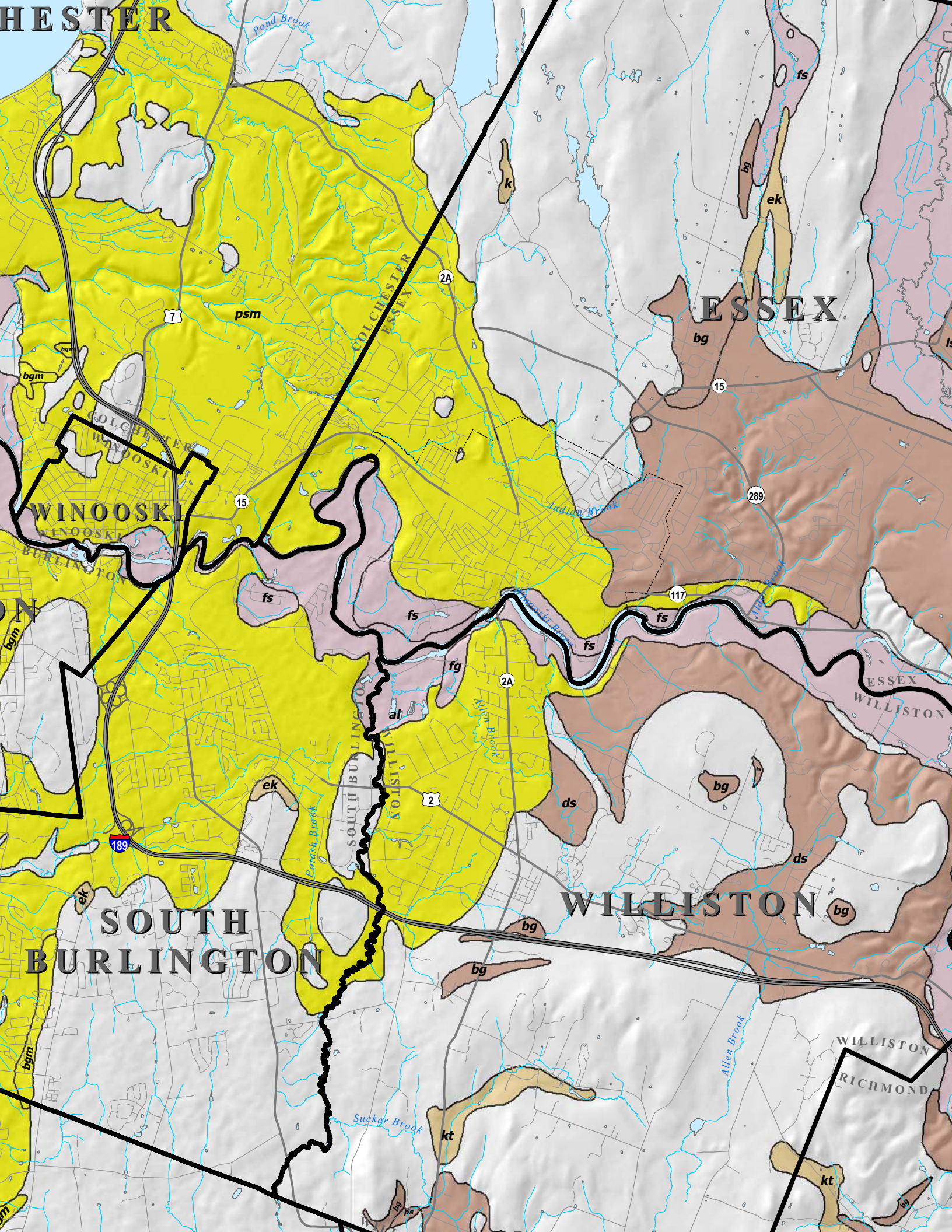
As shown of Figure 4-8, the City of Burlington is underlain largely by sandy soils. The thickness and water-bearing character of the sandy soils is not known. A 31-foot deep well in nearby Winooski reportedly has yielded 450 gpm (Hodges, 1967e), indicating that sandy soils in Burlington might be sufficiently thick and water-bearing for municipal wells. Test-well drilling would be necessary to more fully evaluate the groundwater potential in sand-and-gravel aquifers in Burlington. Based on our evaluation, the floodplain of the Winooski River and the shore of Lake Champlain are the most favorable for groundwater supplies. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled. These activities, as well as mapping of land available to Burlington for municipal wells, would almost surely refine our view of groundwater favorability.

Approximately 80% of Burlington is underlain by rocks that produce 15 to 19 gpm on average, suggesting moderate to high potential for high capacity bedrock wells in Burlington. A well inventory and a detailed fracture-trace analysis would need to be undertaken before planning any well drilling program.

Daily water demand in the City of Burlington has exceeded 5 MGD in the recent past. Because Lake Champlain and the Winooski River represent an enormous potential for groundwater recharge, it is conceivable that groundwater could supplement or fully replace Burlington's existing supply if the soils/bedrock are suitable. Further analysis and testing would be necessary to determine the exact quantity and quality of groundwater in the glacial and bedrock aquifers.

4.6.7 Catamount-Bolton - Summary of Groundwater Favorability

The Catamount-Bolton water system supplies the Bolton Valley Resort and surrounding vacation housing. The water demand of up to .04 MGD is currently being met by surface water from an upland stream. Available mapping indicates that sand-and-gravel aquifers are not present on-site or nearby (see Figure 4-9).


HESTER
Pond Brook
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ESSEX
COLCHESTER
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psm
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COLCHESTER
WINOOSKI
WINOOSKI
WINOOSKI
BURLINGTON
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Indian Brook
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fs
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Allen Brook
ESSEX
WILLISTON
ON
bgm
ek
SOUTH BURLINGTON
WILLISTON
2
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189
Porsch Brook
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ek
WILLISTON
bg
SOUTH
BURLINGTON
bg
bg
bgm
Allen Brook
WILLISTON
RICHMOND
Sucker Brook
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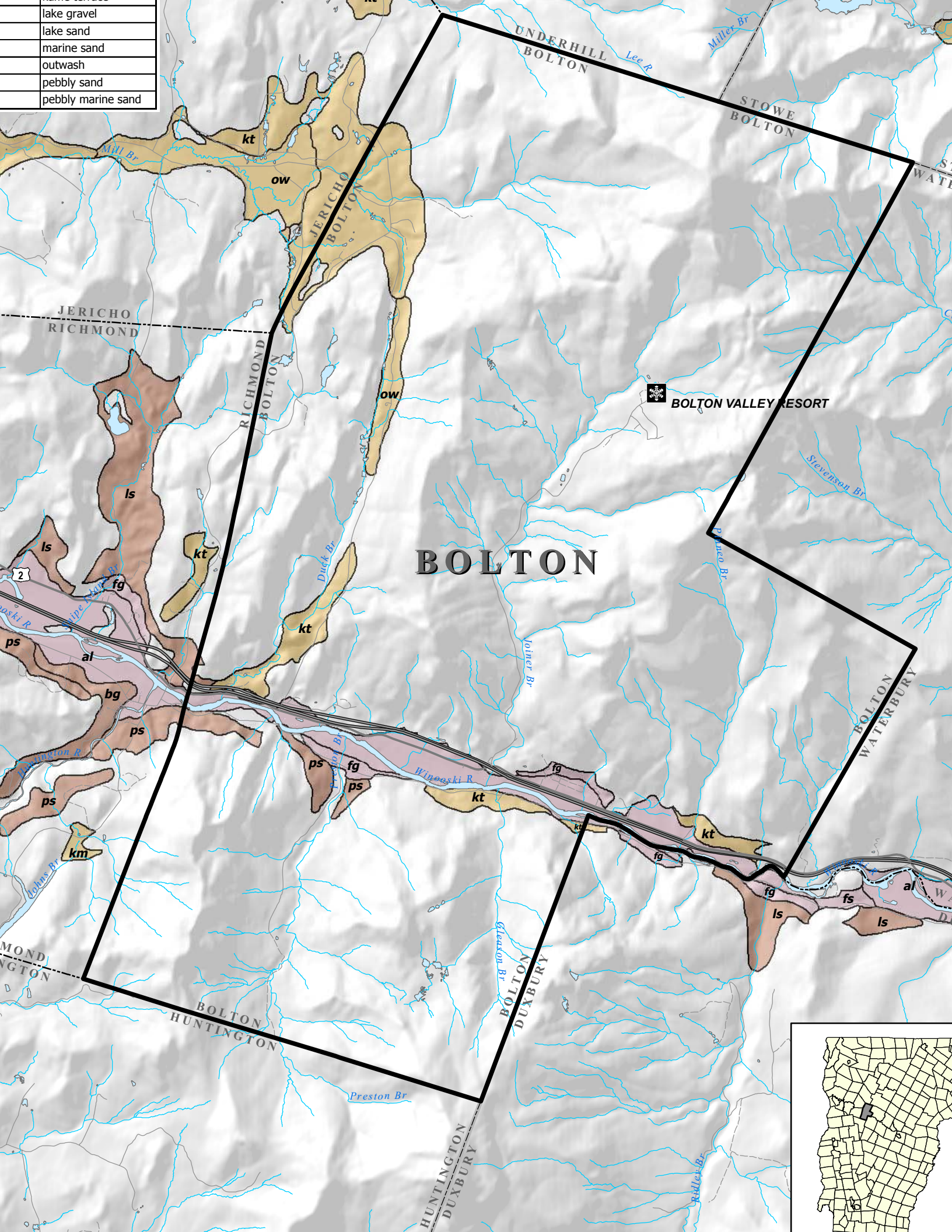
Catamount-Bolton is planning to convert to bedrock well supplies. Table 4-27 suggests that the favorability of groundwater supplies in bedrock in this area is generally low. Catamount-Bolton may find that hydro-fracturing of existing or new bedrock wells will enhance well yield. A well inventory and a detailed fracture-trace analysis would be needed before planning any bedrock well-drilling program. This approach would improve the chances of identifying a suitable bedrock well supply.

4.6.8 Champlain Water District - Summary of Groundwater Favorability

Approximately 40 to 50% of the area in the Champlain Water District (CWD) communities is underlain by sandy or gravelly soils (see Figure 4-8). These soils were deposited predominantly in or along the shores of Glacial Lake Vermont or the Champlain Sea. Glaciofluvial deposits of sand and gravel are much less abundant, and are scattered throughout the CWD communities, particularly in Jericho. Sandy or gravelly alluvium in the floodplain of the Winooski River represents largely re-worked glacial soils that may cover coarse-grained glacial soils suitable for groundwater development. The thickness and water-bearing character of the sandy and gravelly soils throughout the area are not known, which limit our ability to know the groundwater potential. A 31-foot deep well in Winooski reportedly has yielded 450 gpm, indicating that coarse-grained soils elsewhere along the Winooski River might be sufficiently thick and water-bearing for municipal wells. Test-well drilling would be necessary to more fully evaluate the groundwater potential in sand-and-gravel aquifers in the CWD communities. Based on our evaluation, the floodplain of the Winooski River and the shore of Lake Champlain are the most favorable for groundwater supplies. An abbreviated water quality list provided by VTDEC suggests that groundwater treatment for iron and manganese may be required. Two test well programs were historically performed, one in Burlington and the other at Colchester Point. The results of these studies indicated poor water quality and/or low yields. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled. These activities, as well as mapping of land available to CWD for municipal wells, would almost surely refine our view of groundwater favorability.


lake gravel
lake sand
marine sand
outwash
pebbly sand
pebbly marine sand
BOLTON
UNDERHILL
BOLTON
STOWE
BOLTON
JERICHO
BOLTON
JERICHO
RICHMOND
RICHMOND
BOLTON
BOLTON
WATERBURY
BOLTON
DUXBURY
BOLTON
HUNTINGTON
HUNTINGTON
DUXBURY
BOLTON VALLEY RESORT
Mill Br
Lee R
Miller Br
Stevenson Br
Duck Br
Joiner Br
Winooski R
Huntington R
Preston Br
Gleason Br
Johns Br
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The following are our specific comments on groundwater favorability for the individual communities within CWD:

Shelburne – The best potential for municipal groundwater supplies may be in the delta sands at the mouth of the La Platte River (see Figure 4-8). Even if the delta sands are only 30 to 40 feet thick, they could represent a significant groundwater supply.

South Burlington – The floodplain of the Winooski River may represent the best potential for municipal groundwater supplies, though the International Airport may affect groundwater quality. Delta sands along the lake shore may also hold promise, if they are sufficiently thick. Two esker deposits south of Potash Brook may also be suitable, provided the areas are not too urbanized.

Williston – The pebbly sands and delta sands mapped south of the Winooski River may extend beneath the Winooski River alluvium. These soils would be good sources of groundwater, provided they are sufficiently thick and coarse-grained.

Winooski – The floodplain of the Winooski River may contain sufficiently thick and coarse-grained soils suitable for municipal groundwater development. Urbanization in Winooski may impair groundwater quality.

Essex – The floodplain of the Winooski River may contain sufficiently thick and coarse-grained soils suitable for municipal groundwater development. Urbanization in this part of Essex may impair groundwater quality. Esker deposits along Alder Brook southeast of Pages Corner may represent a good source of groundwater, if they are sufficiently thick and coarse-grained. On the positive side, these esker deposits are somewhat remote from human activity, though the relatively small drainage area may limit well yield.

Colchester – About half of the land area in Colchester is mapped as pebbly marine sands, which could be suitable for municipal groundwater supplies if the sands are sufficiently thick and coarse-grained. Pebbly marine sands found along the floodplain of the Winooski River and the shore of Lake Champlain would have the best potential for high capacity wells because of the vast source of recharge.

Milton – About half of the land area in Milton is covered with pebbly marine sands, which could be suitable for municipal groundwater supplies if the sands are sufficiently thick and coarse-grained. The best groundwater potential is the pebbly sands along the Lamoille River, which represents are a large source of recharge. CWD owns one piece of property in Milton which is provided in Figure 4-7a. Note that the size of the lot would make it difficult in obtaining a 200 ft. radius around the well.

Jericho – About half of the land area in Jericho contains sand-and-gravel deposits of glacial origin, which could be suitable for municipal groundwater supplies if these deposits are sufficiently thick and coarse-grained. Despite recent residential development, Jericho remains relatively rural, which normally means better quality groundwater than urban communities. The best potential for groundwater is along the Winooski River, but also along Browns River, Lee River, and Mill Brook.

Table 4-27 suggests a range of favorability – from "low" to "high" – for high-capacity bedrock supplies in the CWD communities. A well inventory and a detailed fracture-trace analysis would need to be undertaken before planning any bedrock well-drilling program. This approach would improve the chances of identifying suitable bedrock well supplies.

Daily water demand in the CWD has exceeded 10 MGD in the recent past. Because Lake Champlain and the Winooski River represent an enormous potential for groundwater recharge, it is conceivable that groundwater could supplement CWD's existing supply, if the soil/bedrock conditions are suitable. Further detailed analysis and testing would be necessary to determine the exact quantity and quality of groundwater available in the glacial and bedrock aquifers.

Groundwater sources scattered throughout the CWD communities would benefit CWD in several ways. First, it could reduce water age, which appears to be CWD's main source of DBP problems. A well or wells in Jericho, for example, could supply Jericho and parts of Essex with relatively young water. Second, multiple groundwater sources would create greater reliability and flexibility in the water system. Overall cost of well operation compared to surface water will depend on the water quality found in the well.

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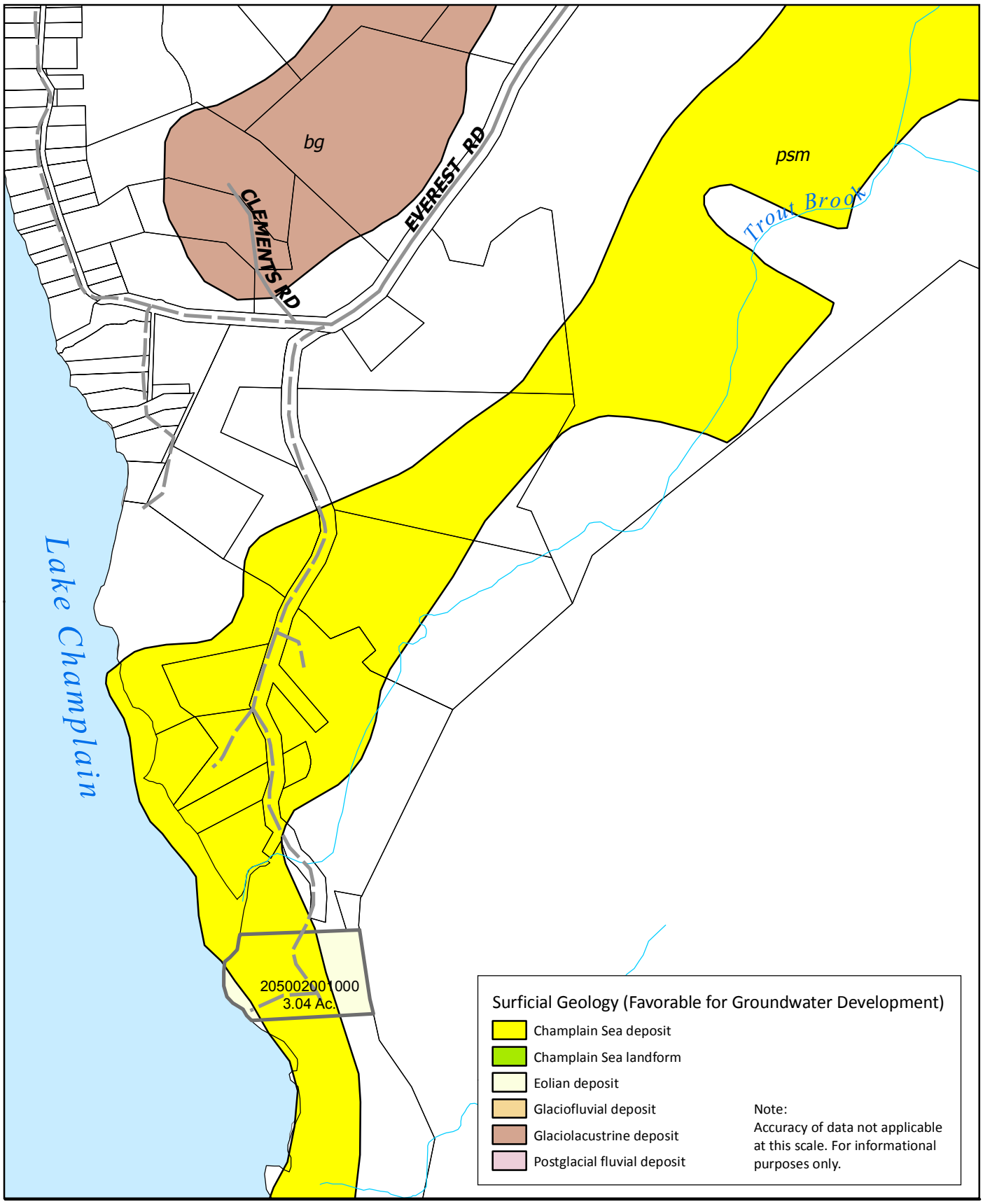

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Figure 4-7A
West Milton Parcel

4.6.9 Grand Isle - Summary of Groundwater Favorability

As indicated on Figure 4-6, only very small areas of Grand Isle are underlain by sandy soils. Isolated bodies of marine sand and gravel may have minor groundwater potential, but we suspect well yields would not be sustainable and not sufficient to meet Grand Isle's maximum day demand of 0.18 MGD.

Table 4-27 indicates generally low favorability for bedrock well supplies in Grand Isle, despite the fact that Hodges (1967a) reports one bedrock well that yields 100 gpm. It is our understanding that well supplies in the islands contain objectionable quality water.

4.6.10 North Hero - Summary of Groundwater Favorability

As indicated on Figure 4-6, only one small body of sandy soils is present in North Hero. The remaining soils are not suitable for municipal groundwater development. From this, we conclude that there is no potential for municipal groundwater supplies from sand-and-gravel aquifers sufficient to meet North Hero's maximum day demand of 0.18 MGD.

Table 4-27 suggests low potential for groundwater supplies in the bedrock.

4.6.11 Proctor - Summary of Groundwater Favorability

A relatively large area of glacially derived sand and gravel is mapped on the east side of Otter Creek in Proctor (see Figure 4-10). This area represents the best potential for groundwater supply, especially because of the potential for significant recharge from Otter Creek. Hodges (1967d) reported a 130-foot deep well in Proctor that yields 280 gpm. It is our understanding that Proctor is considering converting from its surface-water supplies to groundwater. We suspect that one or more groundwater sources could meet the demands of Proctor, which recently were as high as 0.41 MGD (290 gpm). A carefully designed test-well investigation would be necessary to determine well yield and water quality. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled.

Table 4-27 suggests that the bedrock in Proctor has moderate potential for municipal well supplies.

4.6.12 Readsboro Village - Summary of Groundwater Favorability

The land in Readsboro is almost exclusively underlain by glacial till and bedrock exposures unsuitable for municipal groundwater development. As shown on Figure 4-11, several bodies of glaciofluvial deposits exist, one of which is along the Deerfield River downstream of Readsboro Village. Test-drilling here would indicate whether this deposit is sufficiently thick and coarse-grained to support a municipal supply of about 0.037 MGD, which is Readsboro's recent maximum day demand.

Table 4-27 indicates that the bedrock in Readsboro is generally capable of supporting low well yields, though Hodges (1968) reported one 100-gpm bedrock well. A well inventory and fracture-trace analysis would be necessary to more fully evaluate bedrock well potential.

4.6.13 Richford - Summary of Groundwater Favorability

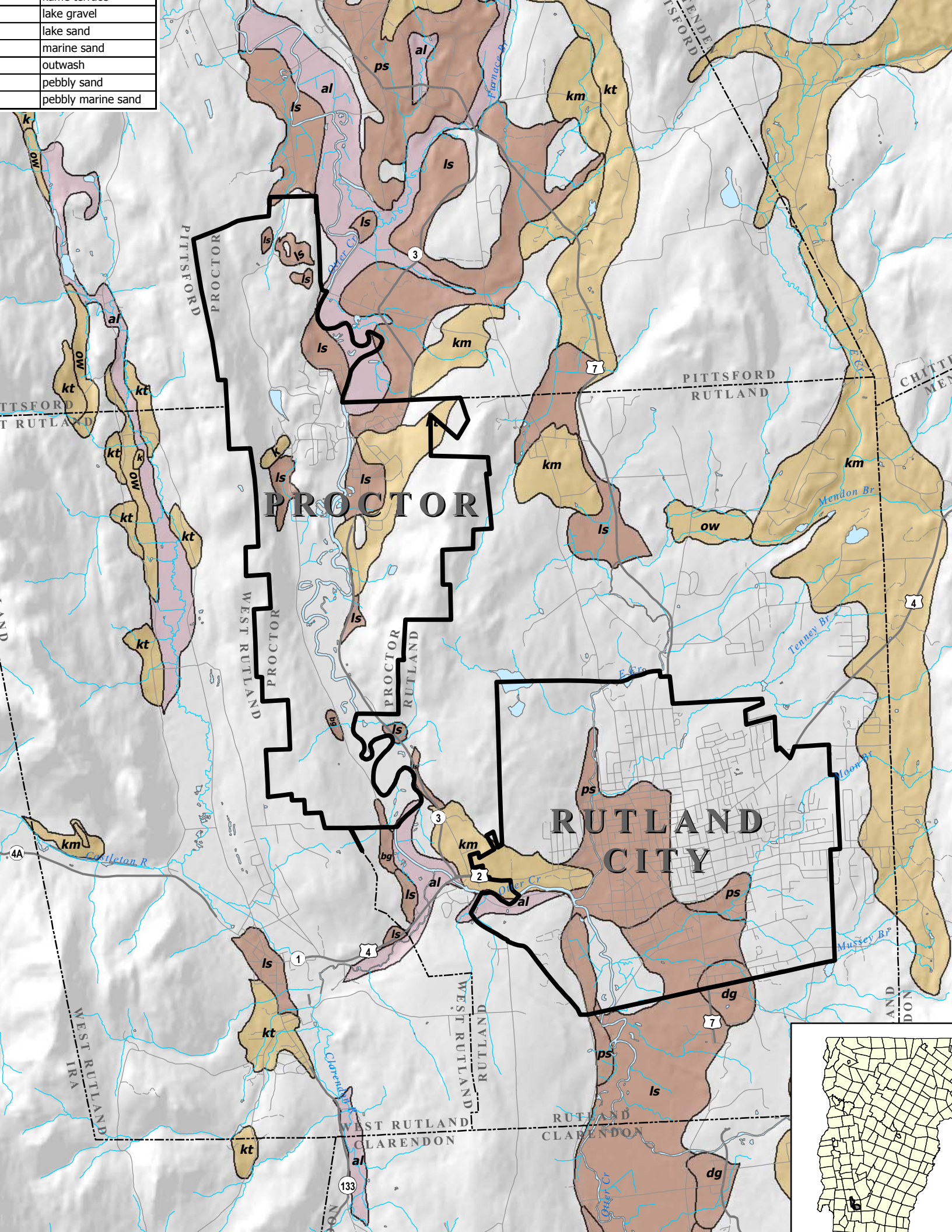
As shown on Figure 4-12, the lowlands along the Missisquoi and North Branch Missisquoi Rivers in Richford are occupied by glaciofluvial deposits, glacial lake pebbly sands, alluvium and delta gravels. These represent the most favorable areas for municipal groundwater development, especially because they rest along the rivers, which represent sources of recharge. Test-drilling here would indicate whether these deposits are sufficiently thick and coarse-grained to support a municipal supply of about 0.16 MGD (about 110 gpm), which is Richford's recent maximum day demand.

Table 4-27 suggests low potential for municipal groundwater supplies in the bedrock.

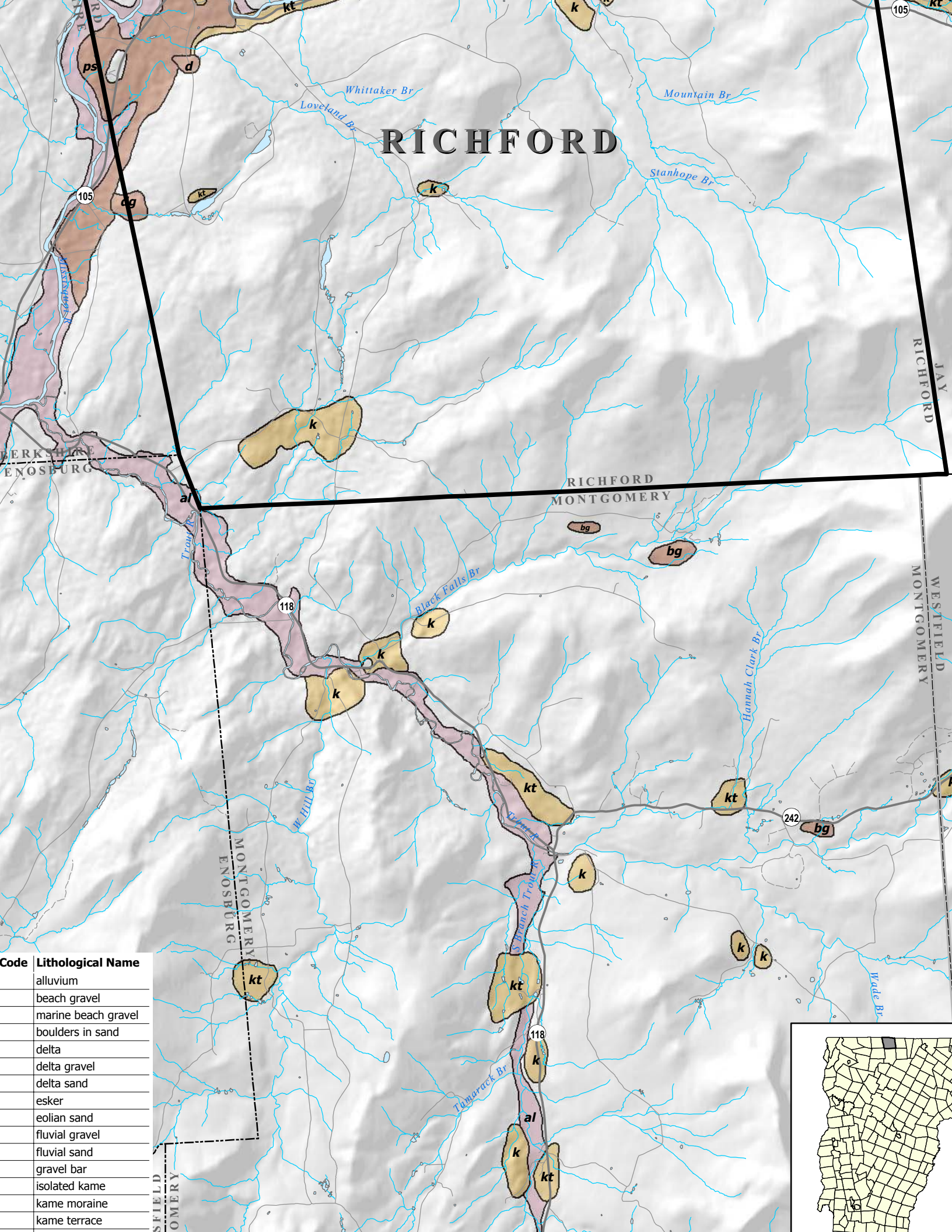
4.6.14 Rutland City - Summary of Groundwater Favorability

As shown on Figure 4-10, relatively large areas of Rutland are underlain by glaciofluvial deposits, pebbly sands, lake sands, and delta gravels favorable for groundwater development, especially along Otter Creek and on the eastern border of town. Detailed surficial geologic mapping by Van Hoesen (2009) suggests that the area near the confluence of East Creek and Mendon Brook in the northeastern side of town might be the most suitable for municipal groundwater supplies. Test-drilling here would indicate

	lake gravel
	lake sand
	marine sand
	outwash
	pebbly sand
	pebbly marine sand



	lake gravel
	lake sand
	marine sand
	outwash
	pebbly sand
	pebbly marine sand



Code	Lithological Name
	alluvium
	beach gravel
	marine beach gravel
	boulders in sand
	delta
	delta gravel
	delta sand
	esker
	eolian sand
	fluvial gravel
	fluvial sand
	gravel bar
	isolated kame
	kame moraine
	kame terrace

whether these deposits are sufficiently thick and coarse-grained to support municipal well development. Test well drilling would also have to be conducted along Otter Creek west and south of town to confirm the suitability of these areas. In recent years, Rutland's maximum day demand has exceeded 7 MGD. Average daily demand is closer to 2.5 MGD. For such a system, it is likely that wells would meet average daily demands and/or part of peak demands.

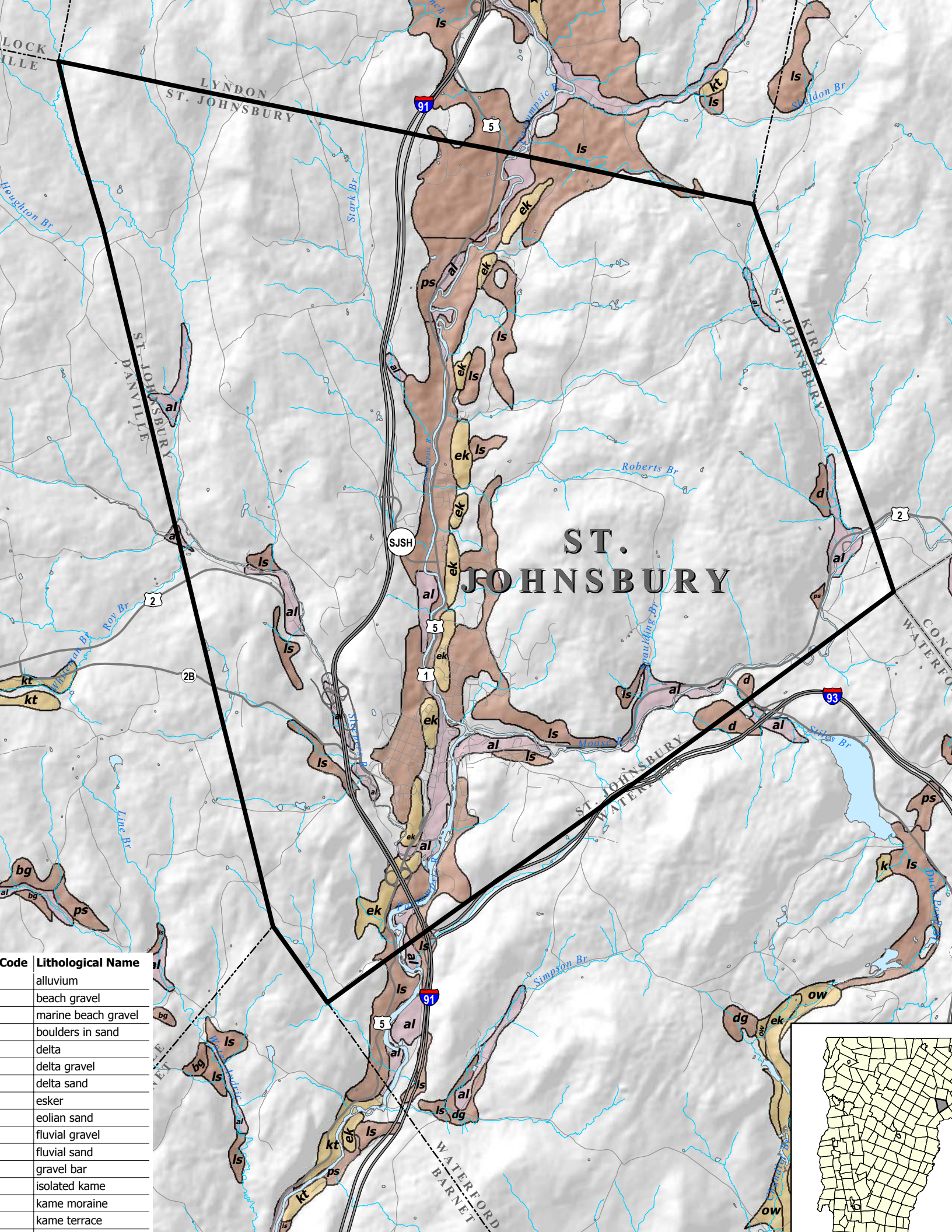
Table 4-27 suggests that the bedrock is highly favorable for development of municipal groundwater supplies. A well inventory and fracture-trace analysis would be necessary to more fully evaluate bedrock well potential.

4.6.15 St. Johnsbury - Summary of Groundwater Favorability

As shown on Figure 4-13, pebbly sands, lake sands, eskers and alluvium occupy the entire length of the lowlands along the Passumpsic River in St. Johnsbury, all of which are favorable for groundwater development. Normally, eskers are the most favorable because they typically contain highly transmissive deposits of gravel, sand, cobbles and even boulders. Ten esker segments are mapped from north to south in the valley. The Passumpsic River would act as a reliable source of recharge to the esker deposits.

Hodges (1967c) reports a 70-foot deep, 800-gpm well in St. Johnsbury developed by the Town. It is conceivable, therefore, that all or part of the Town's maximum day demand of slightly over 2 MGD could be met by groundwater sources. Test-well drilling would be necessary to more fully evaluate the groundwater potential in sand-and-gravel aquifers. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled.

Table 4-27 suggests that the bedrock in St. Johnsbury is moderately favorable for development of municipal groundwater supplies. A well inventory and fracture-trace analysis would be necessary to more fully evaluate bedrock well potential.



Code	Lithological Name
	alluvium
	beach gravel
	marine beach gravel
	boulders in sand
	delta
	delta gravel
	delta sand
	esker
	eolian sand
	fluvial gravel
	fluvial sand
	gravel bar
	isolated kame
	kame moraine
	kame terrace

4.6.16 Swanton - Summary of Groundwater Favorability

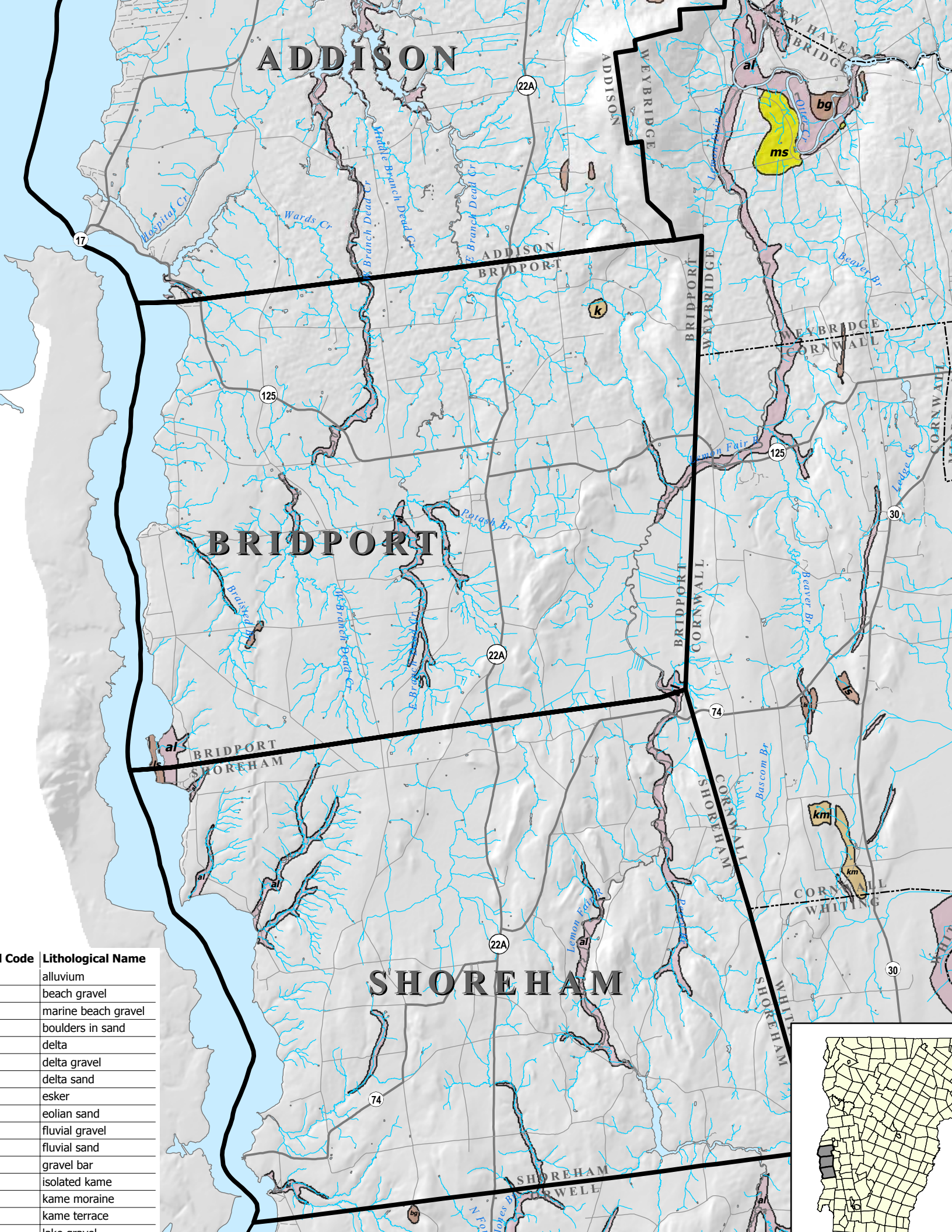
About one-quarter of the land surface in Swanton is underlain by sandy or gravelly soils. A large area of pebbly marine sands near the bend in the Lamoille River (see Figure 4-6) is the most favorable for groundwater development, provided the sands are adequately thick and coarse-grained. If these conditions are met, the Lamoille River would represent a large and dependable source of recharge, which could support groundwater withdrawals of around 0.7 MGD (Swanton's maximum day demand). Test-well drilling would be necessary to more fully evaluate the groundwater potential in the pebbly marine sands. A well inventory and an analysis of land use and potential sources of groundwater contamination should be undertaken before test wells are drilled. Isolated bodies of beach gravels and glaciofluvial deposits may also have groundwater potential.

Table 4-28 suggests that the bedrock in Swanton is moderately favorable for development of municipal groundwater supplies. A well inventory and fracture-trace analysis would be necessary to more fully evaluate bedrock well potential.

4.6.17 Tri-town Water District - Summary of Groundwater Favorability

The land in the Tri-Town Water District is almost exclusively underlain by glacial-lake silt and clay, glacial till, and bedrock exposures, which are unsuitable for municipal groundwater development. A few, small, isolated bodies of sand and gravel are scattered throughout the region (see Figure 4-14), only one of which has promise for developing a groundwater supply. We are referring to a body of glaciolacustrine sand on the shore of Lake Champlain along the border between Bridport and Shoreham. Test-well drilling would be necessary to confirm whether this deposit is sufficiently thick and coarse-grained to support a municipal well capable of meeting Tri-Town's maximum day demand of 0.82 MGD.

Based on Table 4-27, the bedrock in the Tri-Town Water District has low to moderate potential for groundwater supplies.



l Code	Lithological Name
	alluvium
	beach gravel
	marine beach gravel
	boulders in sand
	delta
	delta gravel
	delta sand
	esker
	eolian sand
	fluvial gravel
	fluvial sand
	gravel bar
	isolated kame
	kame moraine
	kame terrace
	lake gravel

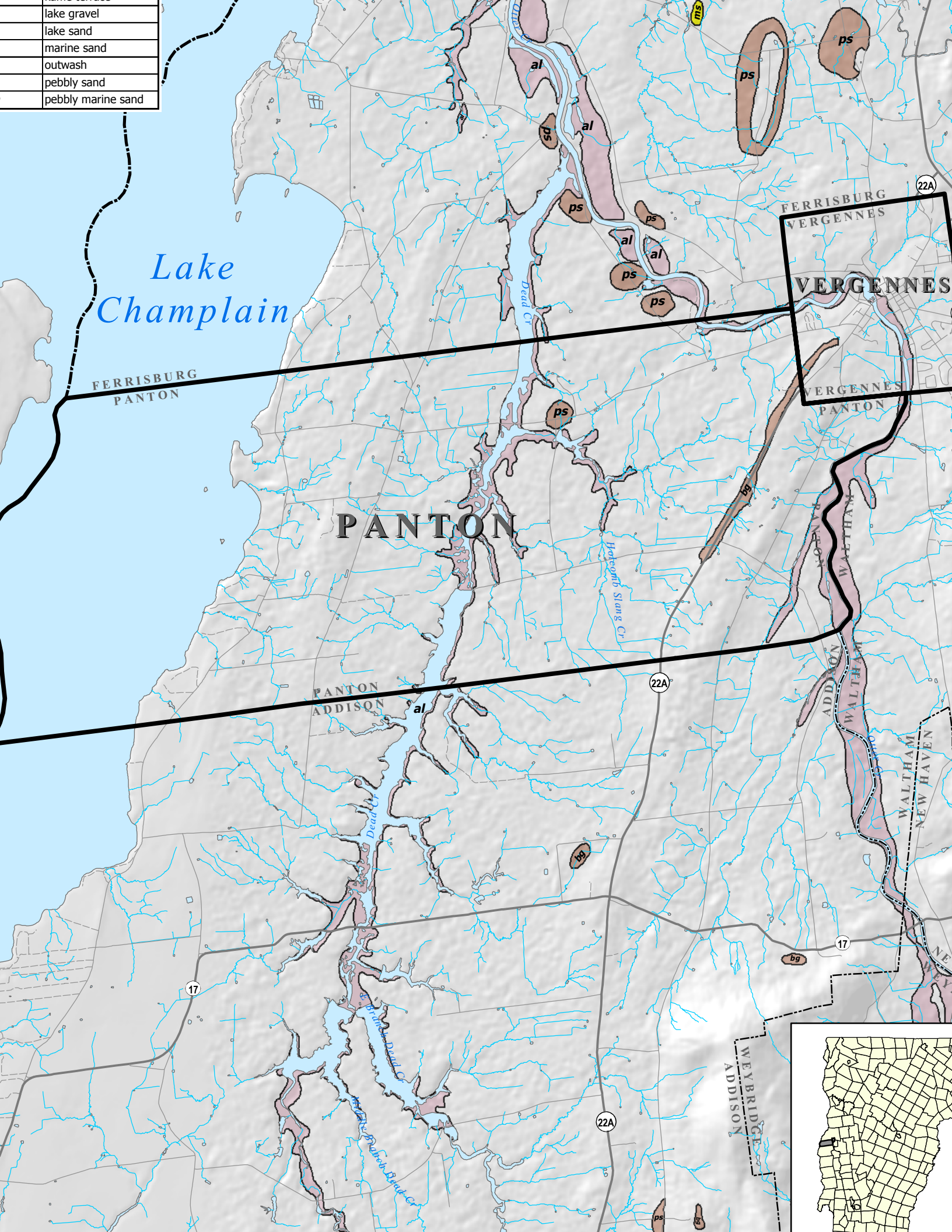
4.6.18 Vergennes- Panton - Summary of Groundwater Favorability

The land in the towns of Vergennes and Panton is almost exclusively underlain by glacial-lake silt and clay, glacial till, and bedrock exposures, which are unsuitable for municipal groundwater development. It is unlikely that the isolated bodies of sand, gravel and alluvium (see Figure 4-15) have sufficient extent or grain-size to support a maximum day demand of 0.75 MGD.

Based on Table 4-27, the bedrock in the Vergennes-Panton area has low to moderate potential for groundwater supplies.

4.6.19 Summary

In summary, AECOM believes that wells are a viable option for select communities in obtaining a source water with lower levels or less reactive TOC with chlorine. In addition, wells are likely to require less disinfection than surface waters and hence wells have a propensity to use less chlorine. In addition, wells can be located within large distribution systems to reduce overall water age. In Section 5, AECOM provides well cost estimates for Proctor, Readsboro and Rutland City.


lake gravel
lake sand
marine sand
outwash
pebbly sand
pebbly marine sand
Lake Champlain
FERRISBURG
PANTON
PANTON
FERRISBURG
VERGENNES
VERGENNES
VERGENNES
PANTON
PANTON
ADDISON
WALTHAM
ADDISON
WALTHAM
NEW HAVEN
WEYBRIDGE
ADDISON
Dead Cr
Hotcomb Slang Cr
Little Dead Cr
Little Dutch Dead Cr
22A
17
al
ps
bg
ms

5.0 - SYSTEM SUMMARIES

5.1 DATA COLLECTION AND MANAGEMENT

AECOM followed the process as outlined in Table 2-2 in seeking input for each utility and DEC. Following the kickoff meeting, data was solicited from each individual utility via electric mail using a survey form. The form requested utilities to provide a minimum of two seasons of water quality data as well as operation data (i.e. flow, chemical dosing etc.) for the WTF. Generally utilities provided data for the late summer and mid winter. In some cases, utilities provided a third season, commonly a fall season dataset. In addition, the DEC provided operational data sheets consisting of chemical dosing and water quality data reported from each WTF as well as distribution water quality data. Following the electronic mail survey, AECOM met with each utility (except Richford) and verified the information provided as well as soliciting missing pieces of information.

The survey results can be found in tabular and graphical format in Appendices 3 through 5. A summary of each utility's operational scheme and distribution characteristics is provided in a table for each utility in Section 5.2: Treatment Technologies Selection.

5.2 TREATMENT TECHNOLOGY SELECTION

5.2.1 Modeling Existing System Performance and Recommended Alternative

The existing water system was modeled using US EPA's Water Treatment Plant Model (model) V. 2.1. The intention of performing this analysis was to obtain a baseline of performance for each system and compare the expected performance in DBP precursor removal, DBP reduction, and where applicable areas of possible improvements. In addition to obtaining a baseline, the model may provide an indication to a poorly performing existing facility. The model was run based on the average water quality provided by each system and where available, was taken from the synthetic water quality database as discussed below. The model was calibrated to the typical winter and summer DBP levels.

The results of the model runs are provided in Appendix 2 - Computer Model Output Summaries. For each community the historical DBP RAA and reduced DBP RAA with the proposed treatment in operation during the peak DBP months are provided in figure format in this section. The location RAA doesn't exist at this time and can not be shown.

Based on data provided by the DEC and each utility, AECOM developed DBP reduction alternatives using chlorine or chloramines as the secondary disinfection. The alternatives that were considered for implementing chlorine and chloramine approaches to disinfection, are listed below.

Chlorine:

1. Stop or minimize pre-chlorination prior to filtration
2. Optimize the coagulant dose to remove as much TOC as possible prior to adding chlorine
3. Improve clarification prior to filtration to allow for more flexibility in coagulant dosing
4. Add additional treatment processes, to specifically remove TOC:
    a. ion exchange,
    b. granular activated carbon adsorption, or
    c. nanofiltration
5. Switch to sources waters (groundwater wells) with lower TOC levels or TOC that is less reactive with chlorine

Chloramine:

1. Use free chlorine for primary disinfection and use chloramine as a secondary disinfectant
2. Use UV for primary disinfection and free chlorine only for primary disinfection of viruses, and use chloramine as secondary disinfectant
3. Use UV and chloramine for primary disinfection, and use chloramine as secondary disinfectant

Costs were developed based on optimizing the system up to the first major unit treatment process installation. For chlorine, this included steps 1 to 4, or 5. For chloramines, costs were developed for steps 1, 2, and 3 as individual options.

However, it should be noted the modeling with US EPA's WTP software was performed mutually exclusive of each recommendation. For example, when GAC was modeled, it did not include the enhanced reduction due to optimized coagulation (i.e. the existing coagulant dose at the WTF was used to during the GAC model run). Thus, the results presented here will be conservative.

5.2.1.1 Synthetic Water Quality for Model Runs

Since the necessary data to forecast water treatment plant performance was not recorded at all facilities, a synthetic water quality database was required. As needed, water quality parameters were pulled from the dataset to make the raw dataset full for the model run. Three main water sources are found in the 15 community dataset:

1. Lake Champlain
2. Open reservoir fed by a upland brook or stream
3. Low alkalinity, upland mountain stream

AECOM relied on the following sources of information in developing the dataset:

1. Input from the Vermont Department of Environmental Conservation
2. Data from participating utilities using similar water sources
3. Lake Champlain Long-Term Water Quality and Biological Monitoring Project
4. AECOM experience in New England

Based on the above information sources, AECOM developed the synthetic water quality shown in Table 5-1. In addition, it is assumed that each facility is required to obtain 3.0 log inactivation/removal of Cryptosporidium, 3.0 log inactivation/removal of Giardia, and 4.0 log of virus inactivation/removal per the SWTR. The required disinfection inactivation values (i.e. CT as defined as residual disinfectant concentration multiplied by contact time) are provided in Table 5-2 through Table 5-4. Conventional systems are anticipated to be credited a 2.5 log removal of Giardia and 1.0 log removal of viruses. Alternative systems are anticipated to receive 2.0 log removal of Giardia and 1.0 log removal of viruses. All systems meeting a filtered turbidity of 0.3 NTU 95% of the time (1.0 NTU for slow sand) receive 3.0 log removal of Cryptosporidium. Table 5-5 provides the credited performance value through each system implemented with the study group of 15 communities based on the information above.

TABLE 5-1
SYNTHETIC RAW WATER QUALITY

Lake Champlain Source		
Parameter	Average Value	Units
pH	7.7	Standard Unit
Temperature	10	C
Total Organic Carbon	4	mg C/L
UV254	0.150	1/cm
Bromide	0.050	mg Br/L
Alkalinity	45	mg/L as $CaCO_3$
Calcium Hardness	40	mg/L as $CaCO_3$
Total Hardness	57	mg/L as $CaCO_3$
Ammonia	0.08	mg N/L
Turbidity	2	NTU
Upland Stream Source		
Parameter	Average Value	Units
pH	6.1	Standard Unit
Temperature	10	C
Total Organic Carbon	2.5	mg C/L
UV254	0.060	1/cm
Bromide	0.050	mg Br/L
Alkalinity	17	mg/L as $CaCO_3$
Calcium Hardness	25	mg/L as $CaCO_3$
Total Hardness	37	mg/L as $CaCO_3$
Ammonia	0.05	mg N/L
Turbidity	1	NTU
Open Reservoir Source		
Parameter	Average Value	Units
pH	6.5	Standard Unit
Temperature	10	C
Total Organic Carbon	3	mg C/L
UV254	0.15	1/cm
Bromide	0.080	mg Br/L
Alkalinity	35	mg/L as $CaCO_3$
Calcium Hardness	40	mg/L as $CaCO_3$
Total Hardness	50	mg/L as $CaCO_3$
Ammonia	0.08	mg N/L
Turbidity	1	NTU

TABLE 5-2
REQUIRED CT VALUES FOR GIARDIA INACTIVATION AT TEMPERATURE OF 0.5 C FOR FREE CHLORINE

	pH 6.5 Log Inactivation		pH 7.0 Log Inactivation		pH 7.5 Log Inactivation		pH 8.0 Log Inactivation		pH 8.5 Log Inactivation	
Free Cl2, mg/L	0.5	1.0	0.5	1.0	0.5	1.0	0.5	1.0	0.5	1.0
0.4	27	54	33	65	40	79	46	92	55	110
0.6	28	56	33	67	40	80	48	95	57	114
0.8	29	57	34	68	41	82	49	98	59	118
1.0	29	59	35	70	42	84	51	101	61	122
1.2	30	60	36	72	43	86	52	104	63	125
1.4	31	61	37	74	44	89	54	107	65	129
1.6	32	63	38	75	46	91	55	110	66	132

Source US EPA, 1991

TABLE 5-3
REQUIRED CT VALUES GIARDIA INACTIVATION AT A TEMPERATURE OF 10 C FOR FREE CHLORINE

	pH 6.5 Log Inactivation		pH 7.0 Log Inactivation		pH 7.5 Log Inactivation		pH 8.0 Log Inactivation		pH 8.5 Log Inactivation	
Free Cl2, mg/L	0.5	1.0	0.5	1.0	0.5	1.0	0.5	1.0	0.5	1.0
0.4	15	29	17	35	21	42	25	50	30	59
0.6	15	30	18	36	21	43	26	51	31	61
0.8	15	31	18	37	22	44	26	53	32	63
1.0	16	31	19	37	22	45	27	54	33	65
1.2	16	32	19	38	23	46	28	55	33	67
1.4	16	33	19	39	23	47	28	57	34	69
1.6	17	33	20	40	24	48	29	58	35	70

Source US EPA, 1991

TABLE 5-4
VIRUS INACTIVATION REQUIREMENTS AT pH 6-9 FOR FREE CHLORINE

Temperature, C	Log Inactivation 2.0	Log Inactivation 3.0
0.5	6	9
5.0	4	6
10	3	4

Source US EPA, 1991

TABLE 5-5
CREDITED AND REQUIRED INACTIVATION VALUES PER TREATMENT SYSTEM

Unit Process	SWTR Required Inactivation		Credited Inactivation via Filtration		Remaining Inactivation Required	
	Giardia	Virus	Giardia	Virus	Giardia	Virus
Aquarius Filter Units	3.0	4.0	2.0	1.0	1.0	3.0
Waterboy Filter Units	3.0	4.0	2.0	1.0	1.0	3.0
Buoyant Media Clarification followed by Filtration	3.0	4.0	2.0*	1.0	1.0*	3.0
Slow Sand Filtration	3.0	4.0	2.0	2.0	1.0	2.0
Direct Filtration	3.0	4.0	2.0	1.0	1.0	3.0

Source US EPA, 1991 and VT DEC
* Champlain Water District has received 2.5 log inactivation credit for Giardia due to a high performing filter system and thus is required to achieve 0.5 log Giardia inactivation instead of 1.0 log Giardia inactivation.

5.2.2 Simultaneous Compliance with Other Regulatory Rules

Simultaneous Compliance issues with several of the regulatory rules are discussed in more detail under Section 3.2.7: Simultaneous Compliance Issues. A brief summary is covered here as an overview for Section 5.

Lead and Copper Rule

Since many existing and proposed systems will likely be dosing more coagulant which will likely be acidic, the pH of the water will drop from the natural pH found in the source water. Hence, the operator will be required to diligently raise the pH after the disinfection contact basin back to the pH implemented for each system's corrosion control strategy. If the coagulant chemical is changed or an ion exchange resin implemented such that there is significantly more chloride in the finished water, some systems may experience an increase in corrosion rates as higher chloride to sulfate ratios tend to favor increased localized corrosion (see section 4.5.3: Effect of Treatment and Disinfection on Distribution System Water Quality).

Systems which switch to chloramine are at risk of dissolving lead protecting pipe scales if there is a significant change in the oxidation-reduction potential (ORP) of the finished water. This is particularly true of highly oxidized lead scales as typical found with high chlorine residuals (see section 4.5.3: Effect of Treatment and Disinfection on Distribution

System Water Quality). Several participants sent copper service pipe to USEPA for scale analysis. No other service pipe material was submitted for analysis. The analysis is provided in Appendix 9 - Service Scale Analysis. This analysis showed that the copper pipe scales were mainly either composed of Cu (I) or Cu (II). These scales can be redox sensitive. While it is unlikely that the copper MCL will be exceeded during phase changes between Cu (I) and Cu (II), there can be elevated levels of other metals which can be released such as manganese and lead. These released metals can cause temporary taste and odors or may exceed SMCL or MCL valves.

Lastly, for those systems with nanofiltration as a toolbox item, consideration should be given for a pilot study on effects on corrosion control as this would be considered a significant change in source treatment. Systems implementing nanofiltration may require a remineralization step to add hardness and/or alkalinity in the finished water for corrosion stabilization.

Total Coliform Rule

As the system looks to reduce the chlorine which is implemented in the system, biological monitoring needs to be increased to provide a warning when biological activity is increasing. AECOM recommends the monitoring provided in Section 4.5.4 "Strategies to Improve Water Quality in Distribution Systems" be developed and followed. Once biological activity is increasing, action steps should be followed to reduce the risk of pathogen contamination.

Surface Water Treatment Rule and Amendments

Since many existing and proposed systems will likely be dosing more coagulant, the solids loading on clarification and filtration systems will be increased. More operator attention will be directed at maintaining filter turbidities below the regulatory limit of 0.3 NTU and below the Partnership for Safe Drinking Water goal of 0.10 NTU by maintaining proper coagulation chemistry as well as filter run times.

National Pollutant Discharge Elimination System Permits

Several systems discharge their backwash to settling basins with the clarified water being discharged back to a surface water source. Other residual receiving sources are local publicly owned treatment works (POTW). High coagulant dosing will require a review of the waste handling systems to ensure compatibility with receiving waters.

5.2.3 System Summary and Technology Recommendation

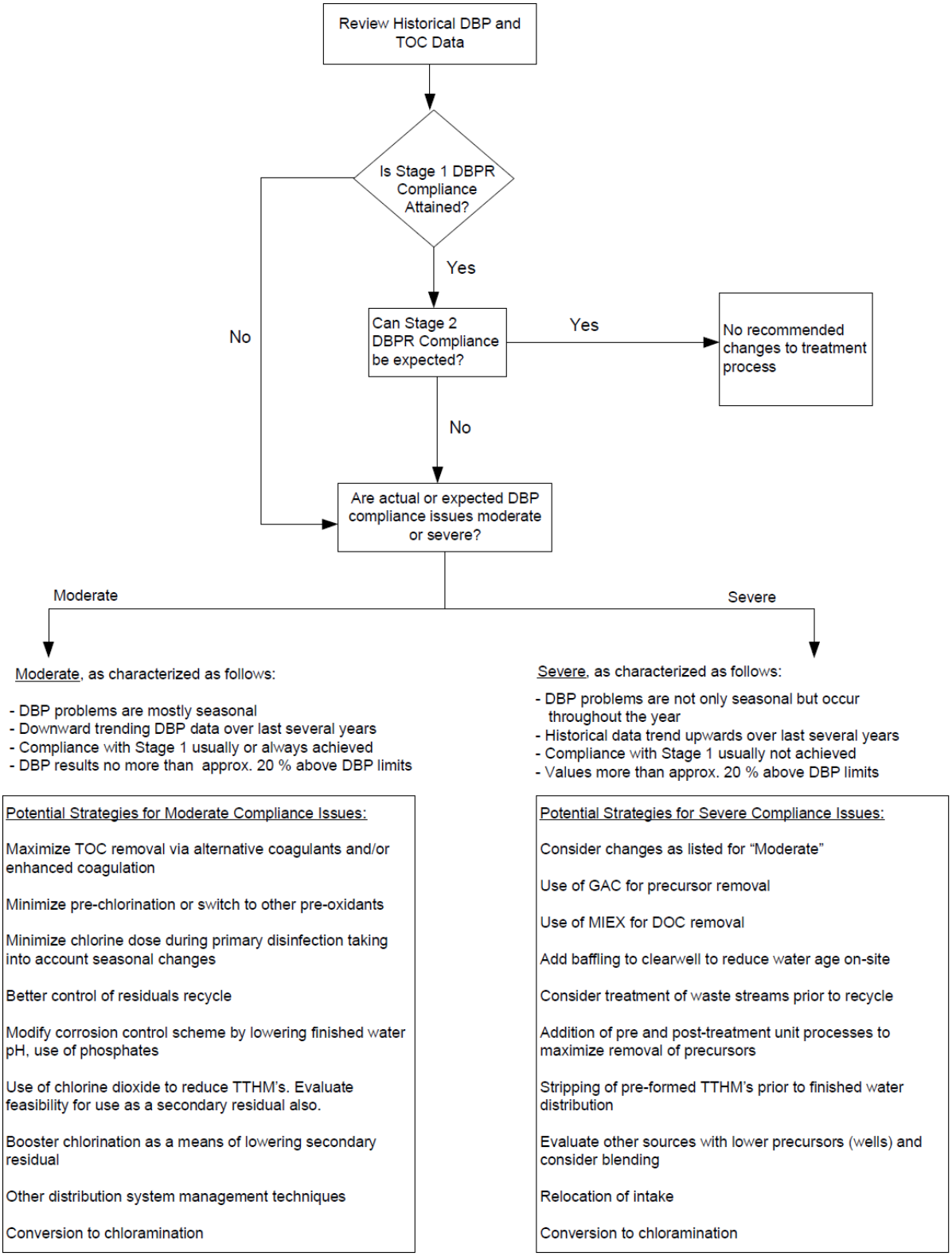
The formation of DBPs can be described as follows:

$$TOC + Cl_2 \xrightarrow{temp, time, pH} DBP$$

The AECOM recommendations provided below are based on this general schema above where precursors (i.e. TOC) and chlorine are looked to be minimized and time within the distribution minimized. An abbreviated version of the decision matrix used in selecting technologies and methodologies for addressing the Stage 2 DBPR is provided in Figure 5-1. The basic strategy when using chlorine as the residual disinfectant is to maximize the removal of TOC as generally the other parameters are relatively fixed. When using chloramine, the general strategy is to minimize the use of chlorine and not focus on TOC removal.

With the above information in mind, a brief discussion is worth presenting as to how the selected technologies were recommended in this section. It should be noted many systems are already in compliance with the Stage 1 DBPR and are on track for being in compliance with the Stage 2 DBPR rule. Further system optimization as presented in this section are likely to aid many systems in further reducing the overall DBP levels and potentially saving the utility from expending the funds for additional unit treatment operations (Other DBP Reduction Alternatives). As such AECOM has provided a step wise cost analysis up to the major treatment process being installed (Proposed Alternatives). We have not provided pricing on these Other DBP Reduction Alternatives due the variability in chlorinated DBP reduction, potential drawbacks with other regulated DBPs, or lack of full scale track record.

FIGURE 5-1: PROCESS FOR DEVELOPING TOOLBOX OF OPTIONS FOR WATER TREATMENT FACILITIES



For systems that are to remain using chlorine as the residual disinfectant, generally the first step is to stop prechlorination. Prechlorination allows chlorine to interact with the highest levels of TOC and has the propensity to form high levels of DBPs. The second step is to remove as much TOC in the source water as possible. This translates into a higher coagulant dose and will be determined by filter run times and/or the point of dimensioning return where increased coagulation dosages yields little increased TOC removal. Once coagulation is optimized, and if the system is still not in compliance, then further TOC removal processes are considered. For most systems AECOM has recommended seasonal (summer/fall) use of granular activated carbon (GAC) as the final endpoint for treatment due to:

1. GAC track record for DBP reduction
2. Operator friendly
3. Regulatory friendly

The GAC life endpoint is determined when the filtered TOC level from the GAC absorber reaches 1.5 mg C/L. The GAC life was forecasted based on research by Summers et al. (1994). In some cases due to long system residence time or site constraints, nanofiltration was recommended with a goal of blended TOC of less than 1.0 mg C/L. In one case MIEX was also recommended as an alternative as enhanced coagulant was not considered viable prior to slow sand filtration. Several systems were provided with costs for a well source as being an alternative source with higher water quality and in some cases an option to reduce water age in the distribution system.

It should be noted that other treatment alternatives could also achieve the desired chlorinated DBP endpoint. It was felt by AECOM that the recommended technology in this section was the best fit based on the information provided by the state and individual utilities. One example of an alternative technology would be ozone. This type of system could easily fit into the existing hydraulic gradeline for many systems, such as slow sand filtration. However due to, operator attention, operator health and safety, potential DBPs, and mechanical requirements, it was elected by AECOM not to recommend this option. Similar reasoning was used for other technologies.

For chloramine, as mentioned previously, the goal is to minimize the amount of contact time with chlorine while maintaining disinfection efficiency. Typically, our recommendation for chloramine has followed the addition of UV to obtain Giardia disinfection credits, followed by chlorine for virus inactivation, and then addition of ammonia for forming chloramine as a residual disinfectant. It should be noted that several systems have a large amount of storage on site and may be able to add chloramine prior to the on-site storage tank. This will be explored further in the final draft report.

The following discussion presents the alternatives which are viable in addressing the utility's ability to obtain compliance with the Stage 2 DBPR. There is no guarantee that one option will solve the DBP formation problem, and it is more likely that several steps will be required both at the water treatment facility and in the distribution system. AECOM has provided specific steps each system can take in addressing the Stage 2 DBPR compliance and has provided a 20 year life cycle cost. In addition, AECOM has provided a list of operational changes both in the distribution system and at treatment facilities which are generally considered good practice. It should also be understood that the alternatives recommended will require VT DEC Water Supply Division approval for implementation, and in some cases, may require new technology approval. AECOM recommends that all alternatives provided below be pilot tested prior to full scale implementation (with some exceptions such as UV disinfection), even if the process has received historical technology approval. This is due to the fact that the regulations have changed since many of the technologies have been approved. In addition, major process changes could have unintended effects on other rules. Pilot and small scale testing allows confirmation that the technologies can meet the requirements of the current rules as well provide a firm foundation for operational requirements and associated costs.

It is noted that most facilities did not record TOC on a frequent interval. Each system should record the source water TOC and finished water TOC as well as UV254 absorbance in order to determine plant performance. The recommended testing should be at least monthly during winter conditions, and weekly during warmer months or seasonal transition periods. This testing range can be revised as seasonal trends are established. This data will guide each system in understanding when operational changes are required. Online instrumentation can be installed (e.g. TOC monitor or UV254 monitor) to provide real time feedback.

5.2.3.1 Alburgh Village - Existing System

The treatment summary for the existing system is provided below. Figure 5-4A provides a schematic of the system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source for Alburgh is Lake Champlain with two intakes. One intake is new and serves as the primary intake; other intake is older and acts as the standby intake. The intake is located at a single elevation, several feet off the lake bottom.

Treatment and Distribution: Treatment consists of two modular Aquarius filter units (flocculation, Tube sedimentation, and filtration) followed by an unbaffled clearwell. The distribution has an elevated storage tank at a remote location. See Table 5-6 for more information. Currently, the Town performs seasonal reduction of chlorine to account for winter temperatures, but is somewhat hampered in chlorine reduction due to poorly baffled clearwell. The Town also flushes the distribution every spring.

In discussions with Phelps Engineering, the Town's primary consultant, much of the distribution system was upgraded in the 1970's to ductile iron pipe, although it is not clear if the pipe is cement lined or not. The system is planning to provide water to a neighboring fire district (F.D.) which will double the number of connections, and potentially add about four times the length of pipe to the existing system. The treatment equipment has reached its useful end of life and Phelps Engineering is working with the Town to install an updated filtration technology within the existing building footprint.

Operator: There is one full time operator and one part time operator working two weekends a month. The WTF generally runs 24 hours a day, seven days a week.

VT DEC Observations: Coagulation does not produce a settable floc. In discussions with the operator, a visible floc is formed.

TABLE 5-6
ALBURGH SYSTEM SUMMARY

Population			
Value	576	Planning to add Fire District with additional 300 to 350 connections	
System Demand, MGD			
	June, 2008	**Jan, 2009**	
Minimum	0.076	0.074	
Average	0.090	0.079	
Maximum	0.108	0.087	
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	5	20 additional miles with F.D. addition	
-Estimated average diameter, inches	6	0.2 mi 12", 1.5 mi 8", rest 6" and smaller	
Approximate System Detention Time	3 days current system (7 days future)		
Number of Storage Tanks	1		
-Provide Size of each, MG	0.20		
-Tank Type	Elevated		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry**
Prechlorination	No		
Preoxidation	Permanganate (only for Zebra Mussel Control)	Typically not used	
Coagulant	Slack Plus (Al/Polymer Blend)	73*	Wet
pH Adjustment	25% Sodium Hydroxide	Target pH 7.5 to 8.0	
Disinfection	12.5% Sodium Hypochlorite	1.0 to 1.5 (Residual) 1.2 required for CT	Dry

*Based on an estimated S.G. of 1.2 bulk diluted to 10% v/v (~SG = 1.05), a treated flow of 0.09 MGD, and chemical pump flow of 2.3 gph.

AECOM Observations: The system is looking to upgrade the existing filtration system. During the site visit by AECOM, the filter units were down for maintenance, thus floc formation could not be confirmed.

5.2.3.2 Alburgh Village - Proposed Alternatives

AECOM recommends that Alburgh follow the steps provided in Table 5-7 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-2 and 5-3 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Alburgh will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Alburgh. The summer DBP level used for calibration was 82 ppb for TTHM and 50 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 67 ppb, HAA5 = 44 ppb
2. UV and post clearwell chloramination: TTHM = 58 ppb, HAA5 = 29 ppb
3. UV and pre clearwell chloramination: TTHM = 12 ppb, HAA5 = 6 ppb

For Chlorine as the Residual Disinfectant (Figure 5-4B)

1. A new packaged clarification and filtration system should be installed which provides flexibility in coagulant dosing.
2. Optimize coagulant dosing for TOC removal.

For Chloramine as the Residual Disinfectant (Figure 5-4C)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, after the existing system is optimized for TOC removal, addition of chloramine will likely reduce the distribution DBPs to a level below MCLs.
2. A UV system will decrease the amount of chlorine required for disinfection at WTF, or alternatively there is a possibility of just chloramine as the primary and secondary disinfectant.

TABLE 5-7
PROPOSED ALTERNATIVES FOR ALBURGH

Treatment- Existing System	Distribution
Chlorine	
1.) Dissolved Air Flotation followed by rapid sand filtration. An alternative system is the Trident HS system.	None proposed
2.) Enhanced Coagulation	None proposed
Treatment Chloramine Distribution	
1.) Install UV to reduce chlorine CT	1.) Chloramine

5.2.3.3 Alburgh Village - Other DBP Reduction Alternatives

A summary of other DBP reduction alternatives for Alburgh Village is provided in Table 5-8 and Figure 5-4D for the interim period when the new WTF is being designed and constructed. The following provides a brief discussion of each alternative.

Treatment Alternatives at WTF

1. Optimize the existing operations for TOC removal. The existing facility appears to be dosing a substantial amount of coagulant. It is anticipated that Alburgh will need to optimize this dose based on the TOC removed through the filtration system. A typical acidic metal coagulant would also reduce the pH of the water and make disinfection more effective, thus reducing the required time for disinfection (or alternatively the required chlorine dose). As shown in Table 5-2 and Table 5-3, the required disinfection values (i.e. CT) for chlorine significantly drops as the pH is lowered.
2. Change caustic dosing location to after the clearwell. This allows the contact basin to take advantage of the lower pH which occurs during coagulation.
3. Add a curtain baffle to increase baffle factor. This will increase the accredited residence time in the clearwell and reduce chlorine required.
4. Strip chloroform from finished water. Chloroform is the major component of TTHMs formed and could be removed either at the clearwell or elevated tank. Note that stripping will have little effect on HAA5 values and also may reduce chlorine requiring booster chlorination.

The original design of the WTF has a PAC room. As an interim process, the Town could install a PAC system for targeted removal of TOC. Historically, PAC is typically used for taste and odor compound removal and not for TOC removal and may not be cost effective.

Alternatives or compliments to treatment would be:

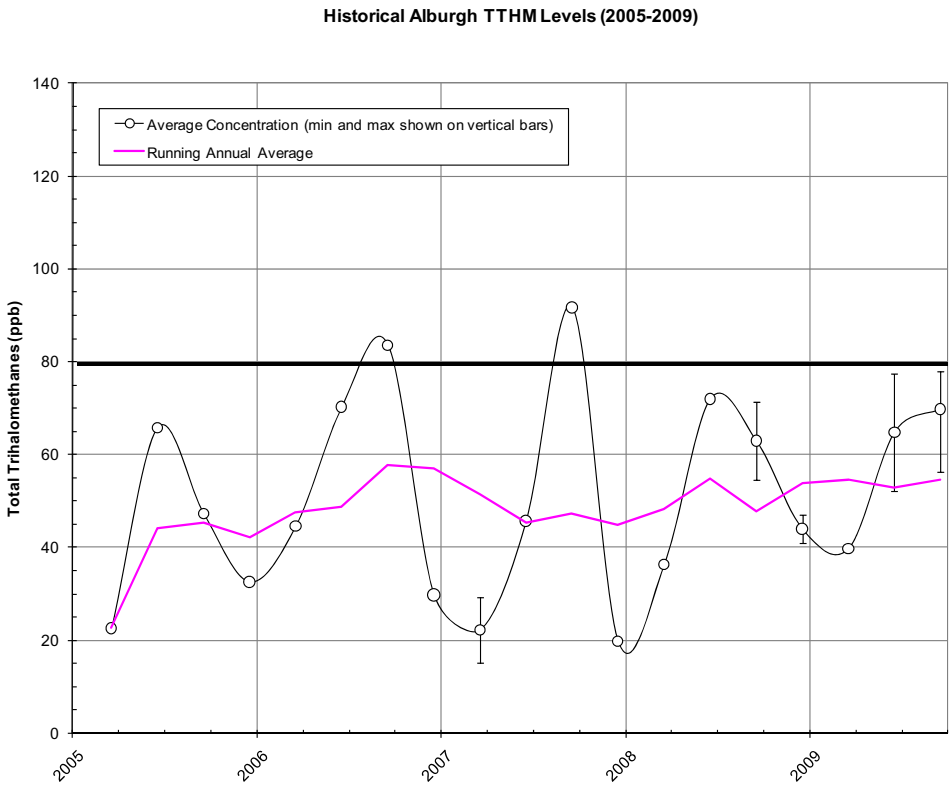
1. Test chlorine dioxide as a residual disinfectant. Note the discussion of benefits and drawbacks of chlorine dioxide (Section 4.3.2.4:Chlorine Dioxide (Preoxidation) and Section 4.4.4:Chlorine Dioxide (Implementation)). This option works best while the system is small due to the expected half life of chlorine dioxide.
2. Since the system contains several dead ends, the utility should consider looping the system so that water has an opportunity to circulate and not sit at a dead end. If this is not possible, then automatic flushing systems should be considered at the dead end. While automatic flushing provides some control over water age in the distribution system, it has the drawback of not using all the water treated for consumption, and should be used only during warm periods when DBP excursions are more likely.

TABLE 5-8
OTHER DBP REDUCTION OPTIONS FOR ALBURGH

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Caustic dosing point after the clearwell	Low	2.) Add automatic flushing device	Low
2.) Add a baffle curtain to clearwell	Low	3.) Chlorine Dioxide as residual disinfectant	High
3.) Strip chloroform	Medium or High		
4.) Install PAC	Low		

Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions



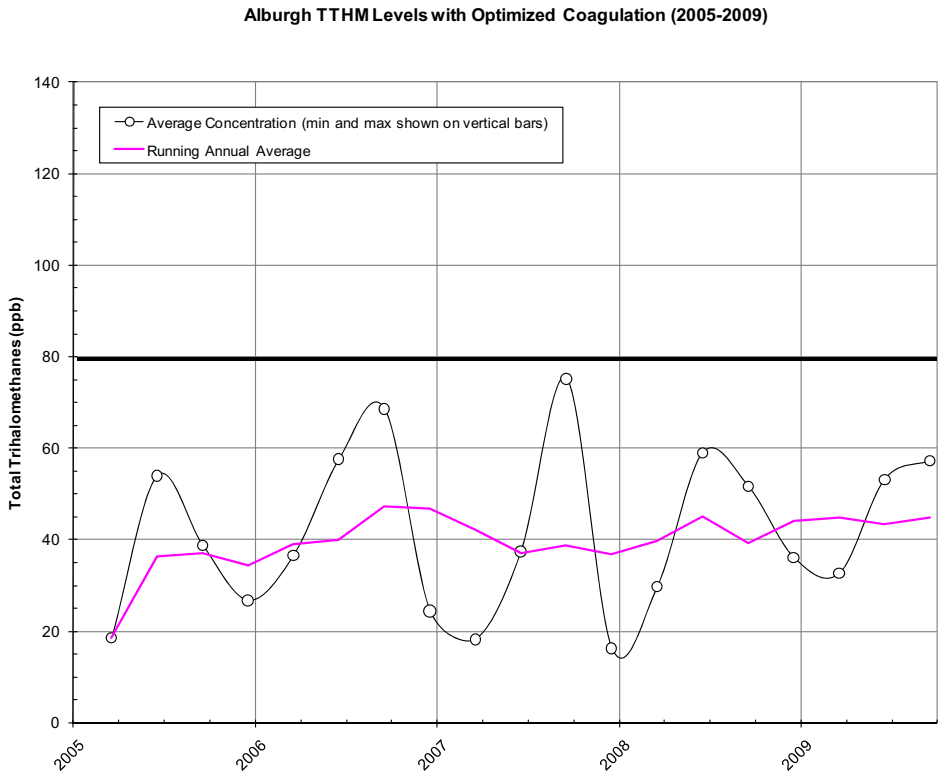
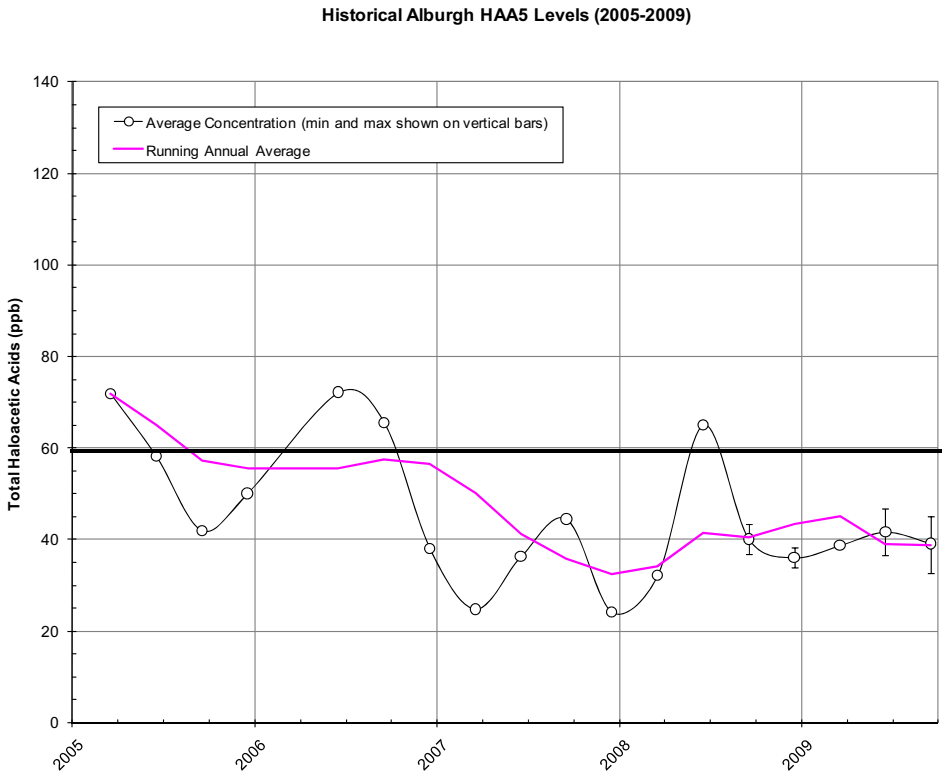


FIGURE 5-2
ALBURGH TTHM LEVELS (2006-2009)



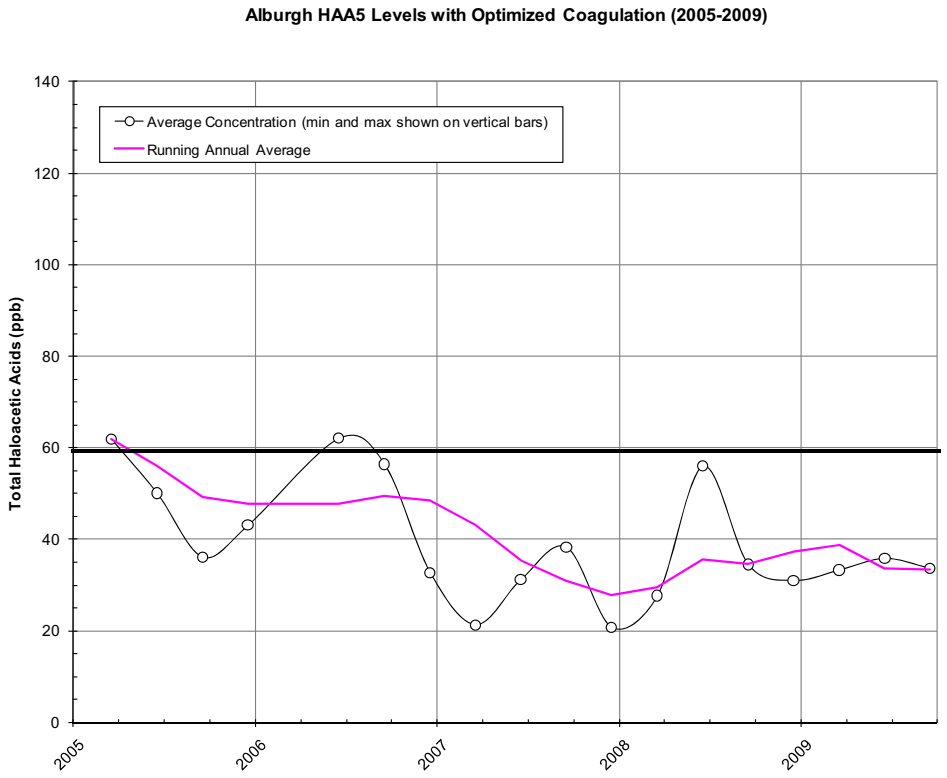


FIGURE 5-3
ALBURGH HAA5 LEVELS (2006-2009)

FIGURE 5-4A
ALBURGH VILLAGE EXISTING PROCESS FLOW DIAGRAM

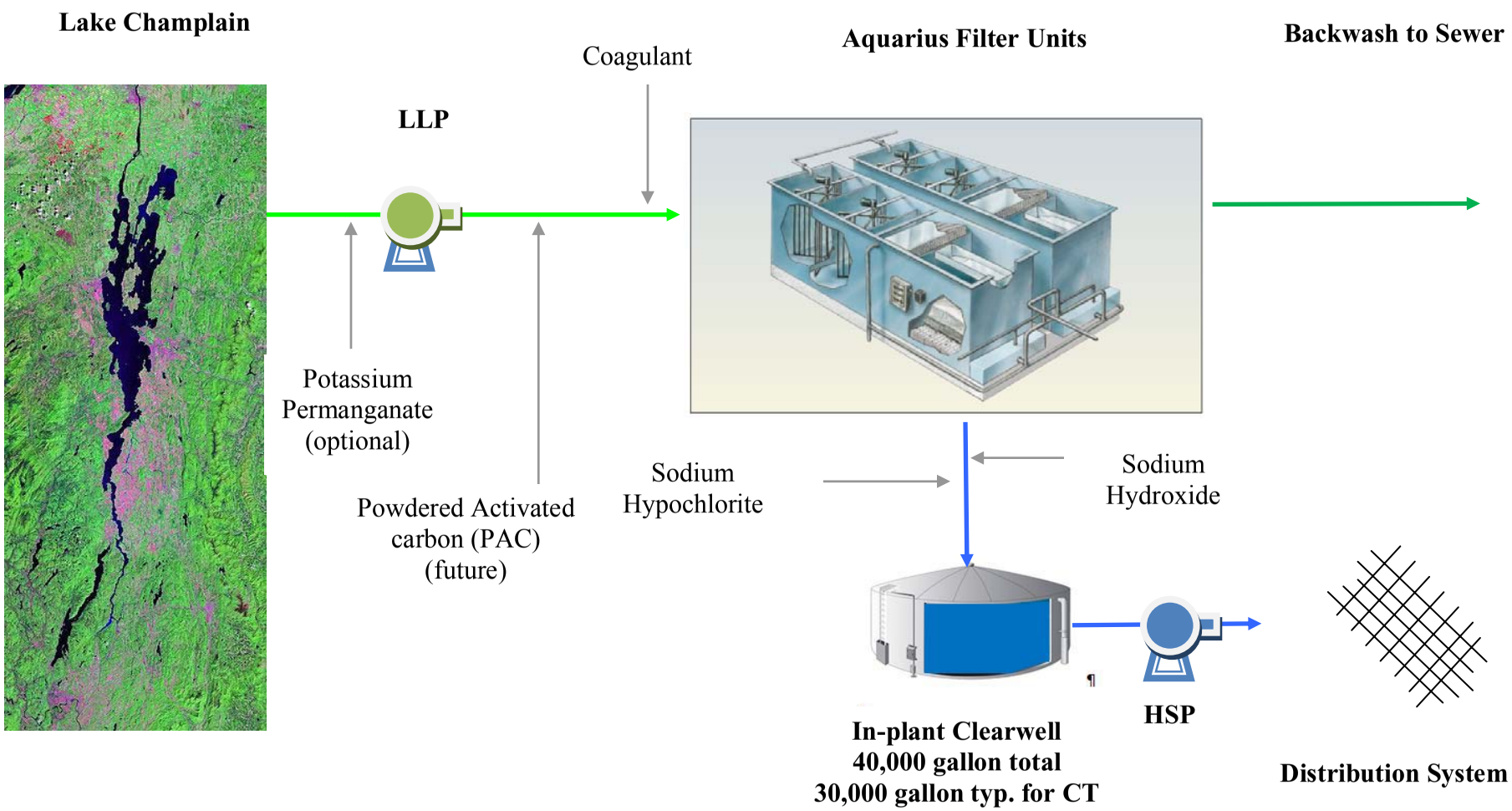


FIGURE 5-4B
ALBURGH VILLAGE UPDATED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

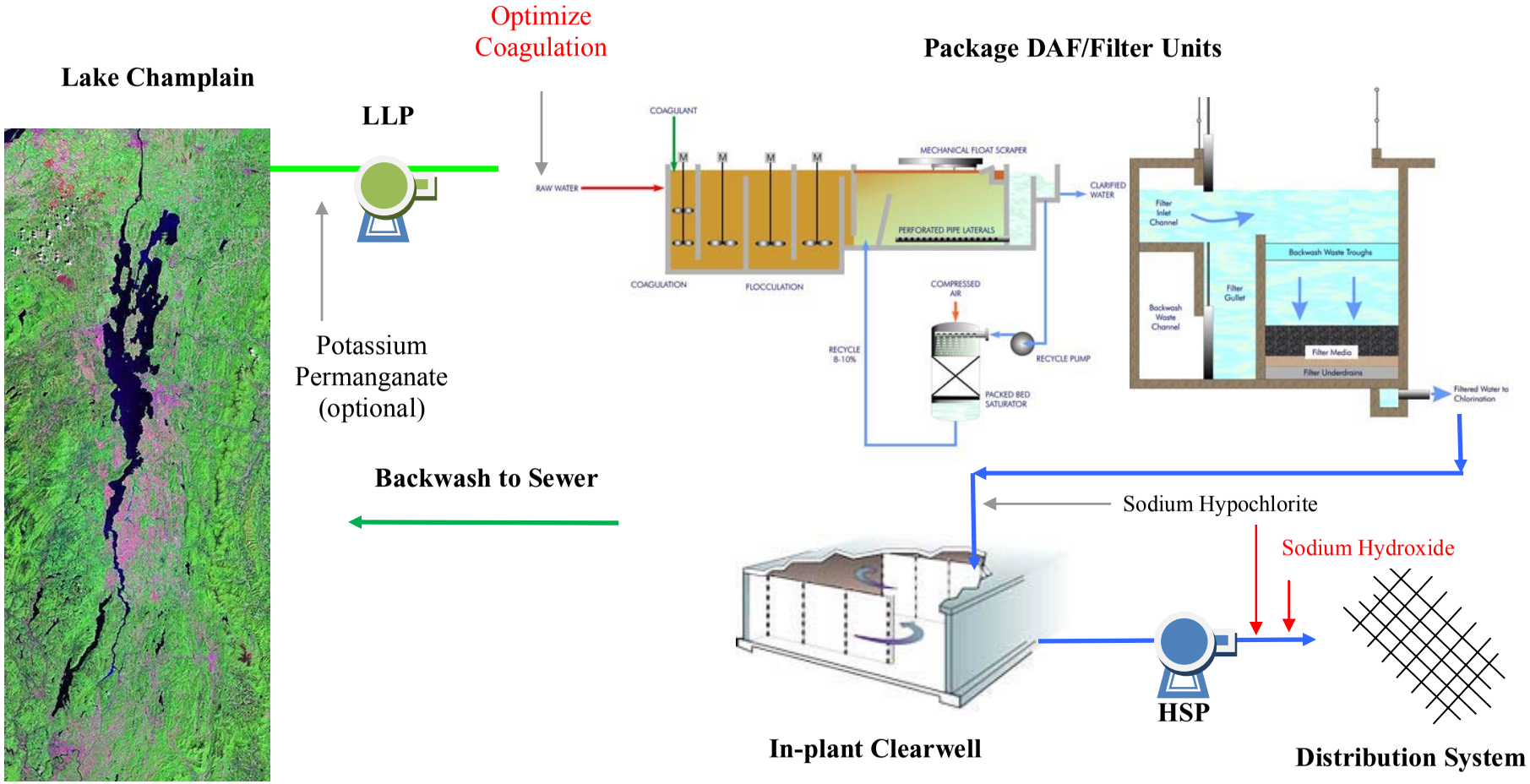


FIGURE 5-4C
ALBURGH VILLAGE UPDATED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

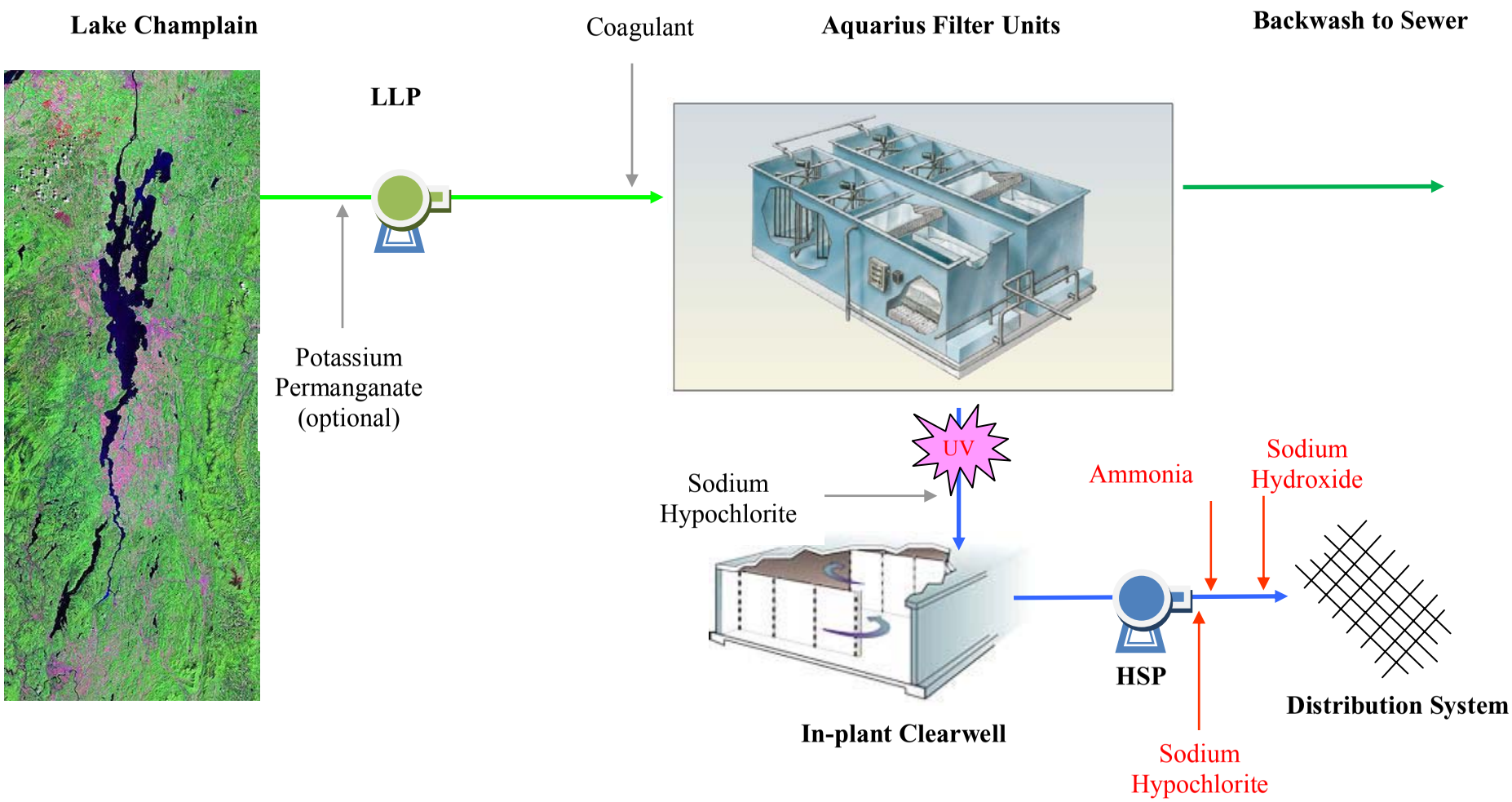
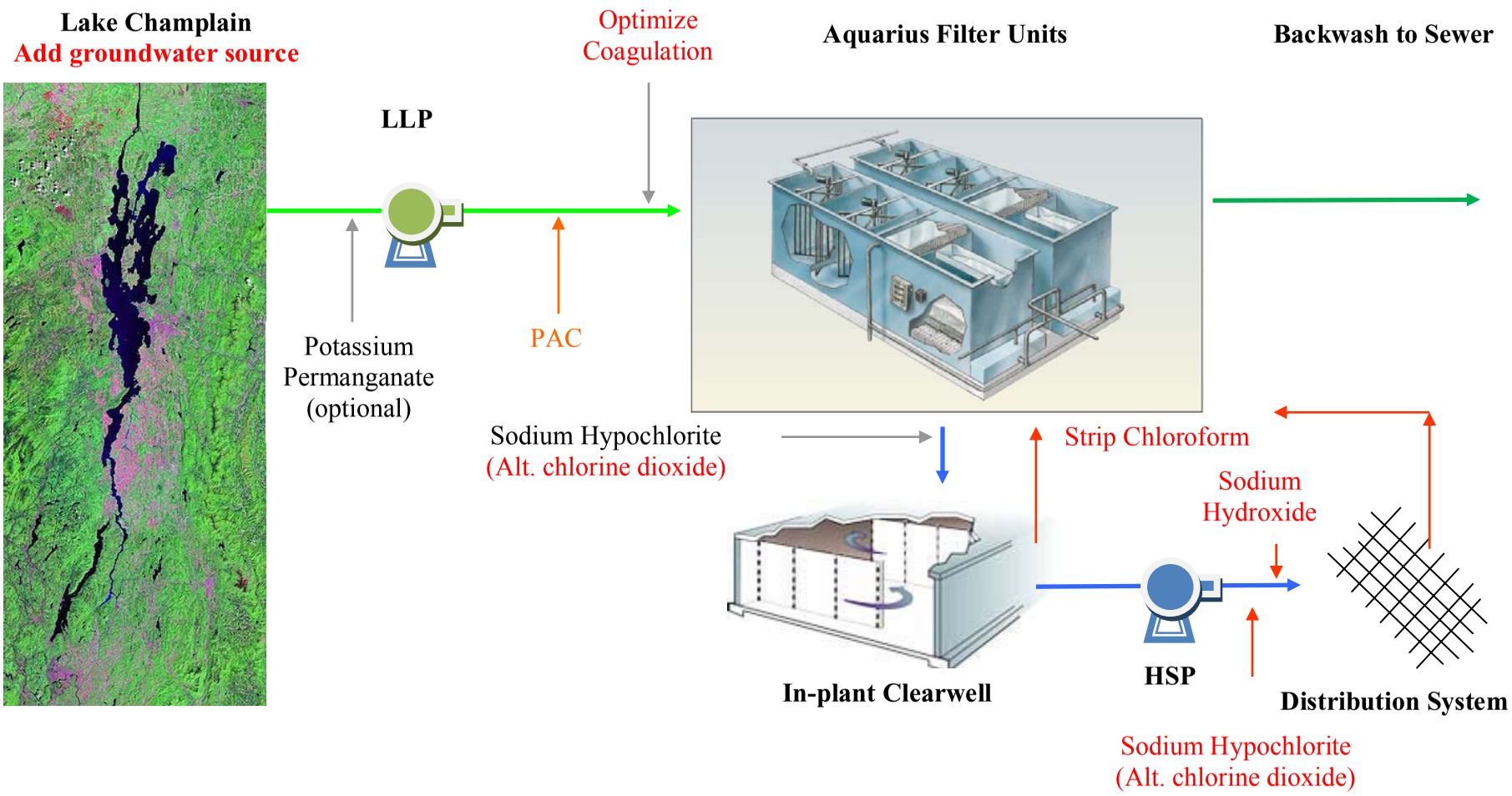


FIGURE 5-4D
ALBURGH VILLAGE PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS
(UPDATES IN RED)



5.2.3.4 Bennington - Existing System

The treatment summary for the existing system is provided below and shown in Table 5-9. Figure 5-7A provides a schematic of the current treatment. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source is Bolles Brook, an upland mountain stream, with low pH and alkalinity. Rain causes spikes in turbidity. There is one intake to the brook.

Treatment: Treatment consists of four modular Aquarius filter units (flocculation, tube sedimentation, and filtration) followed by clearwell all built in 1970's. An upgrade to the WTF was completed in 2007. A cationic polymer is dosed to maintain low filtered water turbidities. No visible floc is formed. The flocculator paddles have been removed and there is no means for mixing beyond turbulent flow in the pipe where chemical is injected.

During rain events, turbidity spikes can cause the filtered turbidities to rise as well, requiring an increase in chlorine dosage to maintain a chlorine residual. Operators typically de-rate the flow through the plant to maintain filter performance. The WTF is flow paced and the coagulant is trimmed using a streaming current meter. The coagulant trimming using the streaming current is somewhat successful.

As part of the WTF upgrade, a tank was added which increased the volume for disinfection, and allowed the discontinuance of pre-chlorination. This lowered the HAA5 levels which formed, but not the TTHM levels. In addition, bicarbonate was replaced with a carbon dioxide and lime system. This was done to add more calcium to the water to prevent corrosion of concrete structures in the distribution system. The target distribution pH of 8.7 has remained constant throughout the upgrades performed at the WTF

Distribution: Water is pumped to an onsite storage tank and system pressure maintained by gravity. There are several elevated storage tanks in the distribution system as well as one wellfield. Bennington flushes the system twice a year and uses a unidirectional flushing plan. In addition, there is one hydraulic and chlorine booster station in the system which feeds an industrial complex outside of main distribution system. Bennington services eight consecutive systems ranging in population size of 63 to 209.

Operator: There is one full time and six alternate certified operators. The WTF is not always manned and operates in automatic mode. The WTF is operated 24 hours a day, seven days a week.

VT DEC Observations: The water quality has always dictated the use of a coagulant dose suitable for direct filtration.

TABLE 5-9
BENNINGTON SYSTEM SUMMARY

Population			
Value	12,000		
System Demand, MGD			
	Typical		
Minimum	1.2		
Average	1.6		
Maximum	3.0		
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	60		
-Estimated average diameter, inches	6in to 8 in Dia.		
Approximate System Detention Time	20 hour average with 100 hour time at system edges		
Number of Storage Tanks	3		
-Provide Size of each, MG	3.0, 0.20, 0.23		
-Tank Type	3.0 MG – Concrete 165ft D x 20 ft H	0.20 MG Contrete Rectangular 40 ft x 50ft x 15 ft H	0.23 MG Steel 35 ft D x 28.5 ft H
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry**
Prechlorination	No		
Coagulant	EC-461	1.0	Wet
Primary Disinfection[1]	12.5% Sodium Hypochlorite	3.5(Warm) 2 (Cold) Target 1.0 residual; ranges from 0.5 to 1.3 Residual	Dry
Corrosion Control[2]	Lime	22(Target pH ~ 8.7)	Dry
Corrosion Control[3]	Carbon Dioxide	25-30	Dry

1. Based on 20 gal/day in winter and 30 gal/day in summer at 1.6 MGD.
2. Based on 18 tons used in 125 days.
3. Based on 350 lbs used in 1 day.

AECOM Observations: The WTF has not taken advantage of enhanced coagulation. Bennington can also take advantage of a high yielding well in Town which can supply presumably high quality water. The well water also would be cooler in the warmer months, aiding in DBP reduction. Analytical testing on well source can confirm this assumption.

5.2.3.5 Bennington - Proposed Alternatives

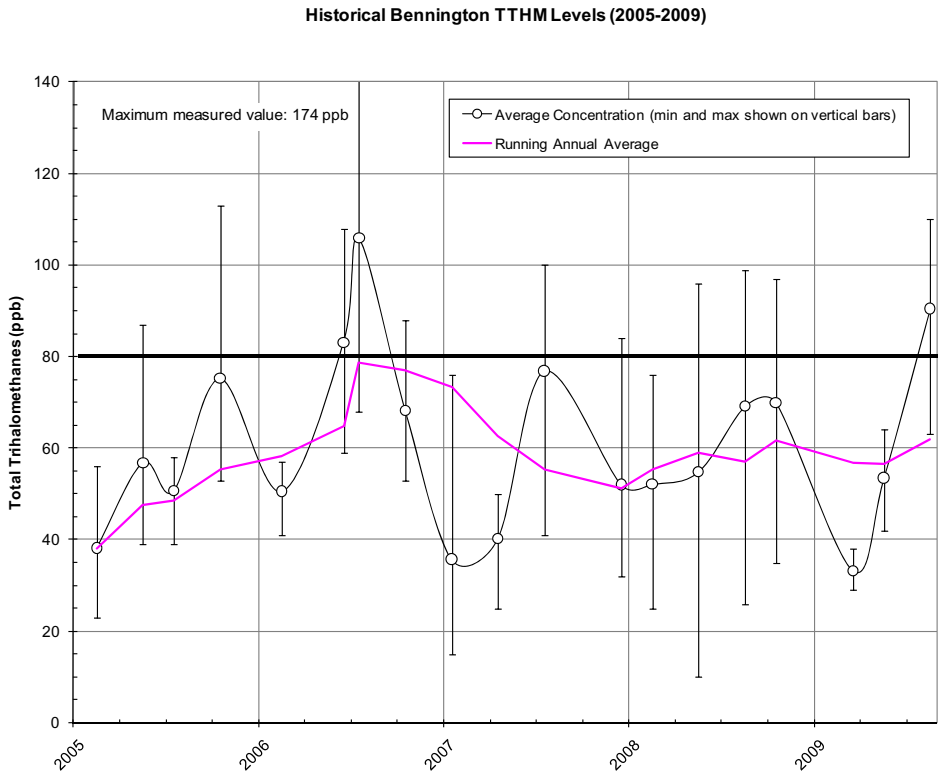
AECOM recommends that Bennington follow the steps provided in Table 5-10 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-5 and 5-6 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Bennington will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Bennington. The summer DBP level used for calibration was 52 ppb for TTHM and 45 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 35 ppb, HAA5 = 38 ppb
2. UV and post clearwell chloramination: TTHM = 16 ppb, HAA5 = 15 ppb
3. UV and pre clearwell chloramination: TTHM = 8 ppb, HAA5 = 7 ppb

For Chlorine as the Residual Disinfectant (Figure 5-7B)

1. Optimize the existing treatment operations to remove TOC. This likely requires more coagulant be dosed to remove TOC and/or an alternative coagulant scheme to current scheme be implemented. The VT DEC observation is that water quality dictates a coagulant dose suitable for direction filtration. If the water is relatively clean and cold, flocculation can be slow to form. The proper selection of polymer, mixing, and if necessary, settling agent can greatly aid in system performance. Installation of the flocculation paddles will be required to aid in the optimization of the flocculation. Increasing the coagulant dose will likely depress the pH and make chlorine more effective in disinfection.



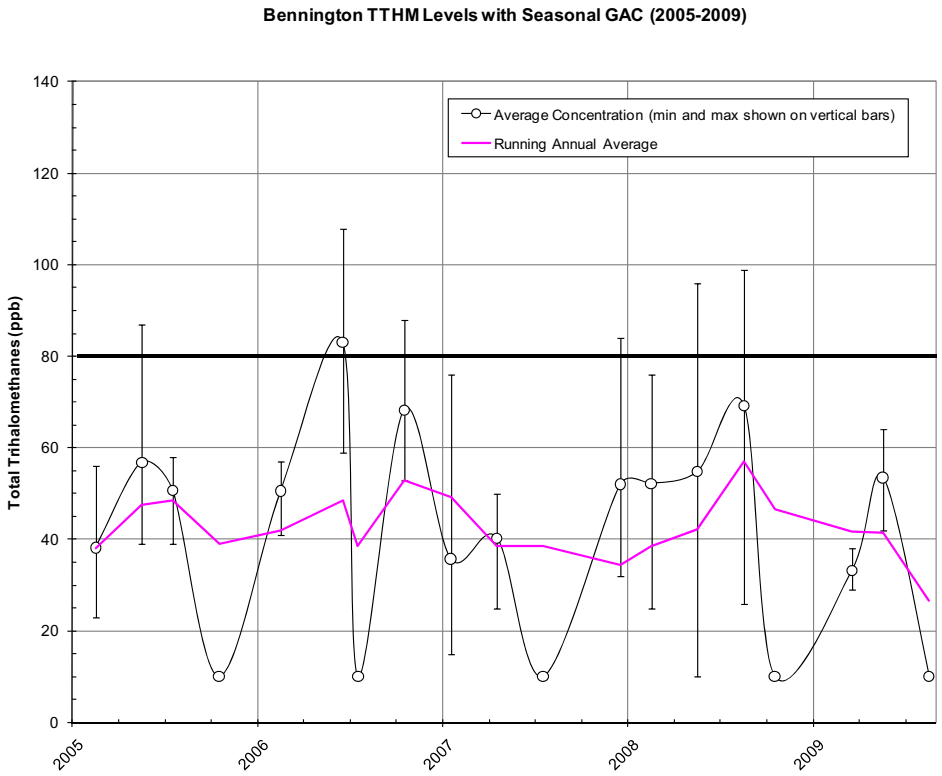
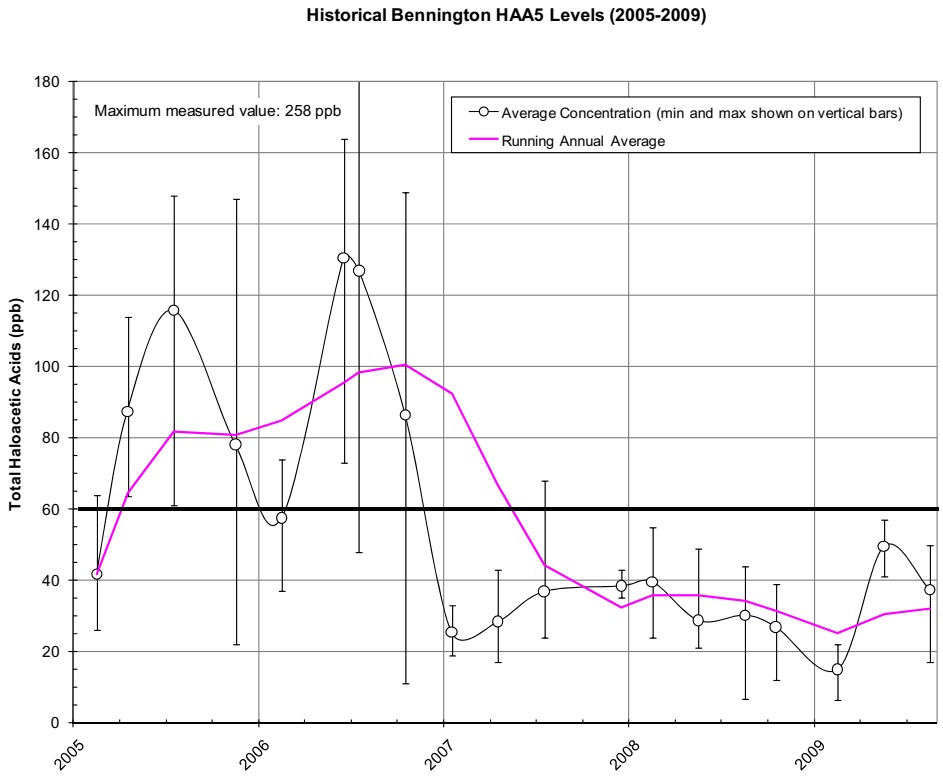


FIGURE 5-5
BENNINGTON TTHM LEVELS (2006-2009)



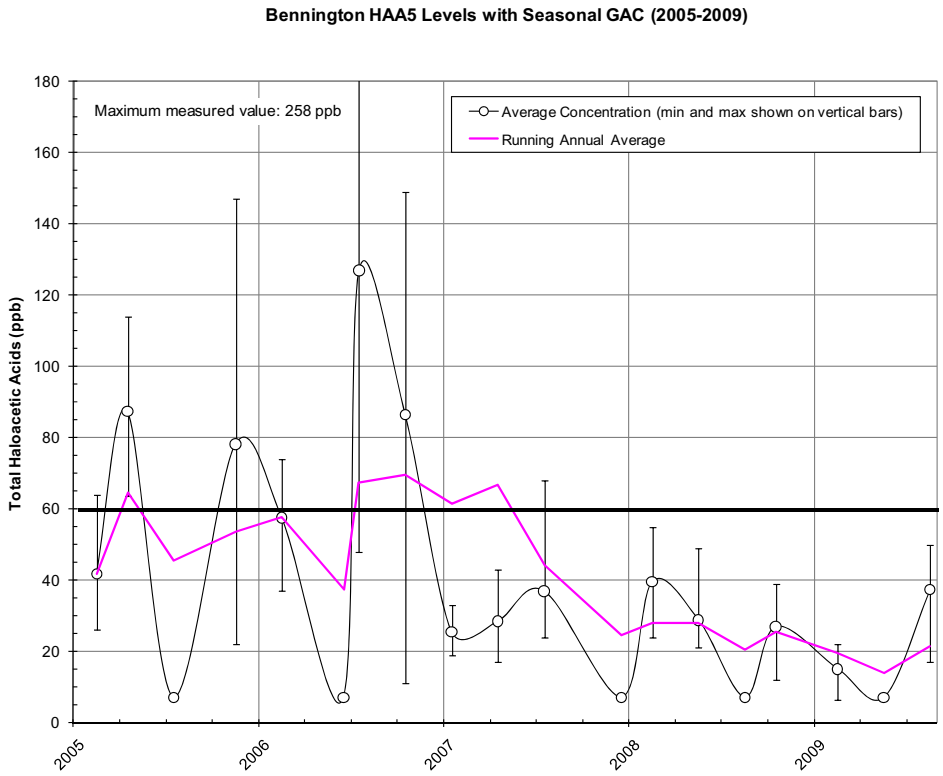


FIGURE 5-6
BENNINGTON HAA5 LEVELS (2006-2009)

2. Install a GAC contact filter after the sand filters to remove TOC from the filtered water.

For Chloramine as the Residual Disinfectant (Figure 5-7C)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.

TABLE 5-10
PROPOSED ALTERNATIVES FOR BENNINGTON

Treatment- Existing System	Distribution
For Chlorine	
1.) Optimize coagulation to remove TOC.	None proposed
2.) Seasonal (summer) post GAC Filtration with EBCT of 20 min	
For Chloramine	
	Chloramine

5.2.3.6 Bennington - Other DBP Reduction Alternatives (Figure 5-7D)

A summary of other DBP reduction alternatives for Bennington is provided in Table 5-11. The following provides a brief discussion of each alternative.

1. Currently, all the chlorine required to maintain a residual in the distribution system is dosed prior to the disinfection contact tank. Current disinfection values are reported ranging from 300 to greater than 700 mg-min/L (Appendix 3- Operational Data and Survey Information). This indicates Bennington has high contact time as the residual chlorine reported appears normal. The operation group has the ability to break up the chlorine such that a smaller hypochlorite dose can be added prior to the contact tank and then the required distribution chlorine dose added after the chlorine tank.

2. As mentioned previously, removing the prechlorination step reduced the HAA5s, but not the THMs. Chloroform specifically dominates the TTHM DBP fraction. Chloroform has a relatively volatile and can be stripped from the disinfection tank and/or distribution tank using various forms of aeration. It should be noted that stripping the chloroform may remove some chlorine and may require booster chlorination.
3. Operate the wells more frequently in the warmer months to reduce water temperature and provide a source water with less TOC into the distribution system. Prior to performing this operation, the well water should be tested for TOC and UV254 as well as performing the SDS test to determine reactivity of the source TOC with chlorine. However, anecdotal information and current water quality tested suggests the well is of high quality and is likely to have lower levels of TOC that is less reactive with chlorine than the surface source water.
4. Monitor the chlorine dose and adjust to take into account seasonal temperature changes and adjust to the lowest feasible chlorine residual in distribution. This should be coupled with more chlorine and biological monitoring as indicated in Section 4.5.4 – Strategies to Improve Water Quality in the Distribution System. This may require neutralization of the chlorine residual in water which is overflowed.
5. Control distribution system water age:
    a. Overflow the tank in the industrial area to cycle the hydraulic and chlorine booster pump more frequently and reduce water age in this section of the distribution system.
    b. Install seasonal automatic flushing unit on select dead ends to lower water age in distribution system.
    c. Both of these options require discharging water which is not consumed, but can result in water with lower age and lower DBP levels.

TABLE 5-11
OTHER DBP REDUCTION OPTIONS FOR BENNINGTON

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Split the chlorine between disinfection tank and distribution residual	Low or Medium	1.) Adjust distribution chlorine to lowest feasible level	Low
2.) Strip chloroform in clearwell	Medium	2.) Overflow tank in industrial area during select periods to reduce residence time.	Low or Medium
3.) Operate the wells in warmer temperature	Medium	3.) Strip chloroform in distribution tanks during warmer months	Medium
		4.) Add automatic flushing at dead ends	Low

Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-7A
BENNINGTON EXISTING PROCESS FLOW DIAGRAM

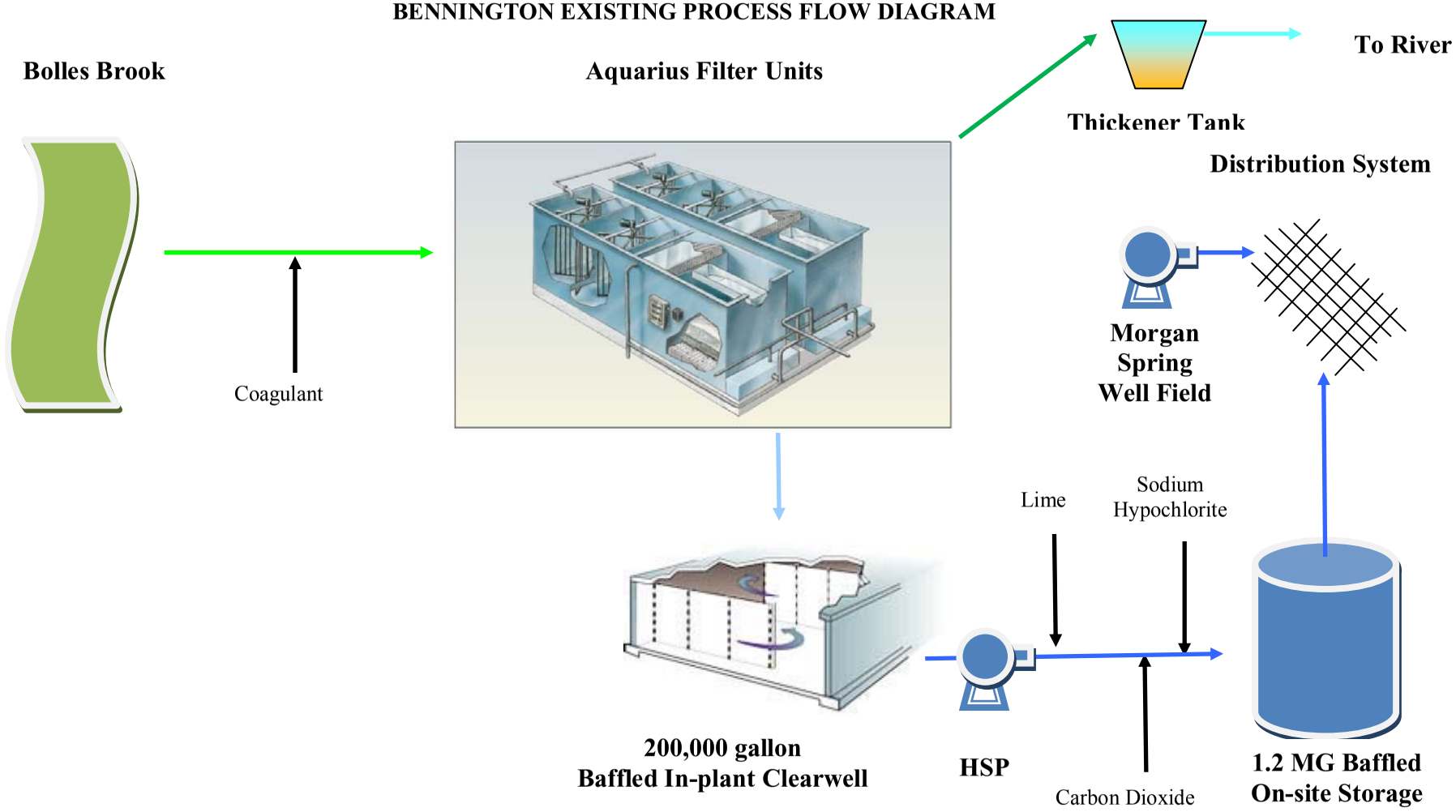


FIGURE 5-7B
BENNINGTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

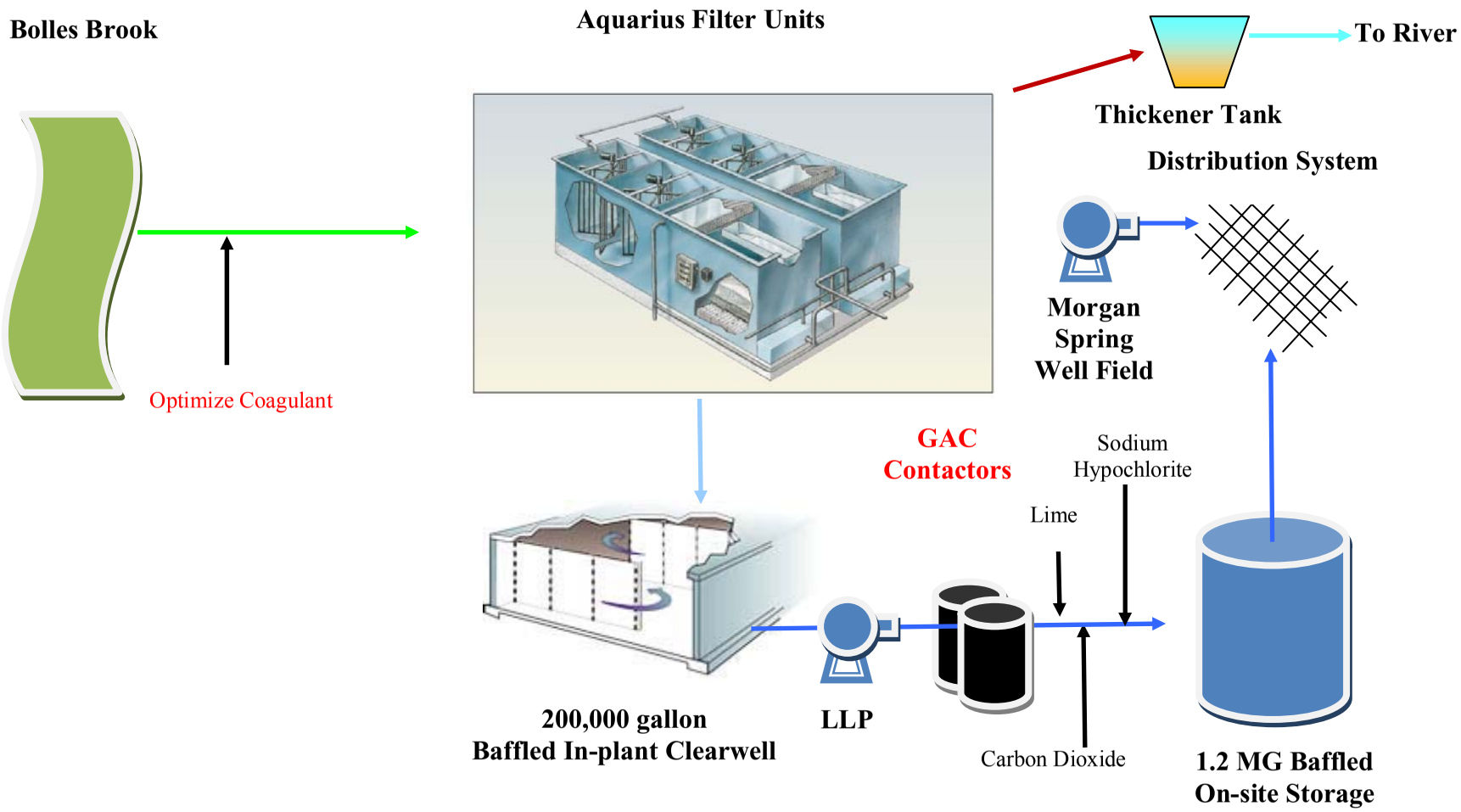


FIGURE 5-7C
BENNINGTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

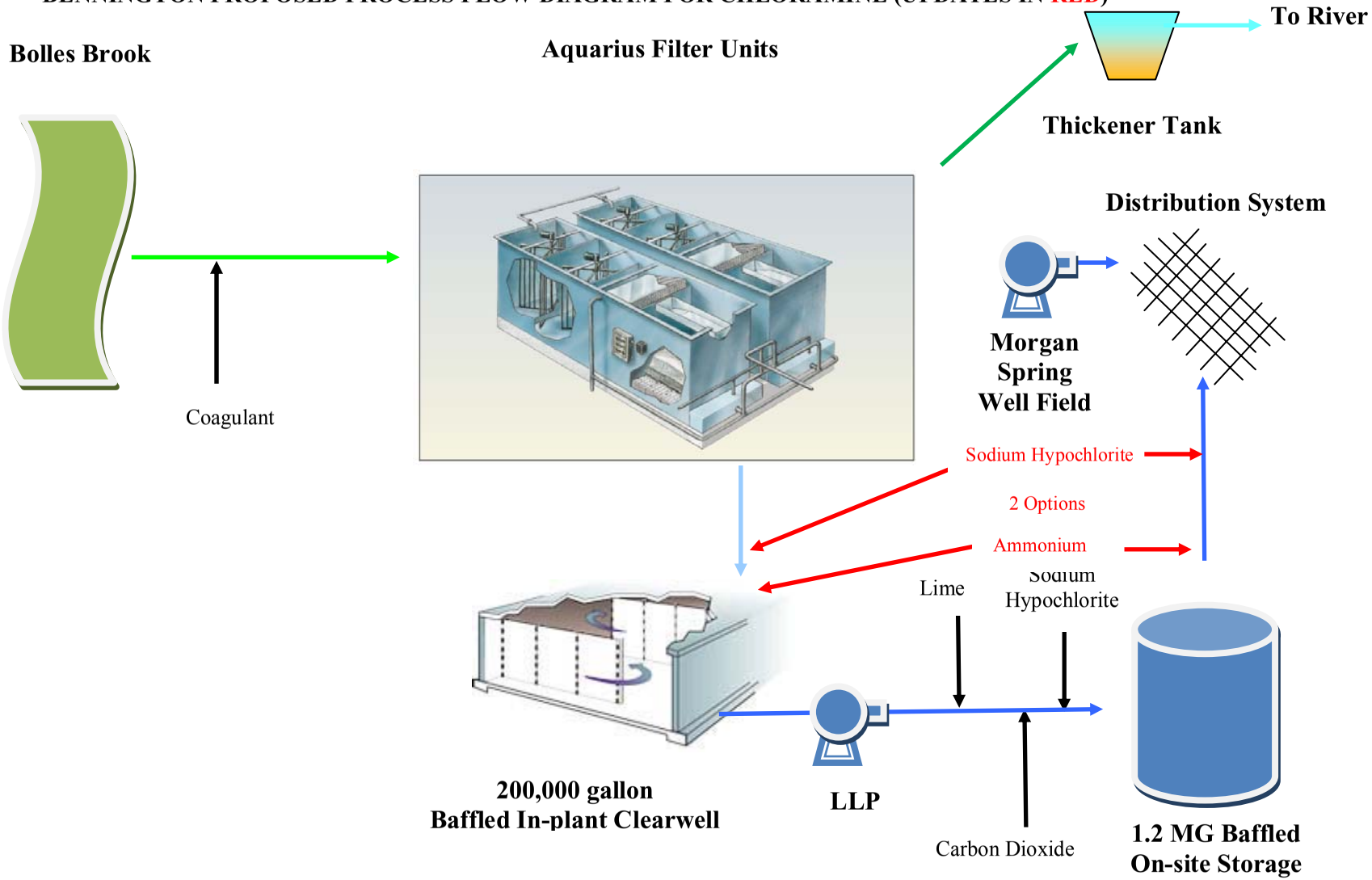
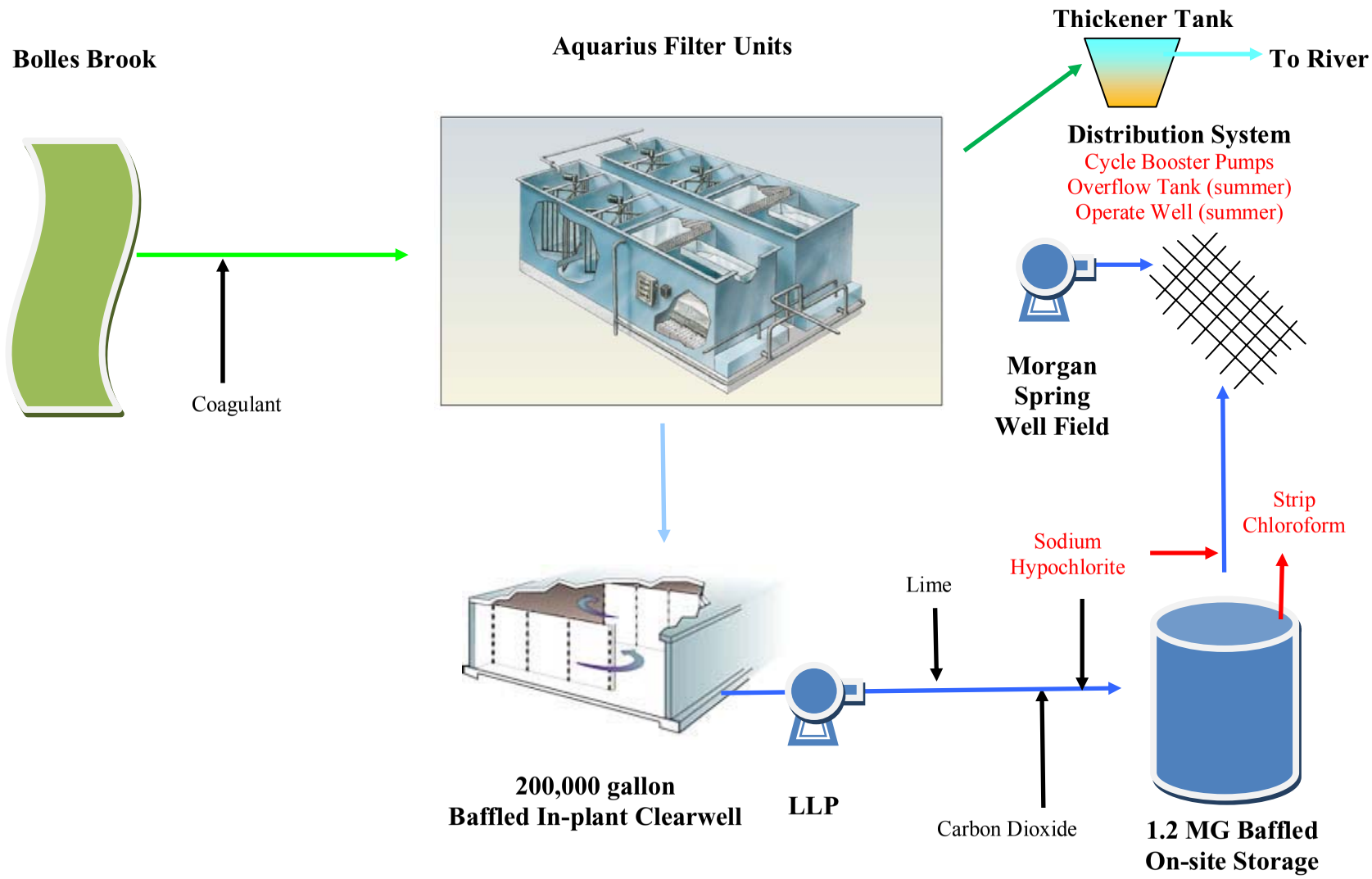


FIGURE 5-7D
BENNINGTON PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.2.3.7 Burlington - Existing System

The treatment summary for the existing system is provided below and in Table 5-12. Figure 5-10A provides the current treatment scheme. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source water is Lake Champlain. There is one intake which is several thousand feet offshore and roughly 40 feet under water and located roughly at the bottom of the lake.

Treatment: The raw water pump station draws water from Lake Champlain and to the rapid mix basins. Permanganate is dosed at the intake, mainly for Zebra Mussel control. Sodium hypochlorite is an alternative oxidant. From the rapid mix basins, water flows by gravity through the decommissioned superpulsator clarifiers and through the ABW traveling bridge filters. Typically, alum is dosed prior to the pre-mix basin and polymer is dosed after the clarifier. Water from the ABW filters flows to a wetwell. Low lift pumps direct the water through the rapid sand filters and water flows by gravity through two baffled 170,000 gallon clearwells in series. Alum and polymer are added again prior to the rapid sand filters. The finished water pumps take water from the clearwell and supply water to the low service zone. The City has a SCADA system which allows process monitoring. Currently, the City is upgrading the outdated Fischer and Porter instrumentation with PLCs to allow for process control and automation.

Distribution: The distribution consists of a two service zones to maintain uniform pressure throughout the system. The finish water pumps provide water to the to the low service zone and to the reservoirs. High service pumps lift water from the reservoirs to the elevated high service tanks. The City is currently developing a unidirectional flushing plan. Currently the system is flushed once a year. Burlington services three consecutive systems ranging from 28 to 8,300 in population size.

TABLE 5-12
BURLINGTON SYSTEM SUMMARY

Population			
Value	38,889		
System Demand, MGD			
	March 2008	August 2008	Jan., 2008
Minimum	3.21	3.77	3.50
Average	4.57	5.57	4.06
Maximum	4.05	5.31	4.59
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	100		
-Estimated average diameter, inches	~6 inch		
Approximate System Detention Time	Average < 24 hrs.	Extreme areas ~ 36 hrs	
Number of Storage Tanks	4		
-Provide Size of each, MG	4.8, 2.2, 0.5, 0.15		
-Tank Type	2 reservoirs (North-4.8 MG and South2.2 MG), 2 elevated (0.5 and 0.15)		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	Yes		
Preoxidation	Potassium Permanganate	0.53	Dry
Preoxidation prior to main rapid sand filters	12.5% Sodium Hypochlorite	0.73	Dry
Coagulant-Moore Prior to Anthracite ABW Filters	48.5% Alum	4.9	Dry
Coagulant-Main Prior to Sand Filter	48.5% Alum	1.4	Dry
Polymer-Moore: Prior to ABW Filters	Superfloc C-572 - cationic	2.3	Wet
Polymer-Main: Prior to Sand Filter	Superfloc C-572 - cationic	2.2	Wet
Disinfection	12.5% Sodium Hypochlorite	2.1	Dry
Corrosion Control	Zinc Orthophosphate	0.24 (0.12 residual)	Dry
Health-Dental	Hydrofluorosilicic Acid	0.83	Dry

Operator: The WTF is always manned and runs 24 hours a day, 7 days a week

VT DEC Observations: Direct filtration dose under utilizes clarifier.

AECOM Observations: The City can modify the existing clearwells so that the clearwell level is lower during warmer water periods. This may require a tracer test to define the baffle factor for each basin. In addition, the City can take advantage of enhanced coagulation. Enhanced coagulation would require a clarification system to be added in the existing superpulsator basin to remove the additional solids generated.

5.2.3.8 Burlington - Proposed Alternatives

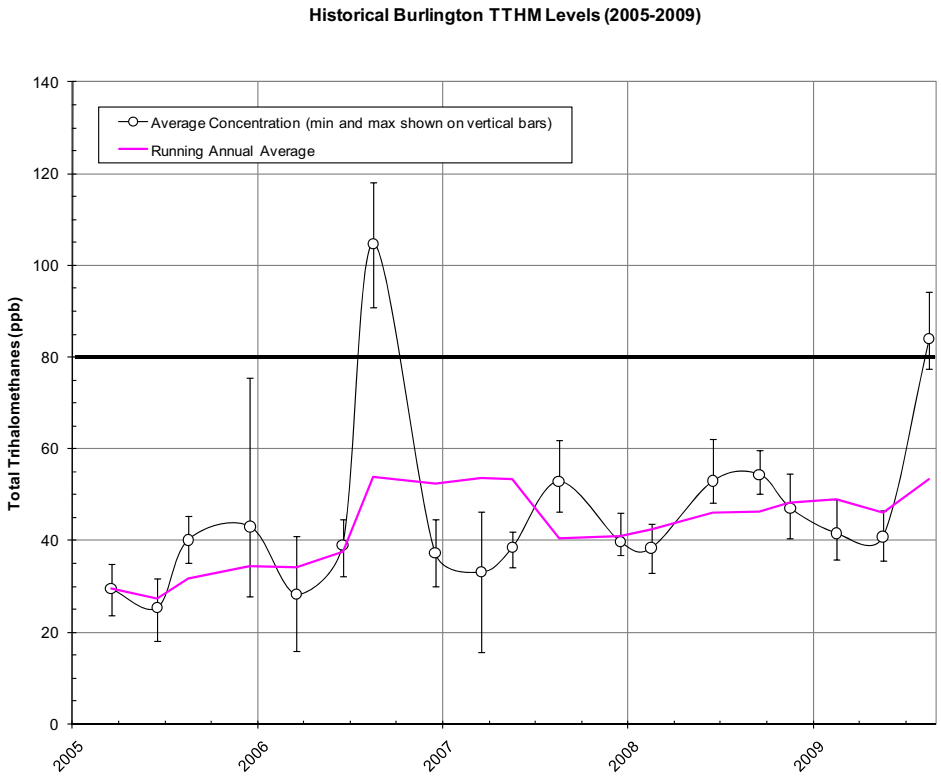
AECOM recommends that Burlington follow the steps provided in Table 5-13 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-8 and 5-9 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Burlington will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Burlington. The summer DBP level for calibration was 82 ppb for TTHM and 50 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 38 ppb, HAA5 = 31 ppb
2. UV and post clearwell chloramination: TTHM = 33 ppb, HAA5 = 20 ppb

For Chlorine as the Residual Disinfectant (Figures 5-10B)

1. Optimize the existing operations to remove TOC. This likely requires that more coagulant be dosed to remove TOC and/or an alternative coagulant scheme be implemented. Increasing the coagulant dose will likely depress the pH and increase the effectiveness of chlorine allowing a lower chlorine dose or less time for disinfection. However, a caustic dosing point will be required after the clearwell to raise the pH for optimal corrosion control.



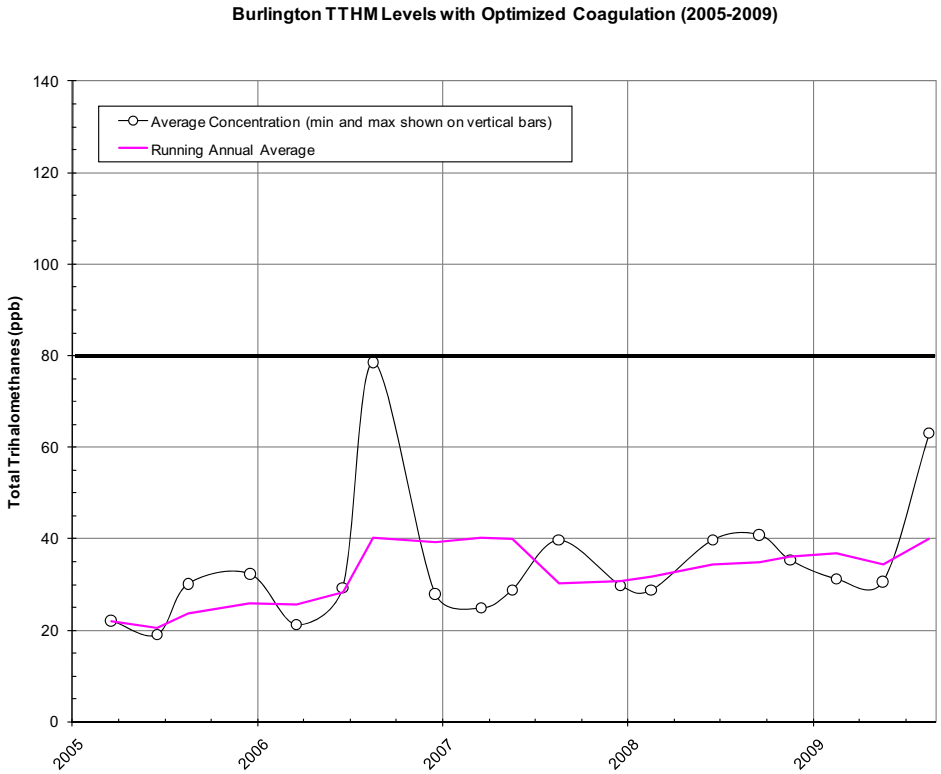
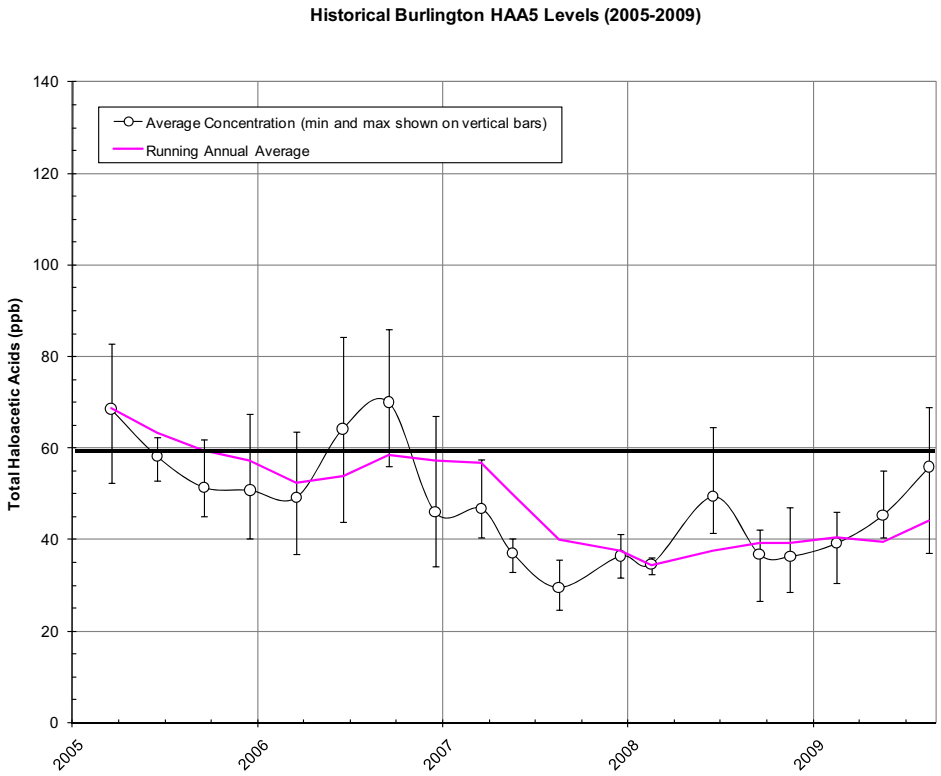


FIGURE 5-8
BURLINGTON TTHM LEVELS (2006-2009)



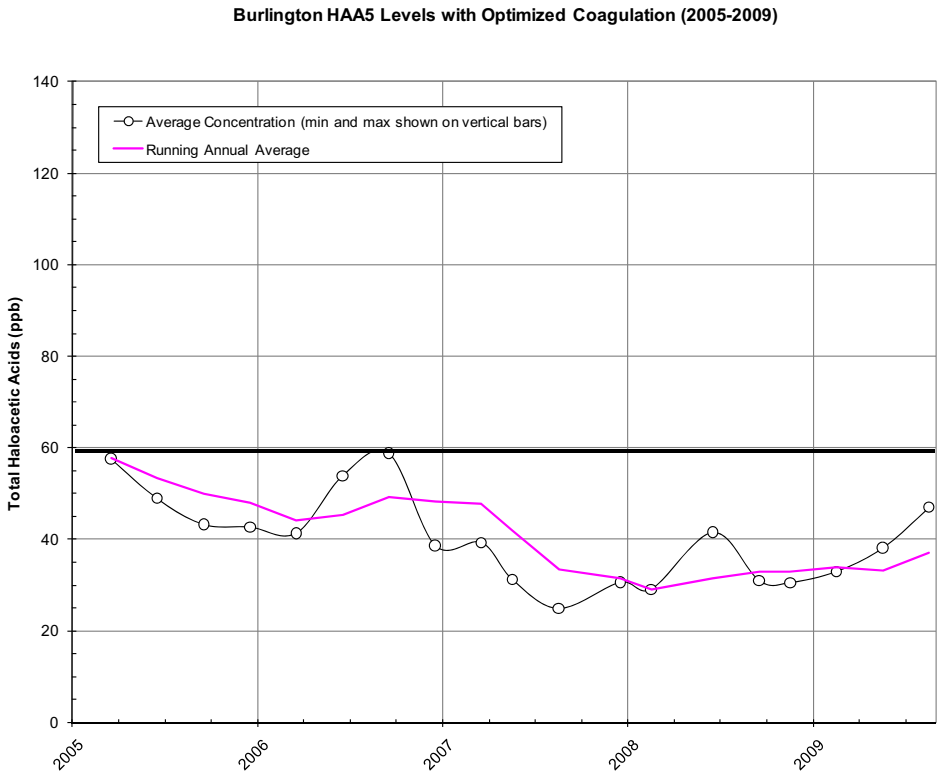


FIGURE 5-9
BURLINGTON HAA5 LEVELS (2006-2009)

In order for Burlington to do this will require that a clarification system be installed prior to the filters. The City has indicated that Actiflo was pilot tested and the results were not favorable. The likely clarification candidate would be to retrofit the existing superpulsator clarifiers to dissolved air flotation clarification.

For Chloramine as the Residual Disinfectant (Figure 5-10C)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, after the existing system is optimized for TOC removal, addition of chloramine will likely reduce the distribution DBPs below MCLs.

TABLE 5-13
PROPOSED ALTERNATIVES FOR BURLINGTON

Treatment- Existing System	Distribution
For Chlorine	
1.) Optimize coagulation to remove TOC. Profile TOC levels in the recycle water	None proposed
2.) Install a DAF clarification process.	
For Chloramine	
	Chloramine

5.2.3.9 Burlington - Other DBP Reduction Alternatives (Figure 5-10D)

A summary of other DBP reduction alternatives for Burlington is provided in Table 5-14. The following provides a brief discussion of each alternative.

1. The City currently has on loan from the State of Vermont a PAC feeding system which can be feed to the raw water line to treat for TFM, a lampricide. This system can be recommissioned to provide PAC as needed to remove a portion of TOC from the raw water. However, it is noted that based on one round of DPB testing, the use of PAC did not conclusively lower DBP levels.

2. Monitor the chlorine dose and adjust to take into account seasonal temperature changes to lower the chlorine dose to the lowest feasible level. Current disinfection values (i.e. CT) are reported about three times the required amount in warmer periods as shown Appendix 3 - Operational Data and Survey Information.
   a. It is noted that that the process drawings do not have a chlorine injection point after the clearwell prior to distribution. Adding an injection point would provide more flexibility in dosing chlorine and limit the contact time chlorine would have with TOC.
3. Booster chlorinate at select locations within the distribution system to reduce the overall time the mass of chlorine is in contact with the finished water in the distribution system.

TABLE 5-14
OTHER DBP REDUCTION OPTIONS FOR BURLINGTON

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Utilize existing PAC system	Low	1.) Booster chlorination	Low
2.) Reduce chlorine contact time	Low		

*Effect is the relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-10A
BURLINGTON PROCESS FLOW DIAGRAM

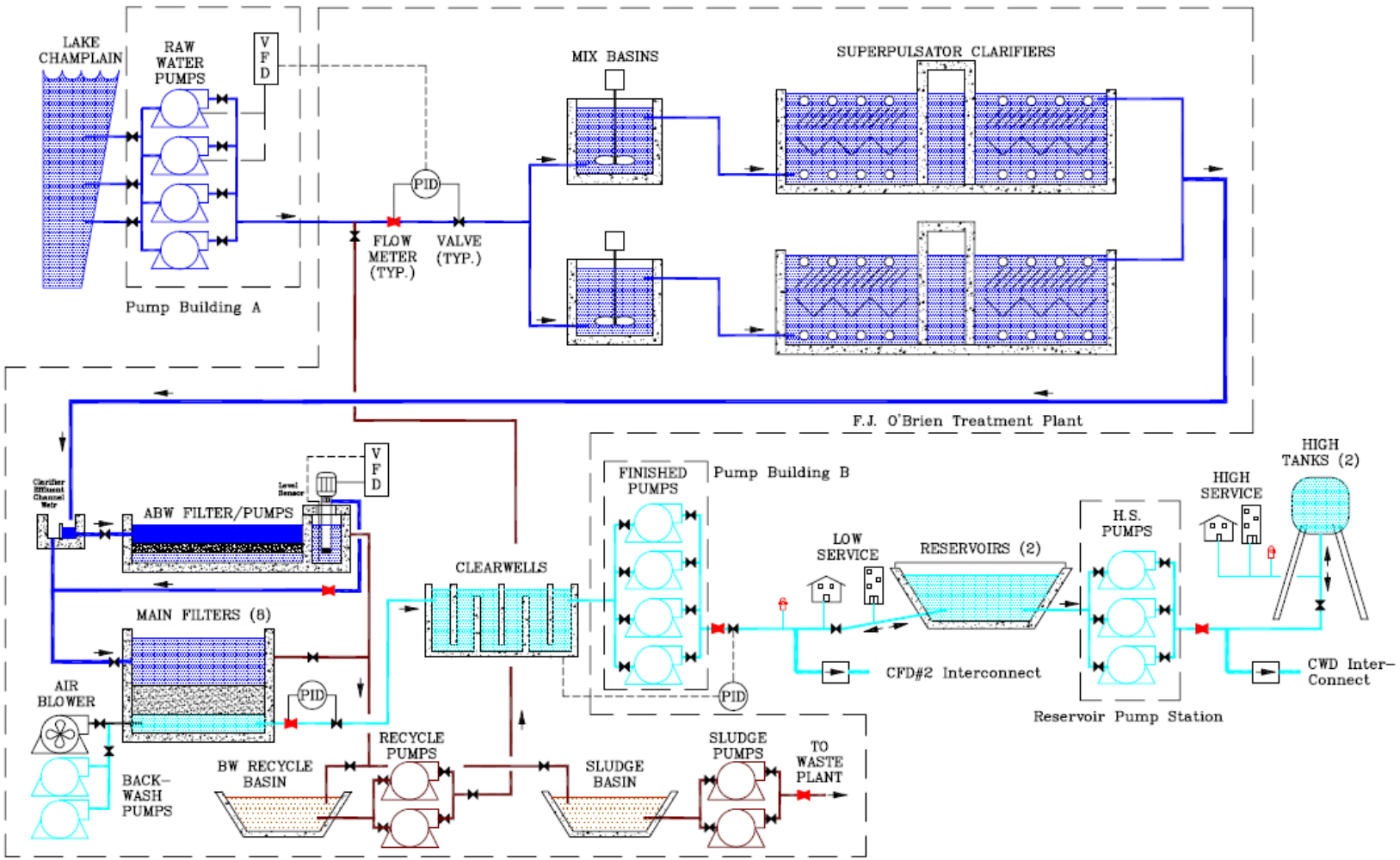


FIGURE 5-10B
BURLINGTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

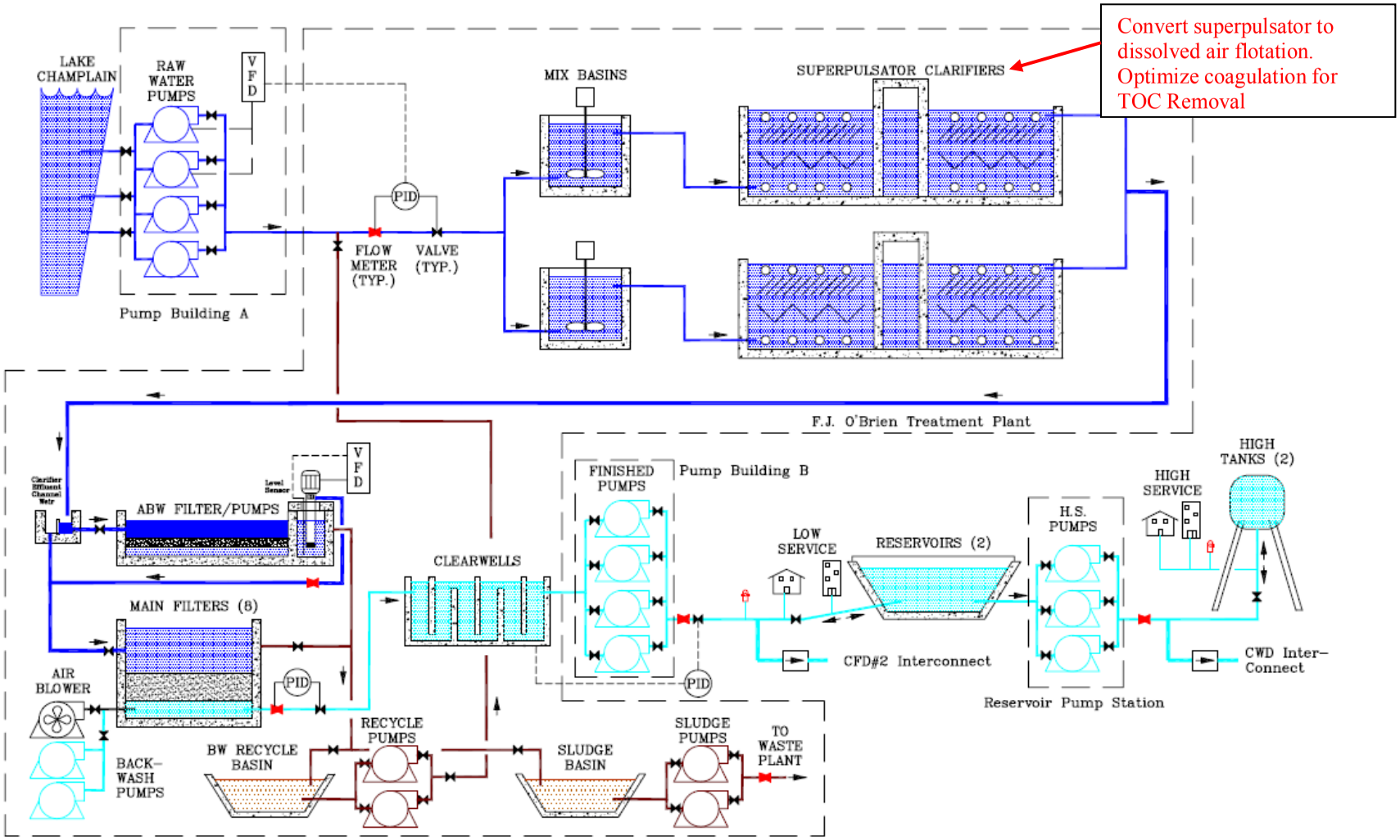


FIGURE 5-10C
BURLINGTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

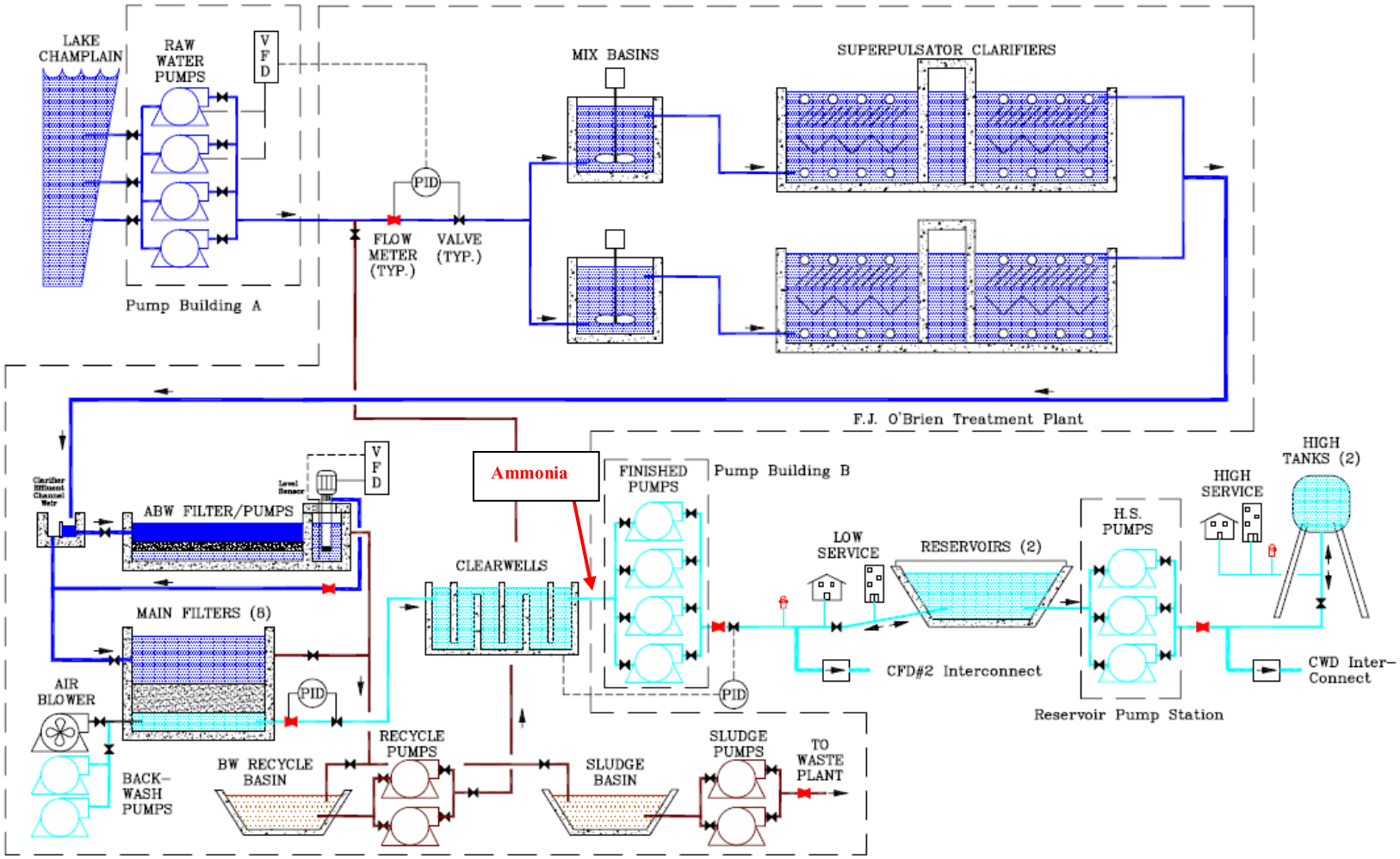
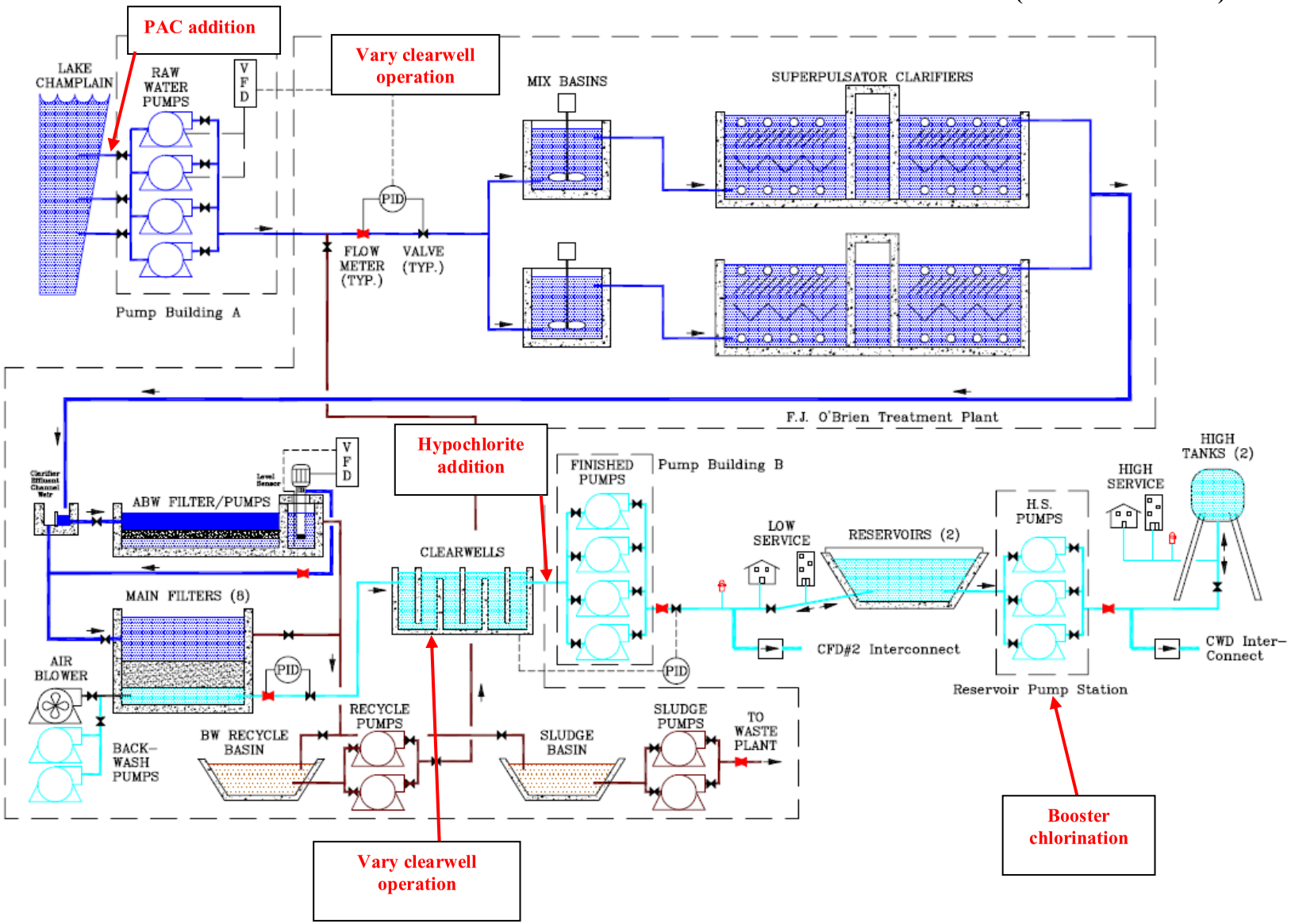


FIGURE 5-10D
BURLINGTON PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.2.3.10 Catamount-Bolton - Existing System

The treatment summary for the existing system is provided below and in Table 5-15. Figure 5-13A provides a schematic of existing treatment. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source water is East Joiner Brook, an upland mountain stream which is very low in alkalinity and variable in pH. Bolton also has a permit to take water from West Joiner Brook. Typically, the water quality in West Joiner Brook is of poorer quality and is only used during periods when East Joiner Brook has low flows. There is only one intake at each source, which is a slide gate control system.

Treatment: The existing system is a Waterboy filter unit with flocculation, tube settlers and mixed media filtration. Currently the flocculation paddle as been removed due to no visible floc being formed. The operator indicates flocculation is difficult to occur and takes a long time (hours). The cationic polymer is added to the head of the WTF. The filters do not exhibit any pressure loss during operation (typically a 24 hour run). Even though no floc is visible, raw water TOC of 3.5 to 3.0 is reduced to 2.5 to 2.0 based on limited sampling set or about 30 percent reduction of TOC.

The building is in good condition as wells as the filter housing. The filter steel tank requires repainting. The electrical components and controls are at the end of their useful life and require replacement.

The chemical feed system and filter control is manually performed. There is no flow pacing as there is no flowmeter. There is no SCADA system and as such there is no link to the water levels to the distribution tank which would normally control WTF operation.

The system provides reliable treatment, but requires substantial operator attention. The WTF has difficulty meeting turbidity requirements during rain events when the turbidity in the brook increases. Typical operation during these events is to de-rate the flow through the facility or turn off the treatment facility until the event passes.

Distribution: The system is a small system with a small distance of pipe. There is a 150,000 gallon tank in the distribution system which maintains system pressure. The system is flushed once per year. There are no consecutive systems serviced by Bolton.

Operator: Two full time operators run the WTF, but the WTF is not always manned. The WTF runs 5 to 6 days a week, 24 hours a day.

VT DEC Observations: Turbidity is low, except under storm conditions. Because of raw water quality, the coagulant dose results in direct filtration so that the flocculator and tube settlers are not utilized as designed. Pre-chlorination is needed to get adequate disinfection prior to the first customer. The system is currently drilling wells and hopes to abandon the surface water treatment plant.

AECOM Observations: The main issue with this facility is that prechlorination is required which creates a large amount of disinfection byproducts as this maximizes the contact time between TOC and chlorine.

TABLE 5-15
CATAMOUNT-BOLTON SYSTEM SUMMARY

Population			
Value	Typically ~ 700	As high as 1,000	
System Demand, MGD			
	Typical		
Minimum	---		
Average	0.036		
Maximum	0.040		
Distribution System Characteristics			
Parameter	**Value**		
Miles of Pipe	1		
-Estimated average diameter, inches	80% of pipe is 8 inch DI 20% of pipe is 6 inch DI		
Approximate System Detention Time	Estimated at 4 days		
Number of Storage Tanks	1		
-Provide Size of each, MG	0.15		
-Tank Type	Rectangular concrete in ground, 12 feet deep		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry**
Prechlorination	Yes		
Preoxidation	12.5% Sodium Hypochlorite	4 as free chlorine residual	Dry
Coagulant	Superfloc 573 Cationic polymer	0.7 to 5.0	Wet
Disinfection Chlorination	12.5% Sodium Hypochlorite	Target a 1.0 residual (ranges from 0.7 to 1.5)	Dry
Corrosion Control	Phosphate	1.8 to 3.0	Dry

5.2.3.11 Catamount-Bolton - Proposed Alternatives

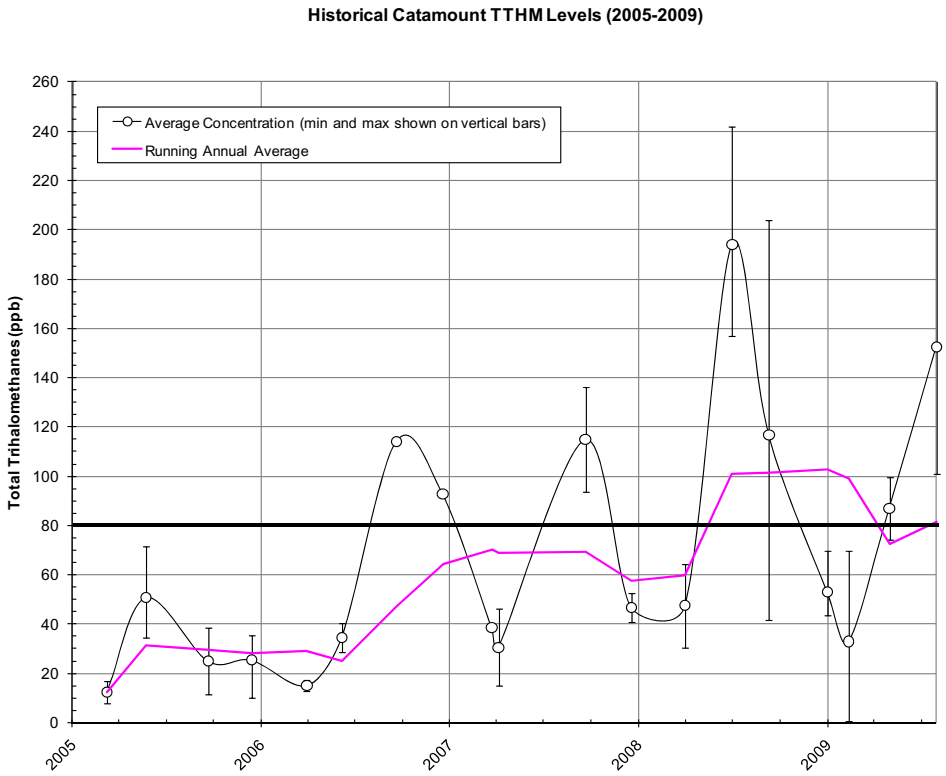
AECOM recommends that Catamount-Bolton follow the steps provided in Table 5-16 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-11 and 5-12 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Catamount-Bolton will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Catamount-Bolton. The summer DBP level for calibration was 107 ppb for TTHM and 102 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 8 ppb, HAA5 = 80 ppb
2. UV and reduced chlorine: TTHM = 10 ppb, HAA5 = 8 ppb
3. UV and post clearwell chloramination: TTHM = 15 ppb, HAA5 = 11 ppb

For Chlorine as the Residual Disinfectant (Figure 5-13B)

1. Install UV disinfection. This will discontinue the practice of prechlorination ahead the filters and allow for an alternative preoxidant (if needed). This is a major reason the DBPs found in the distribution system are high.
2. Optimize the coagulant for TOC removal during filtration. This will likely require higher coagulant levels or alternative coagulant schemes. This will also require that the flocculation paddle be reinstalled to optimize floc formation. Increased coagulant dosing will increase the solids going to waste and possibly volume of water used for backwashing. Increased coagulation dosing also would reduce the pH of the treated water and make disinfection more effective (see Table 5-2 and Table 5-3), thus reducing the time required for disinfection (or alternatively the amount of chlorine required). A caustic dosing point would be required for pH adjustment for corrosion control.



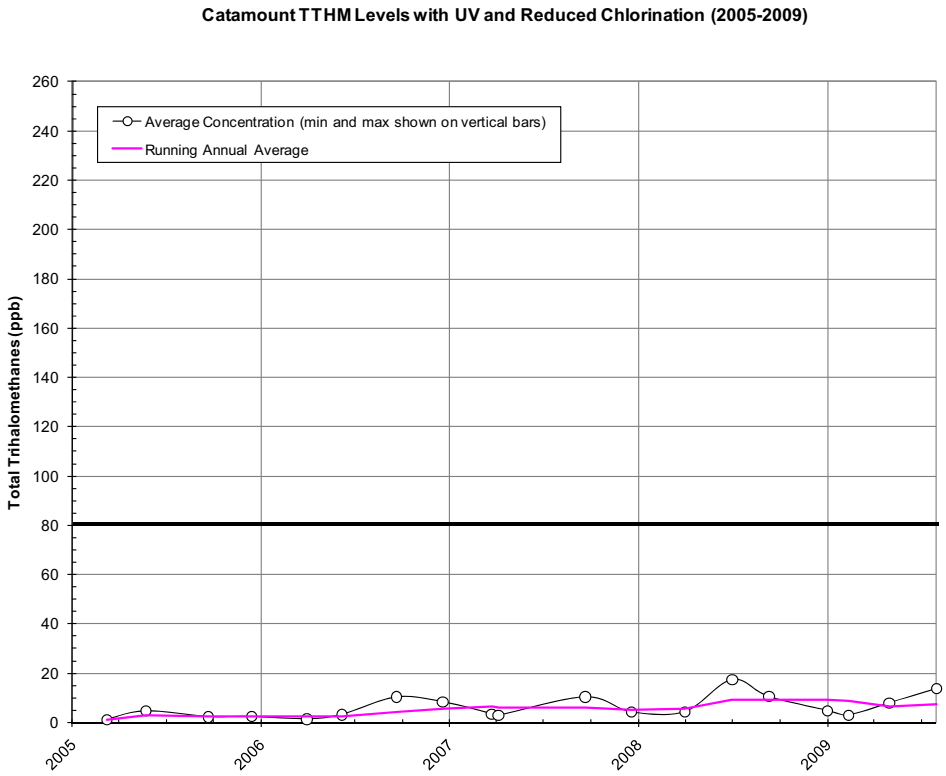
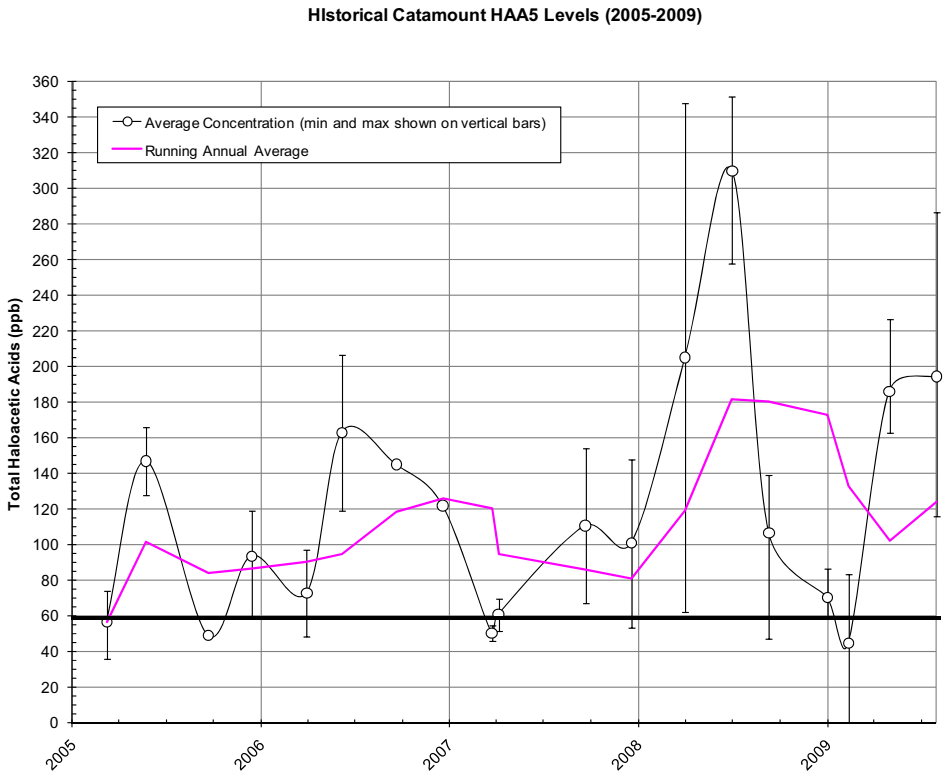


FIGURE 5-11
CATAMOUNT TTHM LEVELS (2006-2009)



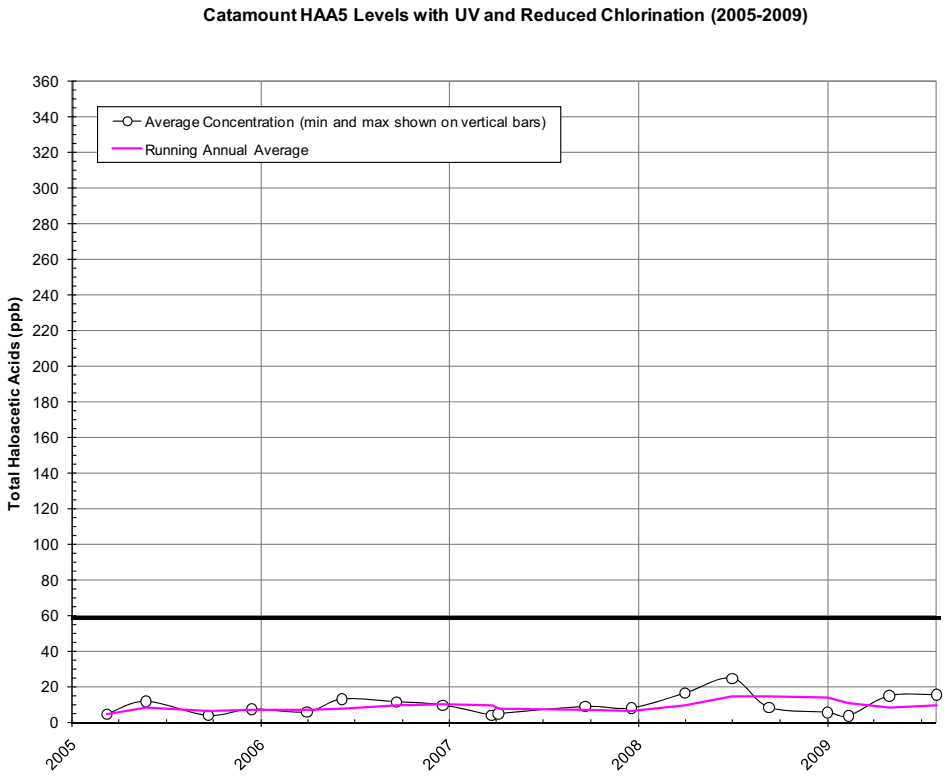


FIGURE 5-12
CATAMOUNT HAA5 LEVELS (2006-2009)

a. Automate the WTF operation. Currently, there is no connection between demand and WTF operation as well as internal automation such as chemical feed. Providing the automation will make the facility more efficient in chemical dosing and will prevent chlorine over or under dosing and allow the plant to optimize chemical usage. In summary, this option would update the valve operators and electrical equipment to provide true unmanned operation.

For Chloramine as the Residual Disinfectant (Figure 5-13C)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.
2. A UV system installed after the filters will decrease the amount of chlorine required for disinfection at WTF and will reduce the DBPs formed.

TABLE 5-16
PROPOSED ALTERNATIVES FOR CATAMOUNT - BOLTON

Treatment- Existing System	Distribution
Chlorine	
1.) Install UV to stop prechlorination for disinfection	None proposed
2.) Optimize coagulation to remove TOC. Install flocculator paddle	
Chloramine	
1.) Install UV to reduce chlorine for disinfection	1.) Chloramine

5.2.3.12 Catamount-Bolton - Other DBP Reduction Alternatives (Figure 5-13D)

A summary of other DBP reduction alternatives for Catamount-Bolton is provided in Table 5-17. The following provides a brief discussion of each alternative.

1. Since the system contains several dead ends, the utility should consider looping the system so that water has an opportunity to circulate and not sit at a dead end. If this is not possible, then automatic flushing systems should be considered at the dead end to reduce the age of the water. Flushing water however makes the distribution less efficient as water that has been treated is not consumed.
2. Replace chlorine as residual disinfectant with chlorine dioxide. Being a small system, the use of chlorine dioxide could be a good alternative residual disinfectant to chlorine. More information on benefits and drawbacks to chlorine dioxide can be found in Section 4.3.2.4:Chlorine Dioxide (Preoxidation) and Section 4.4.4:Chlorine Dioxide (Implementation).
3. The system is currently exploring if a groundwater source can supplement or replace the current source, Joiner Brook. If the wells can replace the surface source water then the requirement for disinfection may be reduced from 1.0 log Giardia to 4.0 log virus inactivation (or less if approved by VT DEC). If the wells provide good quality water and the TOC is low and relatively unreactive with chlorine, the system should have an opportunity to maintain compliance with the DBPR. However, the treatment facility may be required to remove other contaminants which are common to groundwater.
4. The system can implement a use of a GAC absorber to remove TOC after the sand filters.

TABLE 5-17
OTHER DBP REDUCTION OPTIONS FOR CATAMOUNT-BOLTON

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Seasonal use of GAC	Medium or High	1.) Loop system and/or add automatic flushing device at dead end	Low or Medium
		2.) Change to chlorine dioxide as residual disinfectant	High
Replacement of existing surface WTF			
1.) Develop groundwater source	Ranges	None proposed	

*Effect is the anticipated relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-13A
CATAMOUNT-BOLTON PROCESS EXISTING FLOW DIAGRAM

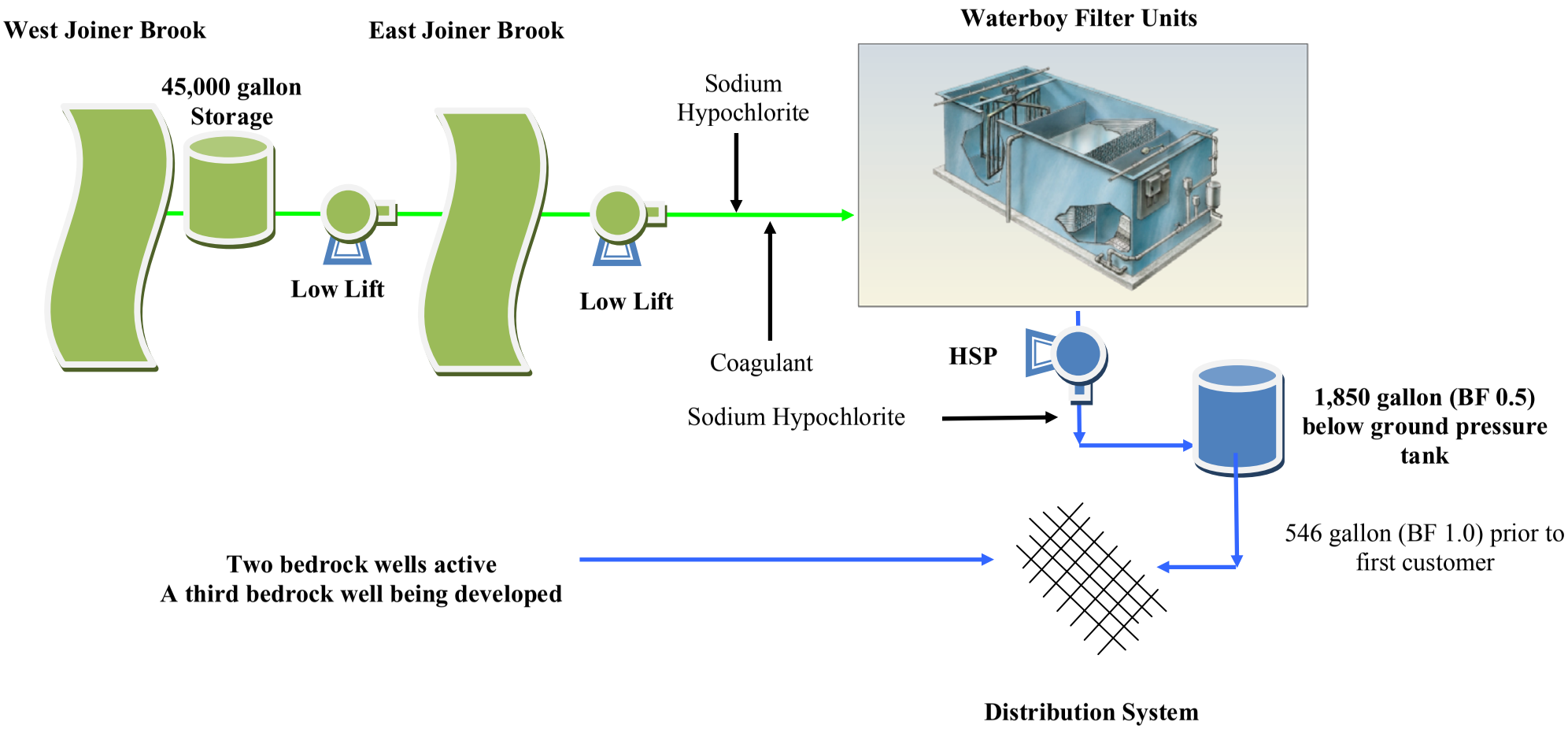


FIGURE 5-13B
CATAMOUNT-BOLTON PROCESS PROPOSED FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

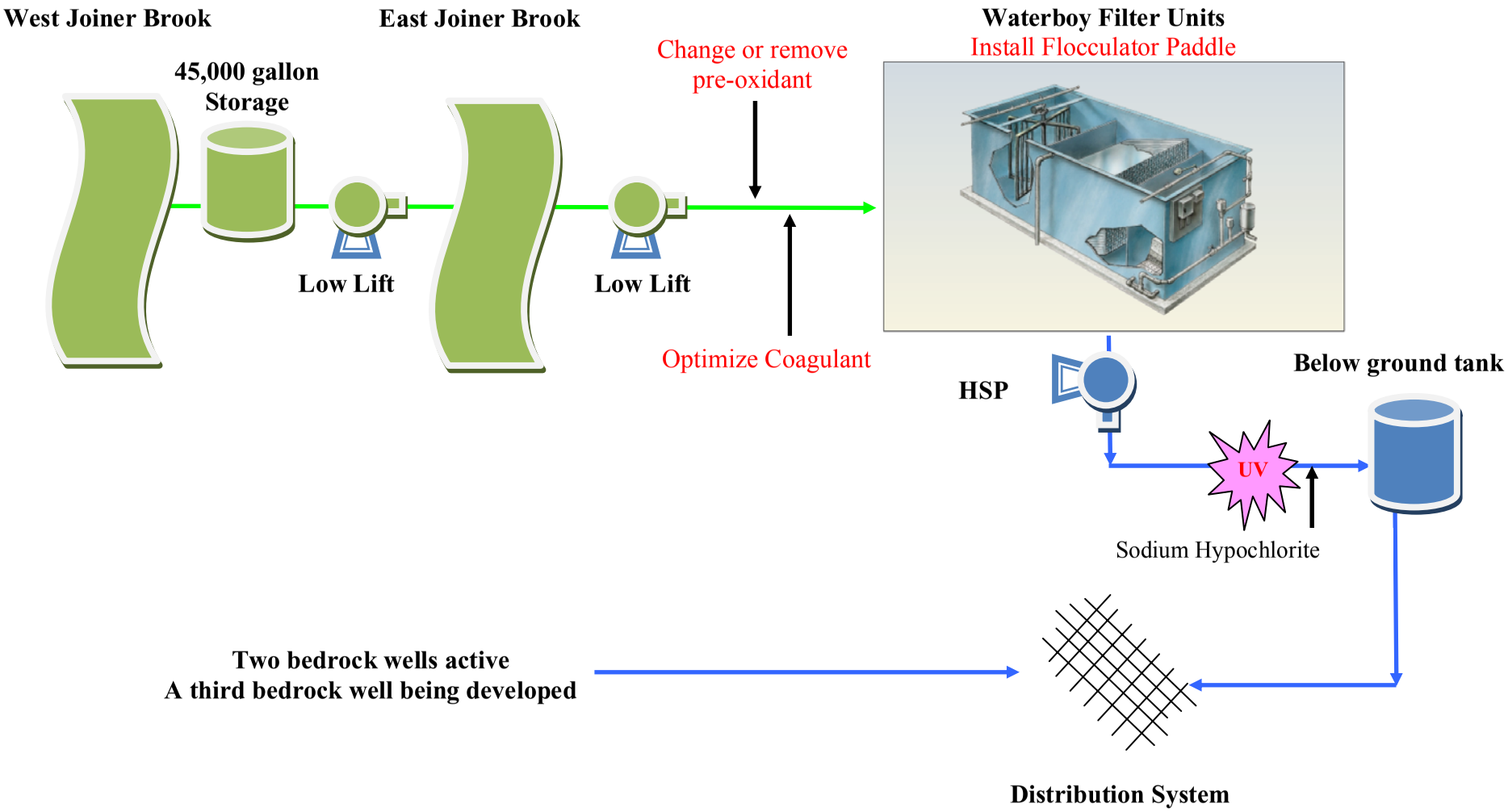


FIGURE 5-13C
CATAMOUNT-BOLTON PROCESS PROPOSED FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

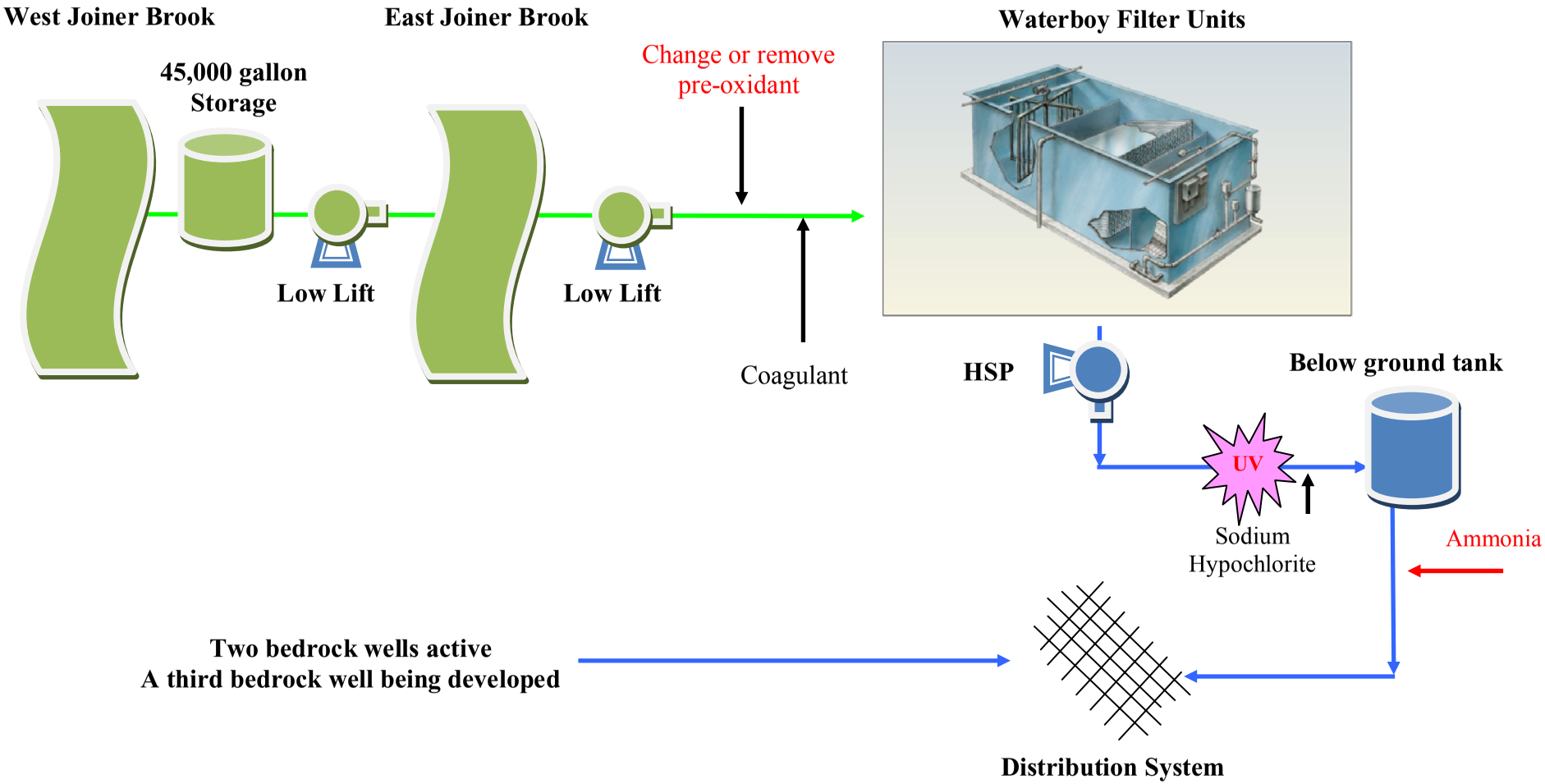
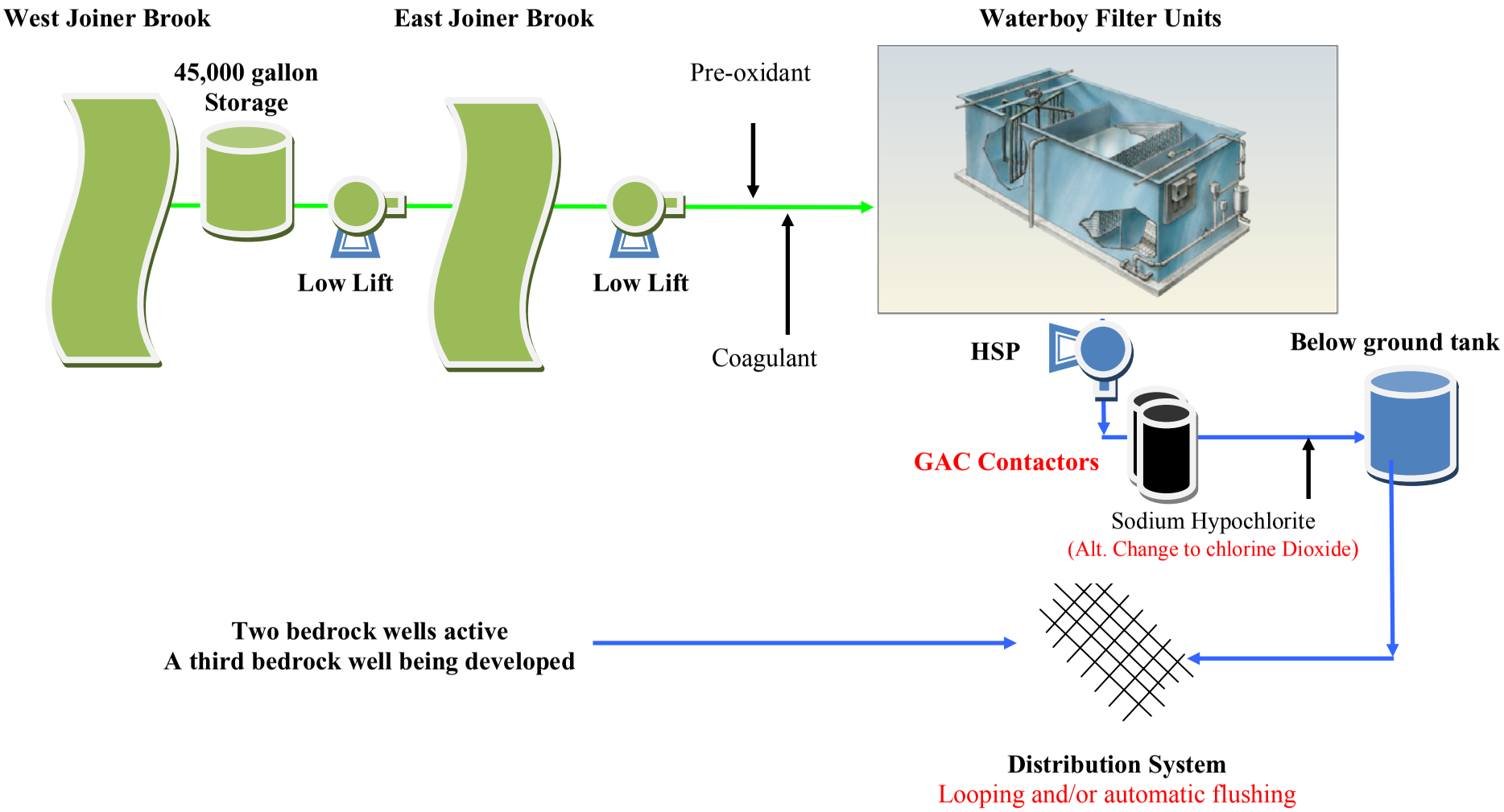


FIGURE 5-13D
CATAMOUNT-BOLTON PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.2.3.13 Champlain Water District - Existing System

The treatment summary for the existing system is provided below and in Table 5-18. Figure 5-14A provides a schematic of existing treatment. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source for District is Lake Champlain. Two intakes are located near the bottom of the Lake and referred to as the North and South intake. The intakes are 10 feet off of the bottom of the underwater canyon in Shelburne Bay in 75 feet of water. Champlain Water District also performs periodic watershed surveys to view the condition of the watershed.

Treatment: The current treatment consists of buoyant media clarification followed by deep bed tri-media filtration. Water is pumped from the Edward Blake Lake Water Pump Station to two onsite 0.5 MG lake (raw) water storage tanks. Permanganate is dosed at the intakes for Zebra Mussel control as well as a pre-oxidant. Hypochlorite can be used as a backup to the permanganate, but more typically is dosed as pre-oxidant prior to the coagulant addition. Water from the lake storage tanks flows by gravity to buoyant media clarifiers. Alum is typically injected prior to the lake storage tanks. Alum as well as a polymer also are typically added prior to the buoyant media clarification and is mixed by using a high pressure jet of water which is injected next to the alum (~10 gpm of water at a pressure roughly 80 psig above line pressure) to provide rapid mix.

The buoyant media clarifier provides the necessary flocculation and partial particle removal. The filters remove the remaining particles and provide the necessary turbidity requirements. TOC removal is good generally removing about 40% of raw TOC. Finished TOC values typically range from 1.5 mg C/L to 2.0 mg C/L. CWD has an “Excellences in Water Treatment” award under the Partnership for Safe Water.

TABLE 5-18
CHAMPLAIN WATER DISTRICT SYSTEM SUMMARY

Population			
Value	68,923		
System Demand, MGD			
Note	Firm WTP capacity is 22MGD Typical Range 8 to 14 MGD		
	October, 2007	July, 2009	
Minimum	9.02	8.67	
Average	9.55	9.31	
Maximum	10.64	10.26	
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	555 plus	55 transmission, 500+ served distribution systems	
-Estimated average diameter, inches	10 inch		
Approximate System Detention Time	15 days	25 days at consecutive system	
Number of Storage Tanks	19		
-Provide Size of each, MG	See Table 5-19		
-Tank Type	1 Elevated, 1 In Ground, 17 Standpipes		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	Yes		
Preoxidation 1	Potassium Permanganate	0.33 to 0.35	Dry
Preoxidation 2	12.5% Sodium Hypochlorite	0.24 to 0.45 (Target trace chlorine prior to clarifier	Dry
Polymer	PolyDADMAC (Praestol I86KH)	Typical: 1.8 to 2.0 Range: 1.8 to 3.3	Wet
Coagulant	48.5% Alum	Typical: 5.5 Range: 3.5 to 7.5	Dry

TABLE 5-18 (Continued)
CHAMPLAIN WATER DISTRICT SYSTEM SUMMARY

Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Primary Disinfectant	12.5% Sodium Hypochlorite	Dose 1.8 to 2.0 Typ. Free Cl_2 Residual 0.92 -0.94 Range Free Cl_2 Residual 0.85 to 1.3	Dry
Distribution Disinfectant 1	12.5% Sodium Hypochlorite	Dose 1.13 to 1.84 Range Total Cl_2 Residual 1.5 to 2.3	Dry
Distribution Disinfectant 2	40% Ammonium Sulfate (4:1 to 4.5:1 chlorine to NH3 ratio)	0.33 to 0.575 as N	Dry
Corrosion Control	Zinc Orthophosphate	0.9 to1.8 Dist. Residual close to dose	Dry as PO_4
Corrosion Control	Sodium Hydroxide	3.6 to 5.4 Target pH 7.6 to 7.8	Dry

Filtered water enters a wetwell where low lift pumps send the water to a 1.0 MG onsite baffled storage tank used to obtain disinfection compliance. Sodium hypochlorite is dosed prior to entering the tank. Finished water out of the disinfection tank is feed to the high service pumps which the water is dosed with a sodium hypochlorite trim.

Ammonia sulfate is also dosed at a 4 (chlorine) to 1 (ammonia as N) to 4.5:1 ratio to form mainly monochloramine to maintain a disinfection residual in the distribution system. Sodium hydroxide is dosed to a target pH of 7.6 to 7.8 for corrosion control as well as zinc orthophosphate.

Distribution System: There is over 555 miles of distribution pipe with nineteen (19) distribution tanks. There are 23 consecutive systems ranging in population size of 37 to 13,920. The District has been targeting a TTHM level goal of less than 20 ppb due to a recent meta-analysis followed by a study of increased risk of birth defects (Hwang et al, 2008).

Operator: There are five (5) full time operators with four (4) back up water quality personnel that provide vacation operator coverage as well as extensive other responsibilities. The WTF is always manned 24 hours per day. The WTF runs seven (7) days a week. The WTF has an updated SCADA system as well has the following online instrumentation:

1. Turbidity
2. Total organic carbon
3. Orthophosphate
4. Ammonia
5. pH
6. Total Chlorine

VT DEC Observations: None

AECOM Observations: The main challenge within each of Champlain Water District's served municipal systems is long water age within the piping infrastructure.

TABLE 5-19
CHAMPLAIN WATER DISTRICT'S SYSTEM TANKS

Tank	Total Capacity (gallons)
Lake Water Tank	500,000
Lake Water Tank	500,00
Filtered Water Tank	1,000,000
So. Burl. East Tank	2,100,000
So. Burl. West Tank (2)	1,000,000
Will. East Tank	300,000
Will. West Tank	500,000
Will. South Tank	282,000
Essex East Tank	680,000
Essex West Tank	2,200,000
Essex North Tank	500,000
Essex South Tank	200,000
Col. East Tank	1,700,000
Jericho Tank (3 cells)	240,000 (80,000/cell)
Milton Tank (2)	1,150,000 (0.50 mg and 0.65 mg)
Owned by Served Systems	
Shel. North Tank	500,000
Shel. South Tank	500,000
Wake Robin in Shelburne	628,000

FIGURE 5-14A
CHAMPLAIN WATER DISTRICT EXISTING PROCESS FLOW DIAGRAM

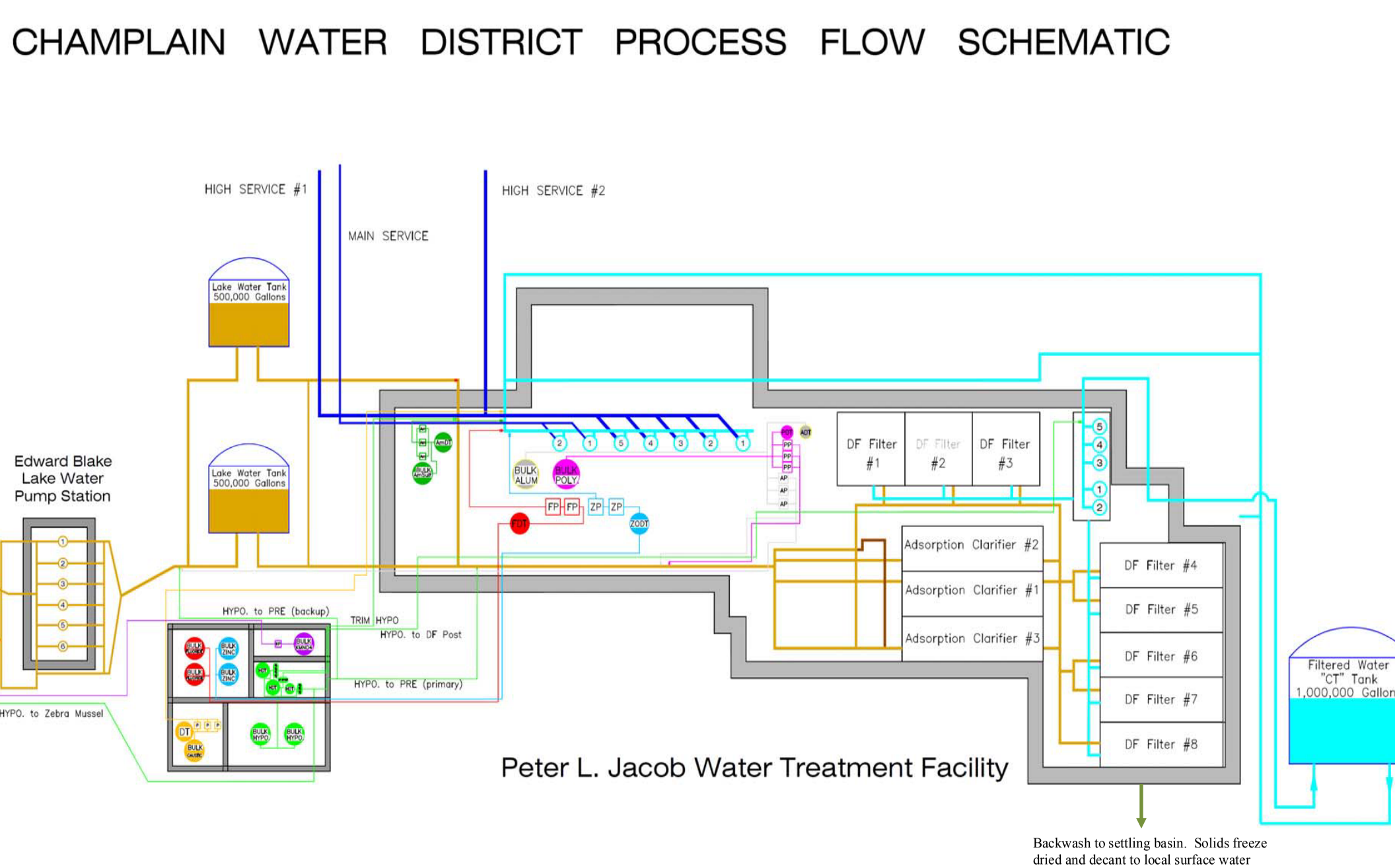
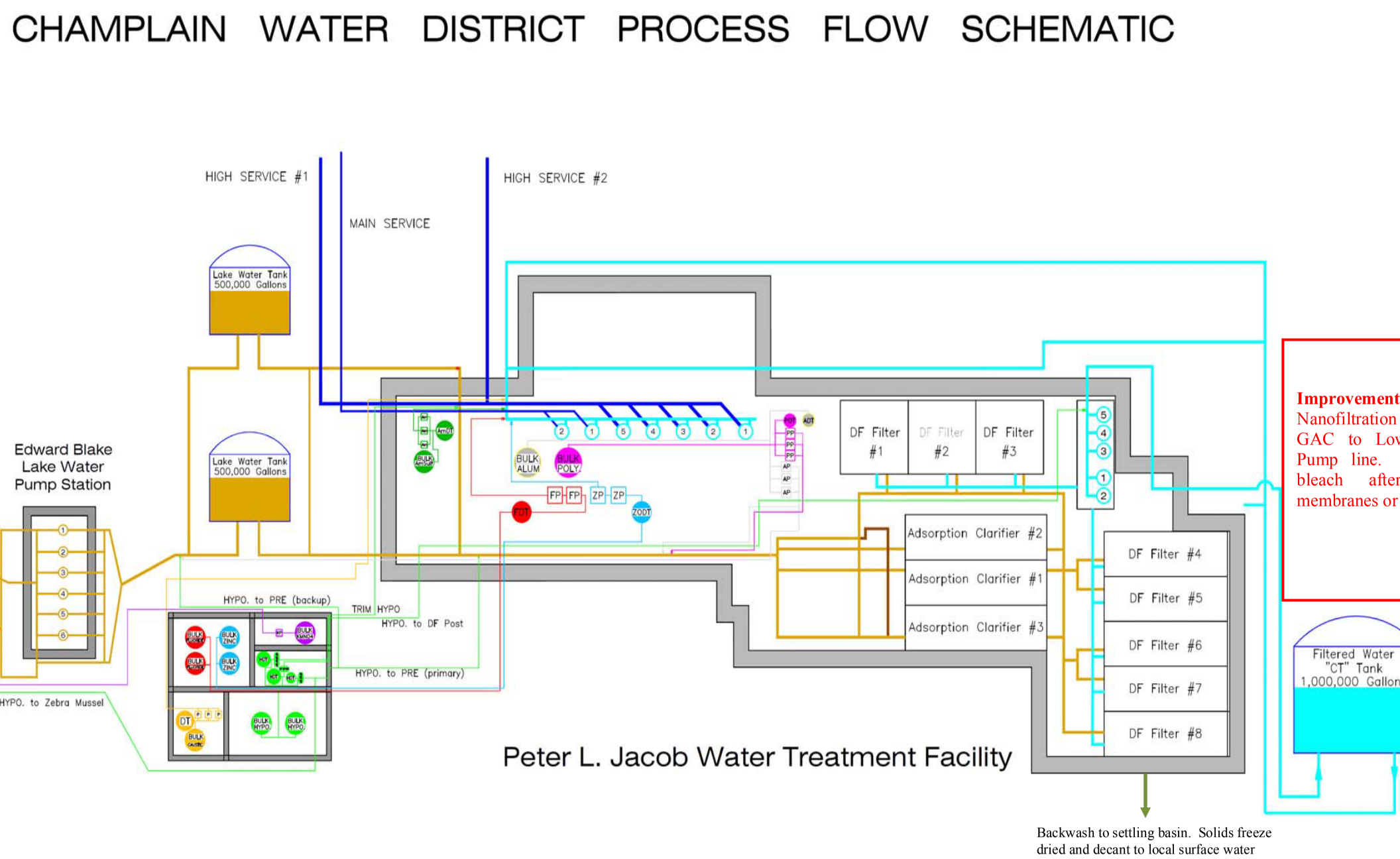


FIGURE 5-14B
CHAMPLAIN WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)



5.2.3.14 Champlain Water District - Proposed Alternatives

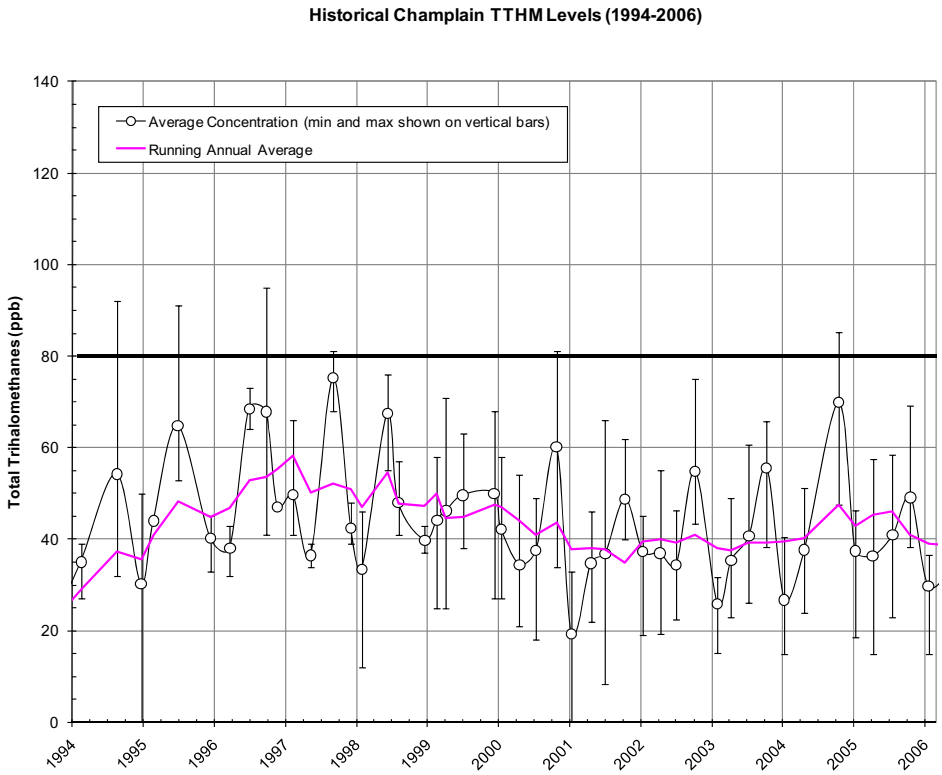
The Champlain Water District (CWD) has optimized the WTF to provide high quality water, not only meeting regulations, but also meeting the goals of AWWA's Partnership for Safe Water. Due to the optimization process, the District has already explored or implemented many options that are available to them. The following discussion provides a summary of the technologies considered and final recommendation.

Figures 5-15 through 5-19 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Champlain Water District will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. Since the CWD has been using chloramine since 2006 as a secondary residual, the DBP levels for chloramine are observed in Figure 5-19 for Champlain Water District. DBP data prior to 2006 was based on chlorine as secondary residual.

Chlorine as Main Distribution Disinfectant (Figure 5-14B)

In reviewing the historical DBP levels (Figures 5-15 through 5-19) for CWD, it becomes apparent that the District was having difficulty in meeting the required MCL for HAA5 of 60 ppb. Thus using chlorine alone as a residual disinfectant without treatment at the source is not possible. In order for CWD to return to using chlorine as the residual disinfectant in the distribution system, the CWD would be required to remove most of the TOC from the source water. This would require a treatment upgrade. AECOM explored the technologies listed in Chapter 4 and narrowed down the options to the following:

1. GAC - after the existing filters
2. MIEX - prefiltration
3. Nanofiltration - after the existing filters



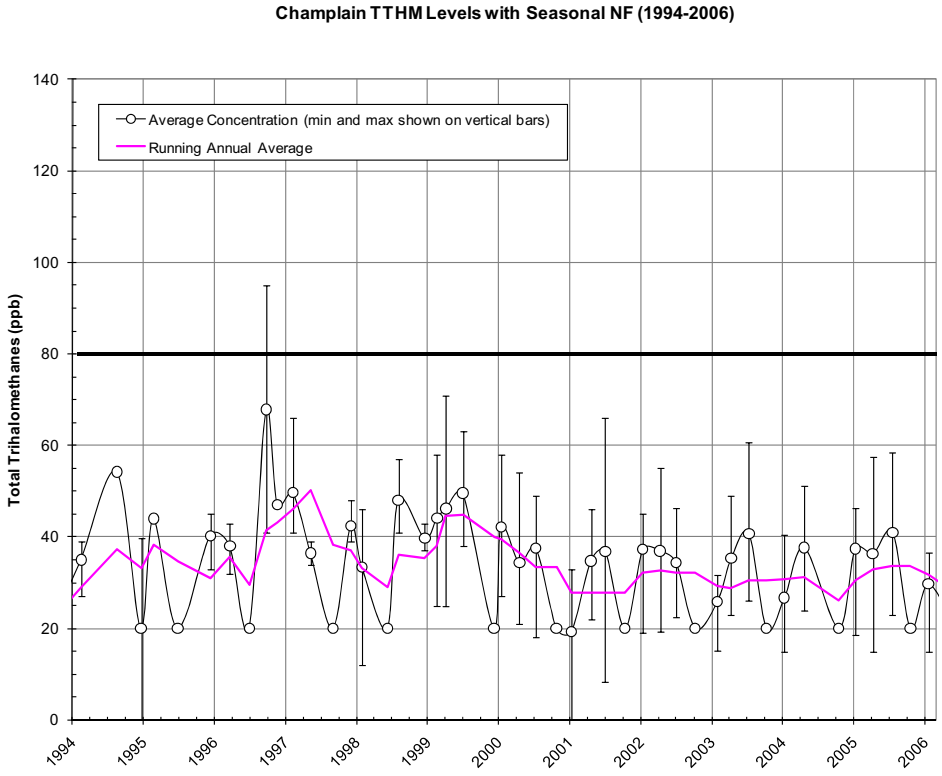
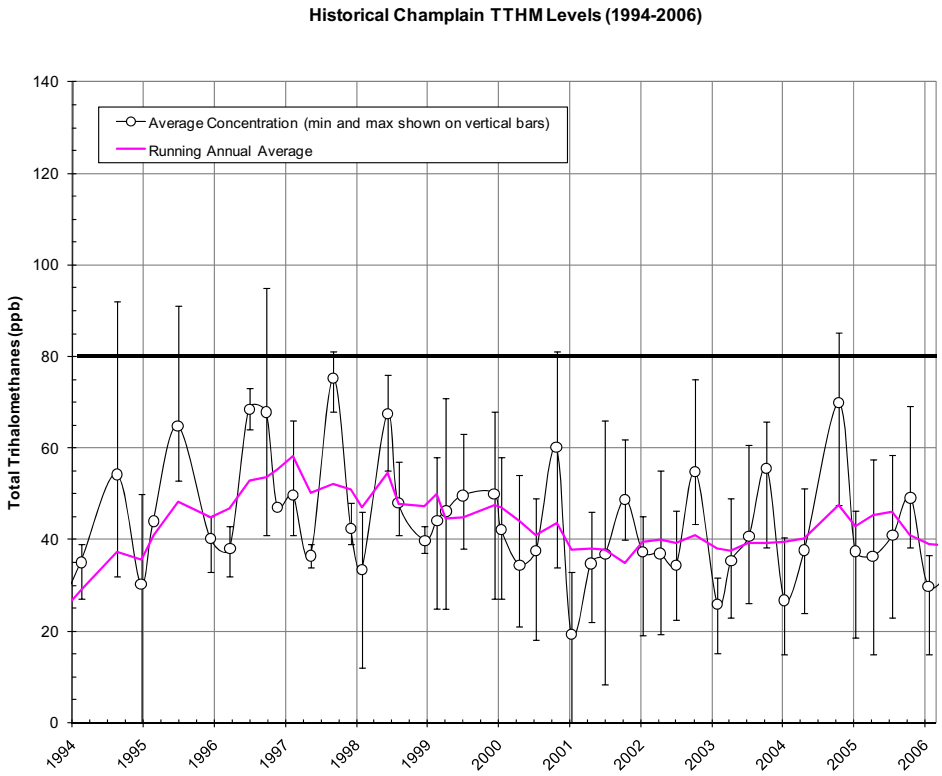


FIGURE 5-15
CWD TTHM LEVELS NF ALTERNATIVE (1994-2006)



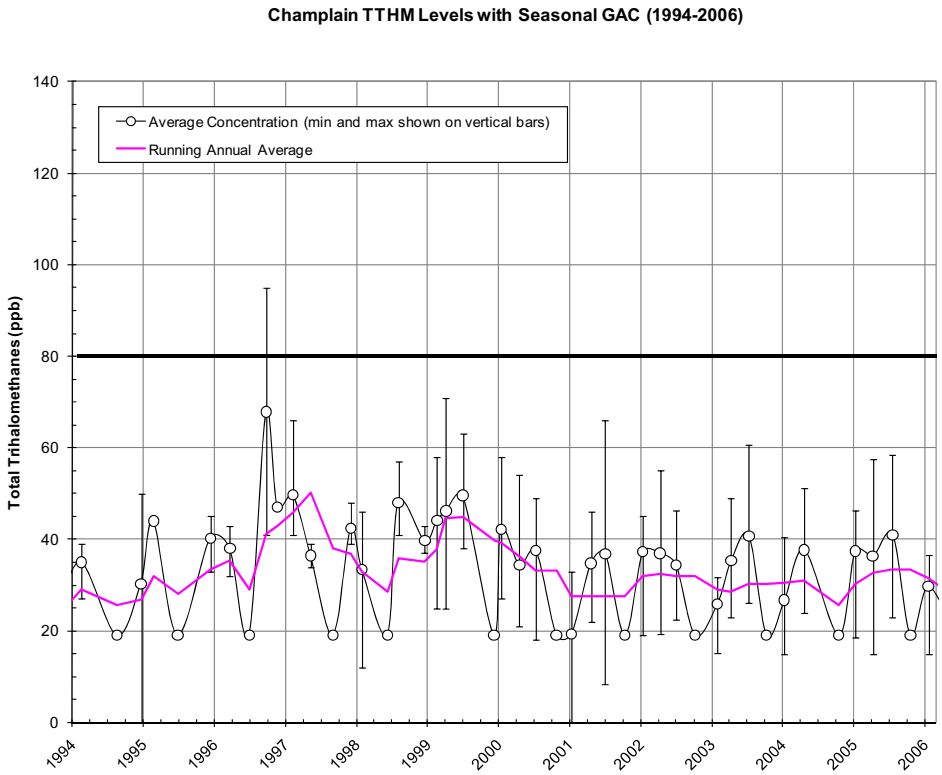
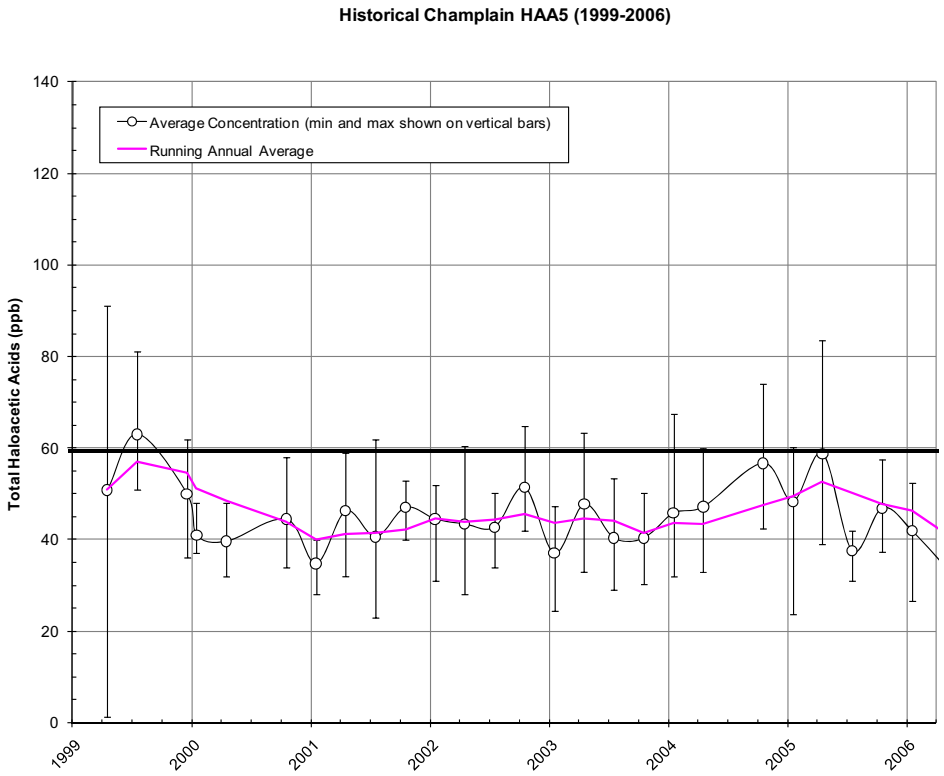


FIGURE 5-16
CWD TTHM LEVELS GAC ALTERNATIVE (1994-2006)



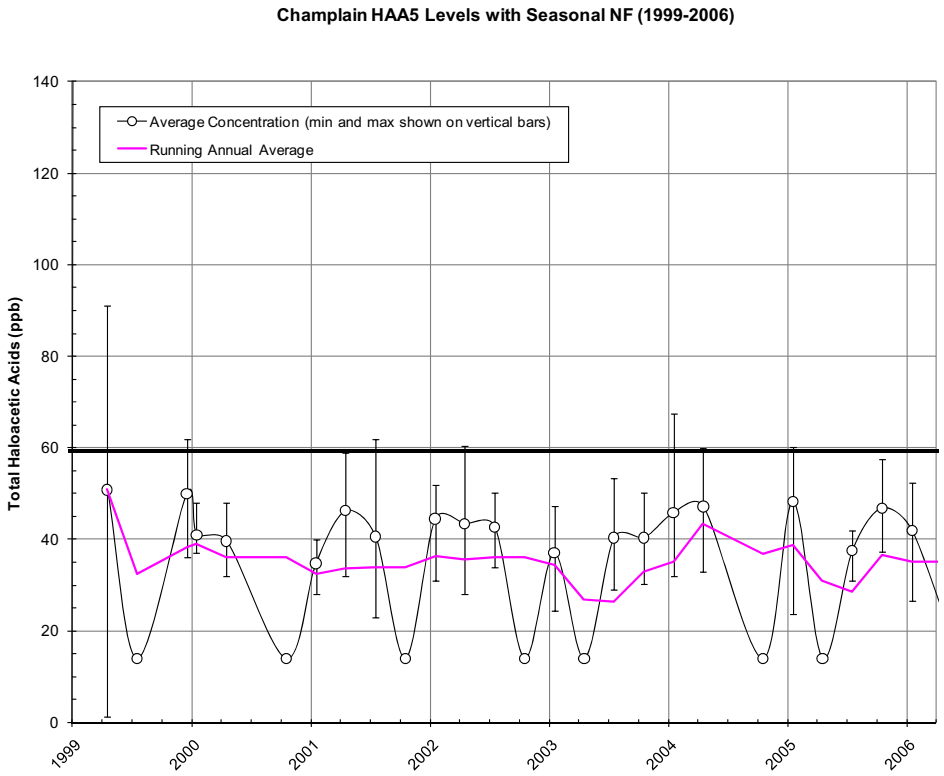
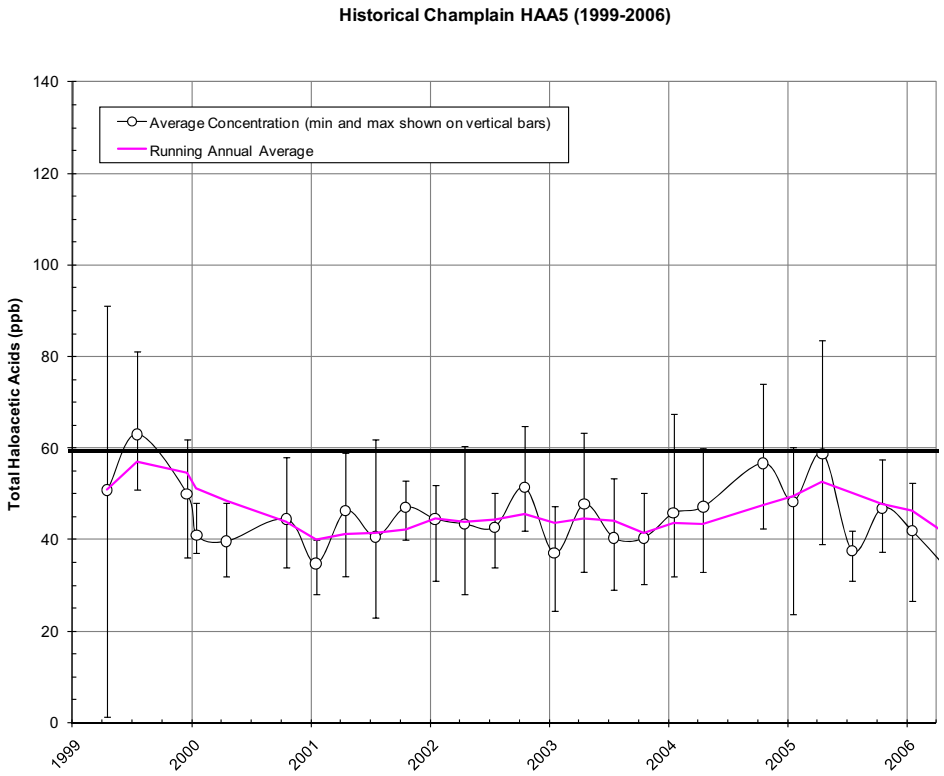


FIGURE 5-17
CWD HAA5 LEVELS NF ALTERNATIVE (1994-2006)



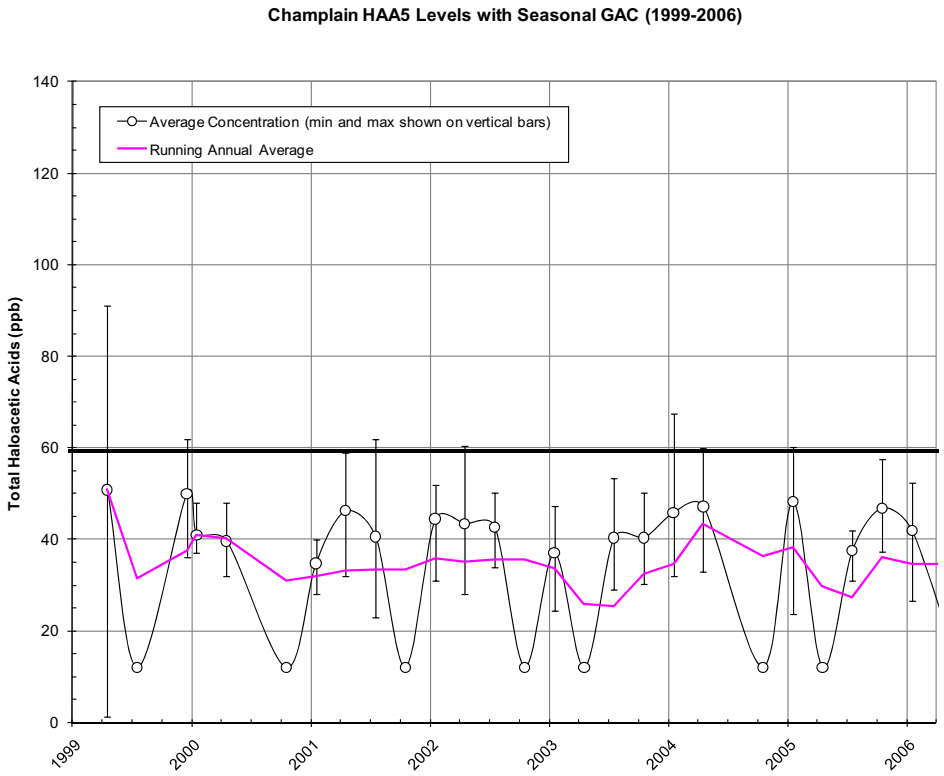
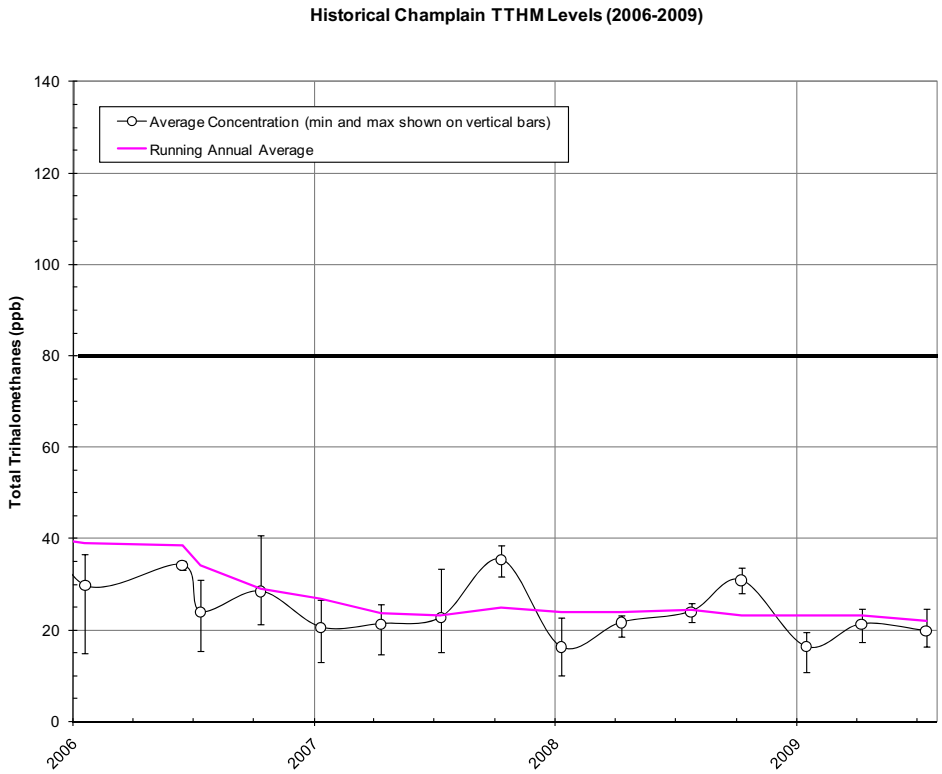


FIGURE 5-18
CWD HAA5 LEVELS GAC ALTERNATIVE (1994-2006)



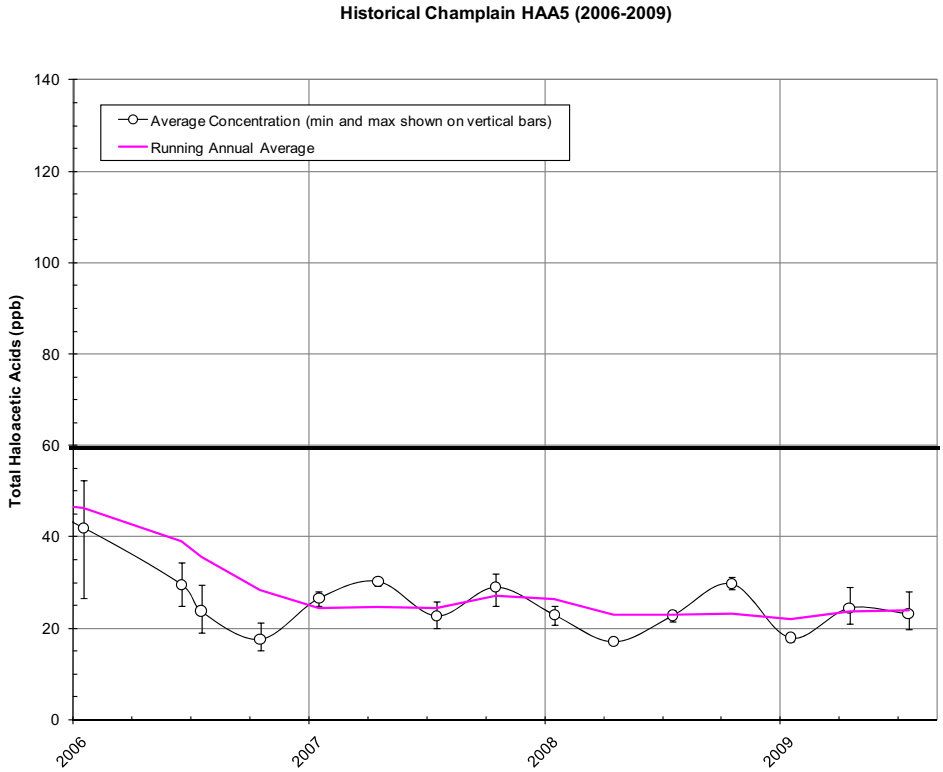
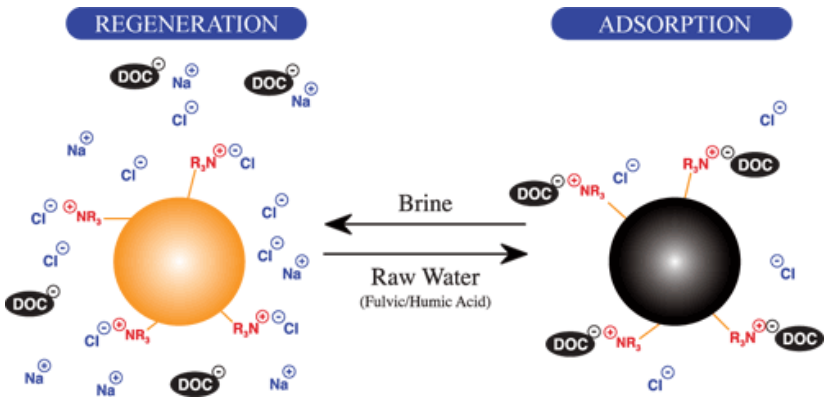


FIGURE 5-19
CWD TTHM AND HAA5 LEVELS WITH CHLORAMINE (2006-2009)

FIGURE 5-20

SCHEMATIC OF MIEX RESIN



Source: http://www.miexresin.com

MIEX was not considered as long term viable technology for CWD. MIEX is a resin containing nitrogen functional groups which exhibit a positive charge. The positive charge attracts negatively charged compounds, specially dissolved organic carbon. Once the resin is exhausted, the resin can be regenerated by sodium from a salt solution (much like a home softening device).

The following concerned exist with MIEX as a viable technology for CWD:

1. Historical resins of this nature tended to have issues with NDMA formation as the resin degrades. Orica Limited, the owner of the MIEX technology, uses a styrene based resin which the company claims has eliminated this issue as long as the pre-chlorination residual prior to the MIEX system does not exceed 0.5 mg Cl_2/L. The company has tested the resin at chlorine levels below this and has not detected NDMA in the treated water systems to date. Since CWD would like to have the ability to prechlorinate at the intake, it was felt this would be in conflict with minimizing NDMA formation.
2. Due to the limited options for residuals disposal, the removal of the salt brine was considered problematic for this site.

Thus, the best available technology to obtain high TOC removals for CWD would GAC absorbers or nanofiltration, each after the existing sand filters. Table 5-20 outlines some of the concerns with each technology as well as the benefits. Lastly, with the system reverting back to chlorine as the residual disinfectant, there will be a need to optimize the chlorine dose to ensure compliance with other regulations as mentioned previously in this chapter which includes possible booster chlorination. Table 5-21 provides the summary of the recommended options.

TABLE 5-20
COMPARISON OF GAC AND NANOFILTRATION FOR CWD

GAC		Nanofiltration (Membrane)	
Benefits	Concerns	Benefits	Concerns
Low electrical usage	High material (carbon) usage	Low material (membrane) usage	High electrical usage
High TOC removal* through surface exchange process	Carbon tariffs - Effects on carbon unit costs	High TOC removal* via a an absolute barrier	Disposal of chemicals required for cleaning membranes
No chemicals required for cleaning	Chromatographic peaking and coordinating changes outs for low TOC levels	Pathogen credits which lower chlorine usage for disinfection	Corrosion control in distribution system
Lower waste water volumes	Mining of carbon		Acceptance of alternative certificates to NSF
	Cross contamination of carbon with other carbon not for potable use.		Knowledge of byproducts which may form from membrane degradation unknown
	Bacterial growth		Impacts of variable filtered water quality on membrane operation
			Neutralize pre-chlorine residual prior to membrane

*Requires pilot testing to confirm

Chloramine as the Distribution Disinfectant

The current system has demonstrated successful implementation of chloramination per regulatory requirements.

TABLE 5-21
PROPOSED ALTERNATIVES FOR CHAMPLAIN WATER DISTRICT

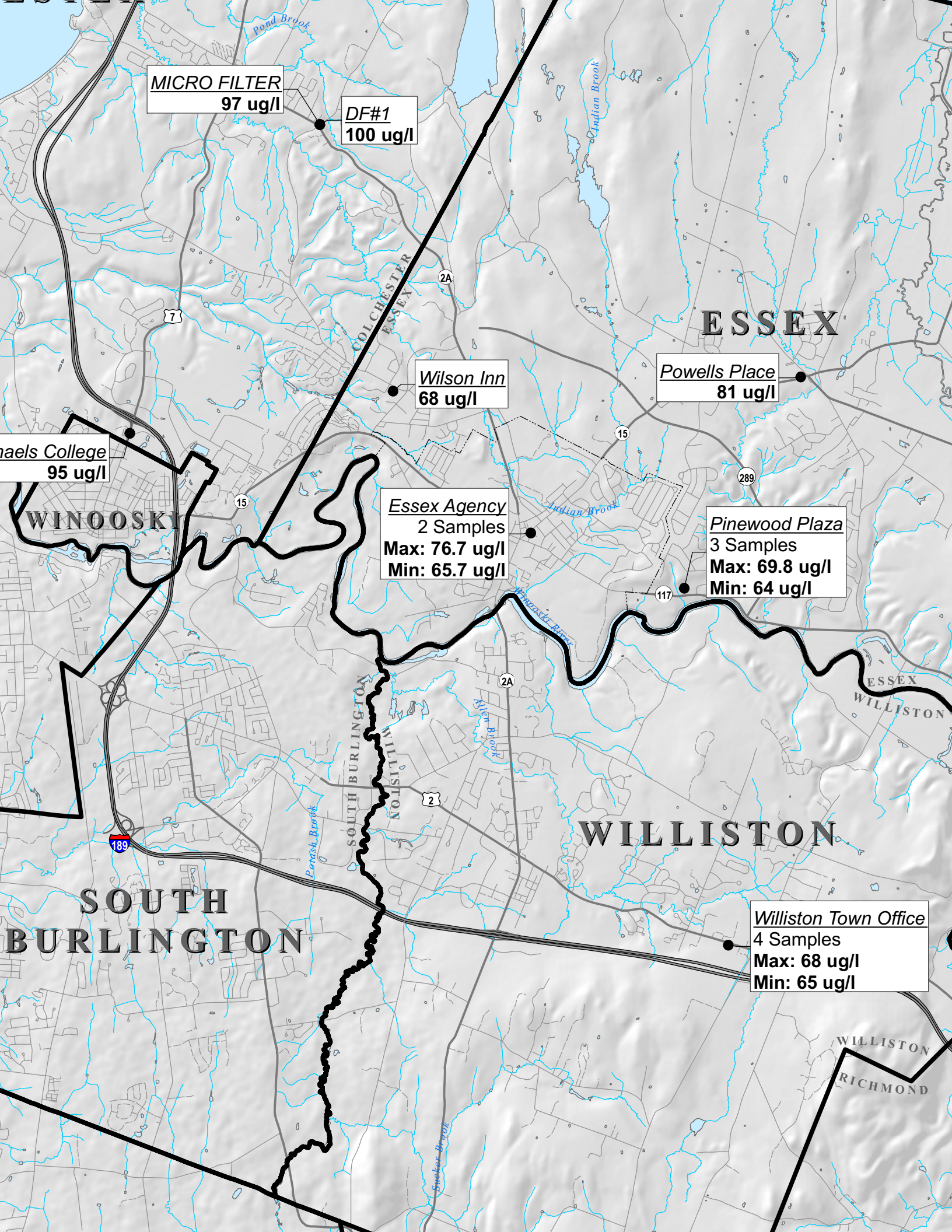
Treatment- Existing System	Distribution
For Chlorine	
1.) Nanofiltration	1.) Booster Chlorination – Located at select consecutive communities*
2.) GAC absorber (20 min EBCT)	
For Chloramine	
None Proposed	1.) Chloramine

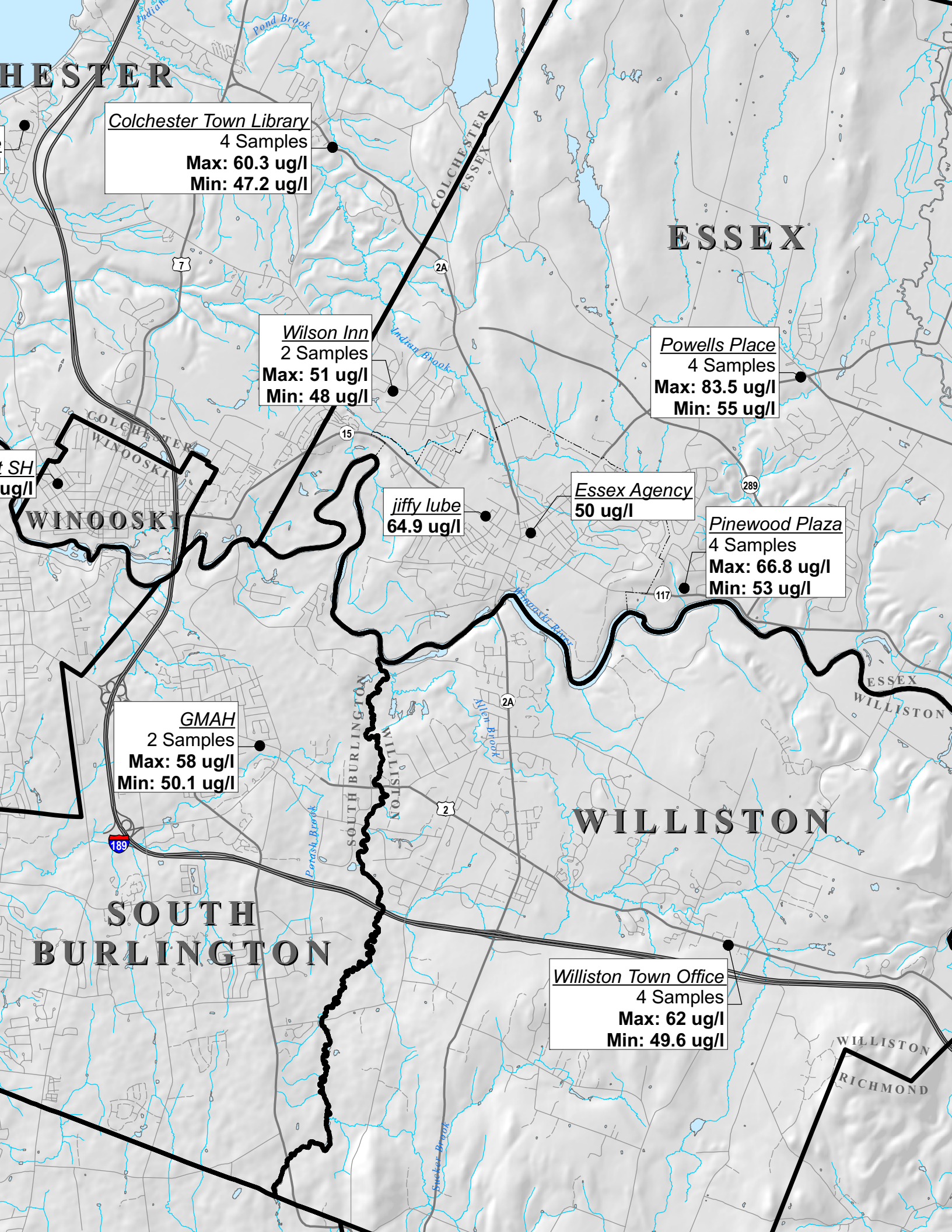
*It is noted that these may be required to ensure a chlorine residual reaches the end of the distribution system. Without pilot testing/full scale testing, it is difficult to determine if these options will be required.

5.2.3.15 Champlain Water District - Other DBP Reduction Options

Table 5-22 provides a summary of the other DBP reductions options. In reviewing the alternatives, AECOM developed Figure 5-21 and Figure 5-22 to show the areas with historical elevated DBP levels while the system was on chlorine. These maps suggest that HAA5 is the major issue facing the District and occurs at elevated levels in several areas of the distribution system. The Champlain Water District has indicated that if the system where to switch back to chlorine, maintaining a chlorine residual in the distribution system would be an issue. The following outline some of the other steps which may be required to address this issue:

1. Automatic flushing at select dead ends which cannot be looped. This may not be acceptable to the public as water which is produced is not consumed. This step would be required to be coupled with the Proposed Alternatives Section above, and is not considered a standalone alternative.
2. Since the distribution system is long, it is difficult for water which has been treated at the WTF with free chlorine to reach the ends of the distribution system. A more decentralized approach which looks for opportunities to


MICRO FILTER
97 ug/l
DF#1
100 ug/l
Wilson Inn
68 ug/l
Powells Place
81 ug/l
naels College
95 ug/l
Essex Agency
2 Samples
Max: 76.7 ug/l
Min: 65.7 ug/l
Pinewood Plaza
3 Samples
Max: 69.8 ug/l
Min: 64 ug/l
Williston Town Office
4 Samples
Max: 68 ug/l
Min: 65 ug/l
ESSEX
WINOOSKI
WILLISTON
SOUTH BURLINGTON
COLCHESTER
ESSEX
SOUTH BURLINGTON
WILLISTON
ESSEX
WILLISTON
WILLISTON
RICHMOND
Pond Brook
Indian Brook
Indian Brook
Winooski River
Allen Brook
Potash Brook
Sucker Brook
2A
7
15
289
117
2
189


HESTER
Colchester Town Library
4 Samples
Max: 60.3 ug/l
Min: 47.2 ug/l
Pond Brook
COLCHESTER
ESSEX
ESSEX
Wilson Inn
2 Samples
Max: 51 ug/l
Min: 48 ug/l
Indian Brook
Powells Place
4 Samples
Max: 83.5 ug/l
Min: 55 ug/l
COLCHESTER
WINOOSKI
WINOOSKI
jiffy lube
64.9 ug/l
Essex Agency
50 ug/l
Pinewood Plaza
4 Samples
Max: 66.8 ug/l
Min: 53 ug/l
Winooski River
ESSEX
WILLISTON
GMAH
2 Samples
Max: 58 ug/l
Min: 50.1 ug/l
SOUTH BURLINGTON
WILLISTON
Allen Brook
WILLISTON
Pondy Brook
SOUTH
BURLINGTON
Williston Town Office
4 Samples
Max: 62 ug/l
Min: 49.6 ug/l
WILLISTON
RICHMOND
Sucker Brook

develop wells in those systems which are further away from the WTF as discussed in Section 4.6 – Well Sources. This would allow an opportunity for the effective residence time to be reduced as well as provide water with lower TOC. The drawback with wells will be the quality and quantity of water which can be produced as well as acquiring the necessary land.

TABLE 5-22
OTHER DBP REDUCTION OPTIONS FOR CHAMPLAIN WATER DISTRICT

Treatment- Existing System		**Distribution**	
For Chlorine			
	Effect*		**Effect***
As discussed in Proposed Alternatives		1.) Automatic flushing during summer (select dead-ends which demand is typically low)	Low
Compliment Existing Surface WTF			
1.) Develop groundwater source	Ranges	As discussed in Proposed Alternatives	

*Effect is the anticipated relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

5.2.3.16 Grand Isle Consolidated Water District - Existing System

The treatment summary for the existing system is provided below and in Table 5-24. Figure 5-25A provides a schematic of existing treatment. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data over Time.

Source: The District has a shared intake to Lake Champlain with a Fish Hatchery. The intake has a deep water intake and a shallow water intake. Typically, the deep water intake is used for the source water.

Treatment: The system is set up so that water is pumped from the Lake and through a Kinetico pressure filtration process to a 300,000 gallon onsite storage tank. From there water flows by gravity into the distribution system. A proprietary polymer is added prior to filtration. The chemical is injected in the raw water pipe and turbulent flow provides mixing. MIOX is used primarily to

generate chlorine to obtain disinfection compliance. The facility has been tracking TOC values in the raw and finished water roughly every two weeks since October 14, 2008. The monitoring results are provided below.

TABLE 5-23
STATISTICS FOR TOC REMOVED THROUGH THE WTF FROM OCTOBER, 2008 TO DECEMBER, 2009

	TOC-Raw, ppm	TOC-Finished, ppm	Percent Removal
Average	2.9	2.2	22%
Median	2.8	2.2	23%
Maximum	4.6	3.6	31%
Minimum	2.3	1.9	0%

Distribution: There is approximately 29 miles of mostly plastic pipe in the distribution system. There is one 150,000 gallon distribution standpipe in the distribution. The operation staff flushes the whole system once a year. Select dead ends are flushed one additional time during the year. There are no consecutive systems serviced.

Operator: There is a part time operator at the WTF. The WTF runs about 8 hours a day in the winter and 12 hours a day in the summer. In both seasons, the operation is seven days a week. The WTF runs based on tank level. The distribution and onsite tanks fluctuate about 25 feet in the summer and about 10 feet in the winter.

VT DEC Observations: None

AECOM Observations: The WTF does not remove sufficient TOC, nor is it designed to do so.

TABLE 5-24
GRAND ISLE CONSOLIDATED WATER DISTRICT SYSTEM SUMMARY

Population			
Value	1,113		
System Demand, MGD			
	January, 2009	May 2009	August 2009
Minimum	---	---	---
Average	0.09	0.13	0.18
Maximum	---	---	---
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	29 - almost all plastic		
-Estimated average diameter, inches	6.5 inches		
Approximate System Detention Time	Not provided		
Number of Storage Tanks	1		
-Provide Size of each, MG	0.15		
-Tank Type	Standpipe		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	No		
Polymer	EC-462 Cationic Blended Polymer (Aluminum blended with DADMAC polymer and/or Polyamines polymer)	2-6	Wet
Distribution Disinfectant and CT	Sodium Hypochlorite as MIOX	1.4 chlorine residual into onsite tank. 0.5 chlorine residual out of tank.	Dry

5.2.3.17 Grand Isle Consolidated Water District - Proposed Alternatives

AECOM recommends that Grand Isle follow the steps provided in Table 5-25 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-23 and 5-24 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Grand Isle Consolidated Water District will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Grand Isle Consolidated Water District. The summer DBP level for calibration was 65 ppb for TTHM and 38 ppb for HAA5. The modeled reduced DBP levels are as follows:

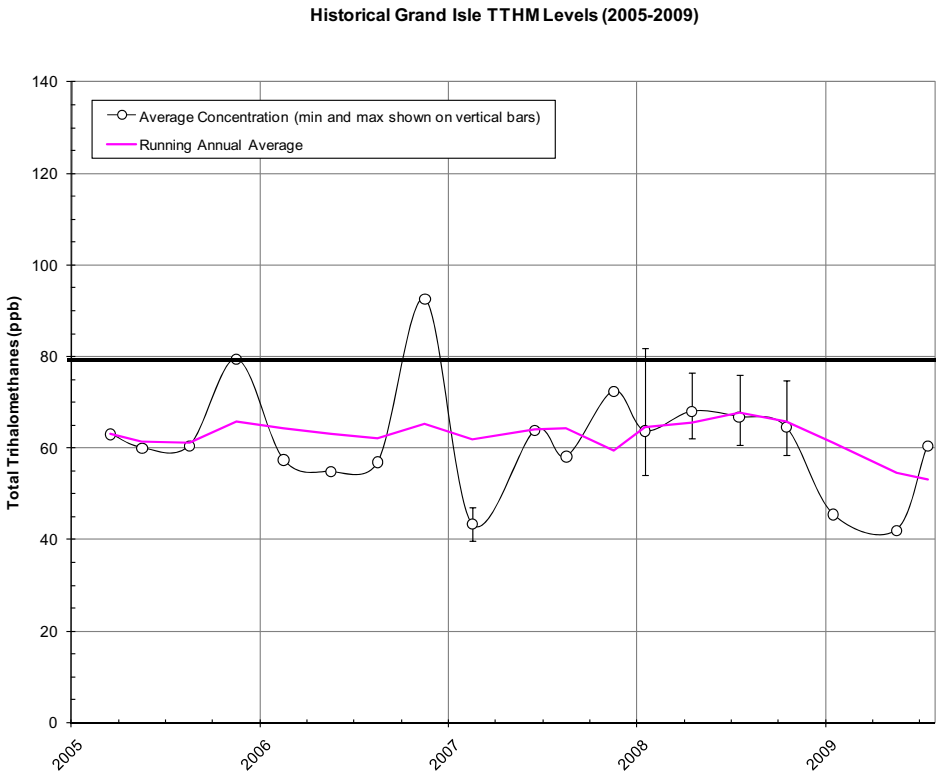
1. Chloramine as secondary residual: TTHM = 60 ppb, HAA5 = 36 ppb
2. UV and post clearwell chloramination: TTHM = 58 ppb, HAA5 = 30 ppb
3. UV and pre clearwell chloramination: TTHM = 9 ppb, HAA5 = 4 ppb

For Chlorine as the Residual Disinfectant (Figure 5-25B)

1. A 20 minute EBCT post filter GAC absorber for TOC removal. The GAC can be used seasonally, during the warmer months to reduce the DBPs which typically spike in the later summer.
2. MIOX should be dosed after on storage tank so that residual required for distribution does not enter on-site storage tank.

For Chloramine as the Residual Disinfectant (Figure 5-25C)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience addition of chloramine will likely reduce the distribution DBPs to below MCLs.



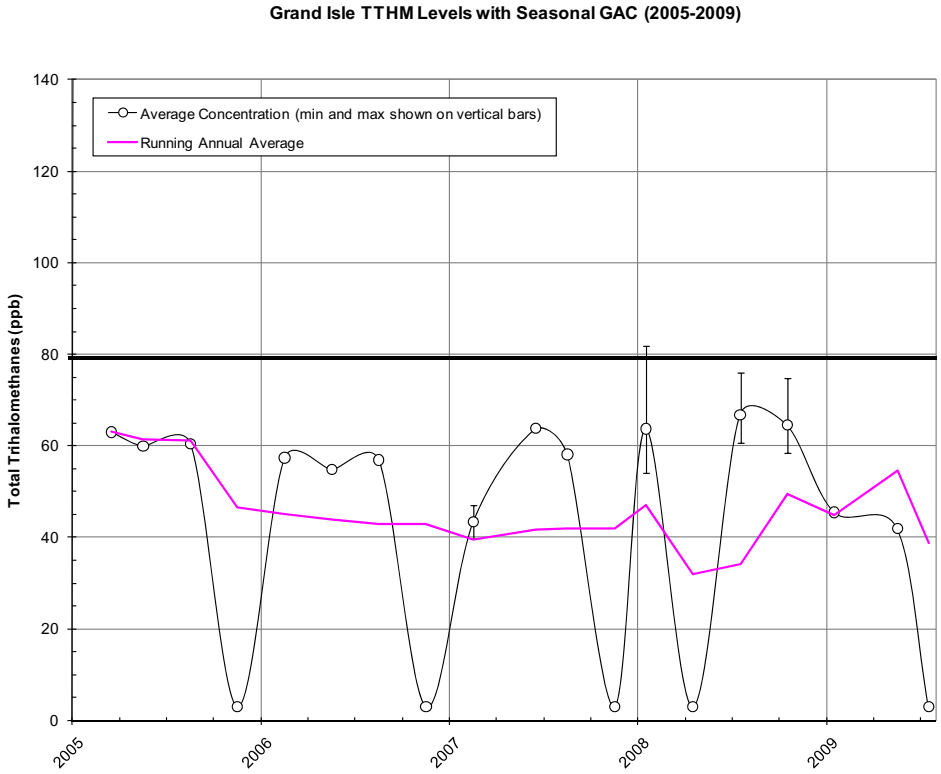
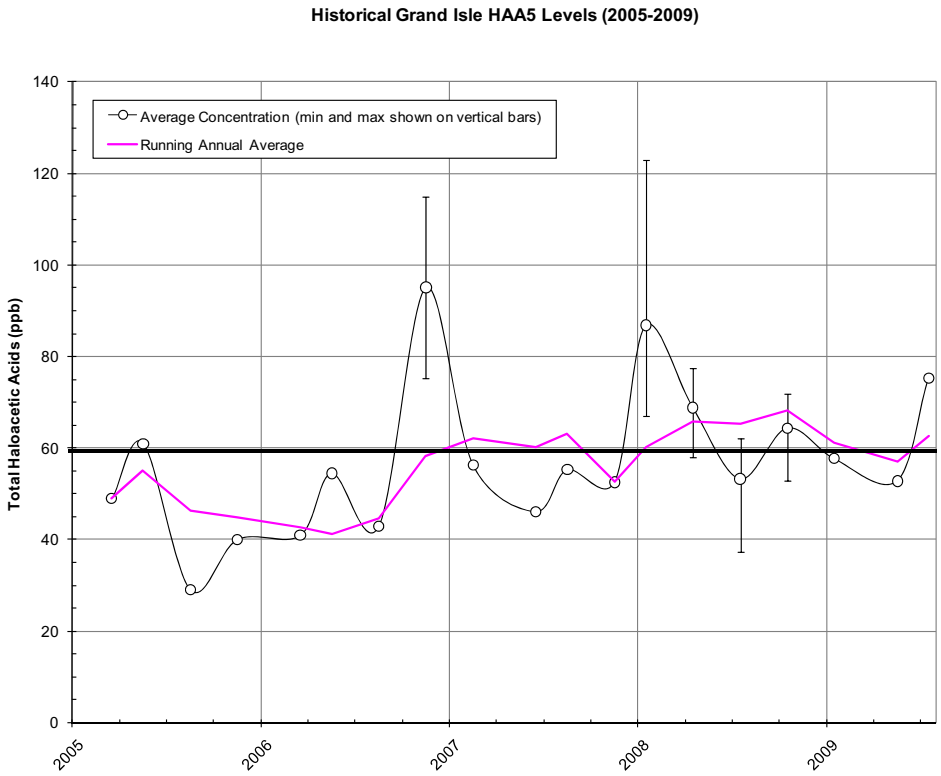


FIGURE 5-23
GRAND ISLE TTHM LEVELS (2006-2009)



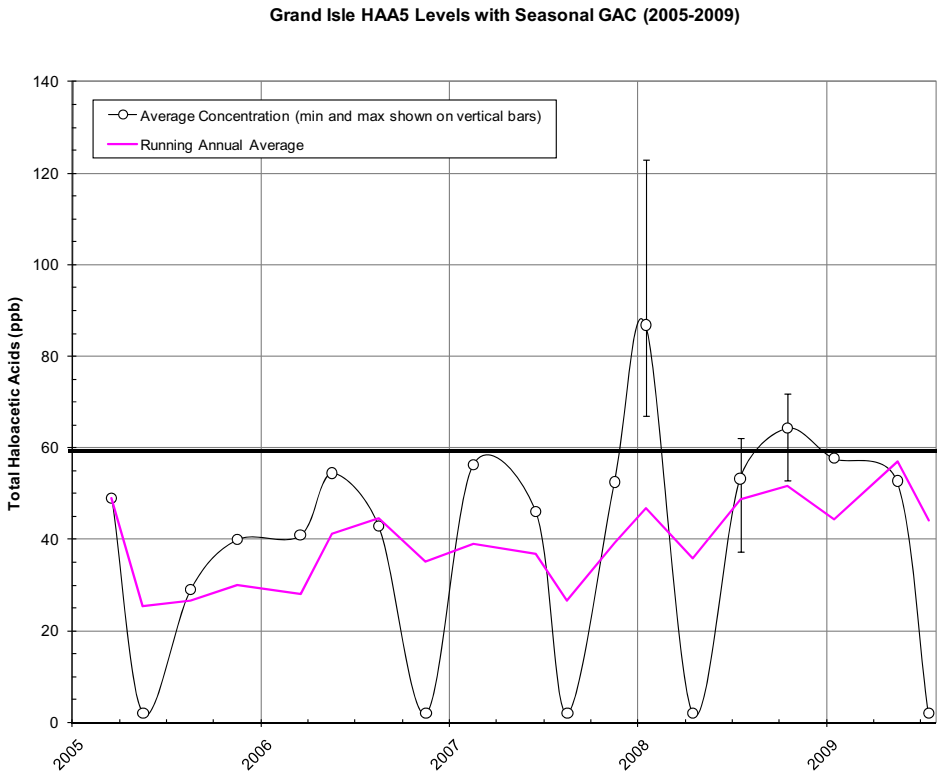


FIGURE 5-24
GRANDE ISLE HAA5 LEVELS (2006-2009)

TABLE 5-25
PROPOSED ALTERNATIVES FOR GRAND ISLE

Treatment- Existing System	Distribution
Chlorine	
1.) GAC post absorber with 20 minute EBCT after Kinetico filters for seasonal use	None proposed
2.) Add MIOX after on-site storage	None proposed
Chloramine	
None proposed	Chloramine

5.2.3.18 Grand Isle Consolidated Water District - Other DBP Reduction Alternatives (Figure 5-25D)

A summary of other DBP reduction alternatives for Grand Isle is provided in Table 5-26. The following provides a brief discussion of each alternative.

1. Add UV disinfection units (if chlorine is chosen distribution residual) to obtain disinfection credits and reduce reliance on chlorine for disinfection.
2. Automatic flushing systems should be considered at the dead end.

TABLE 5-26
OTHER DBP REDUCTION OPTIONS FOR GRAND ISLE

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Install UV to reduce chlorine required for disinfection	Low or Medium	1.) Add automatic flushing device at dead end	Low

*Effect is the relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-25A
GRAND ISLE CONSOLIDATED WATER DISTRICT EXISTING PROCESS FLOW DIAGRAM

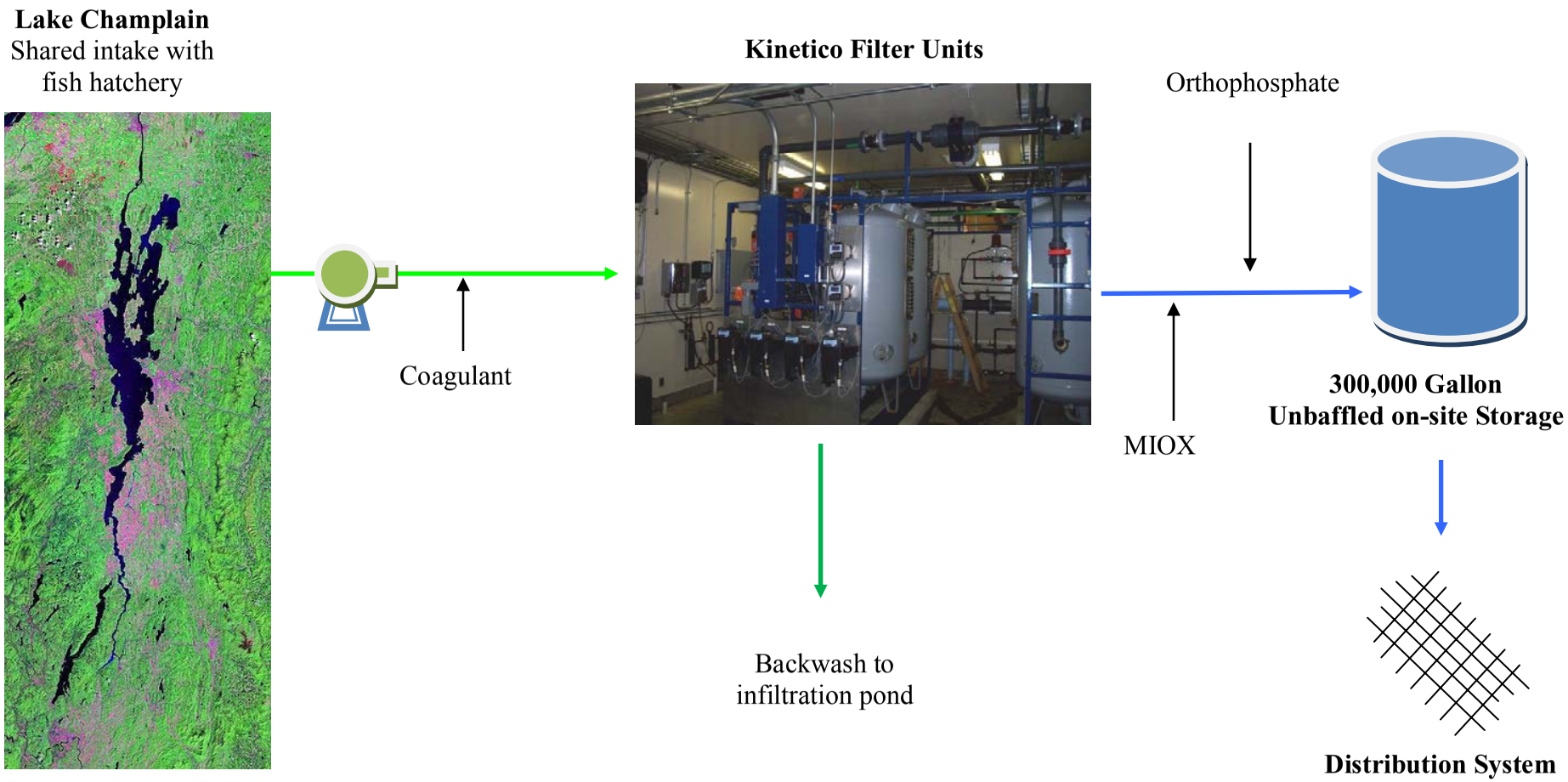


FIGURE 5-25B
GRAND ISLE CONSOLIDATED WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE
(UPDATES IN RED)

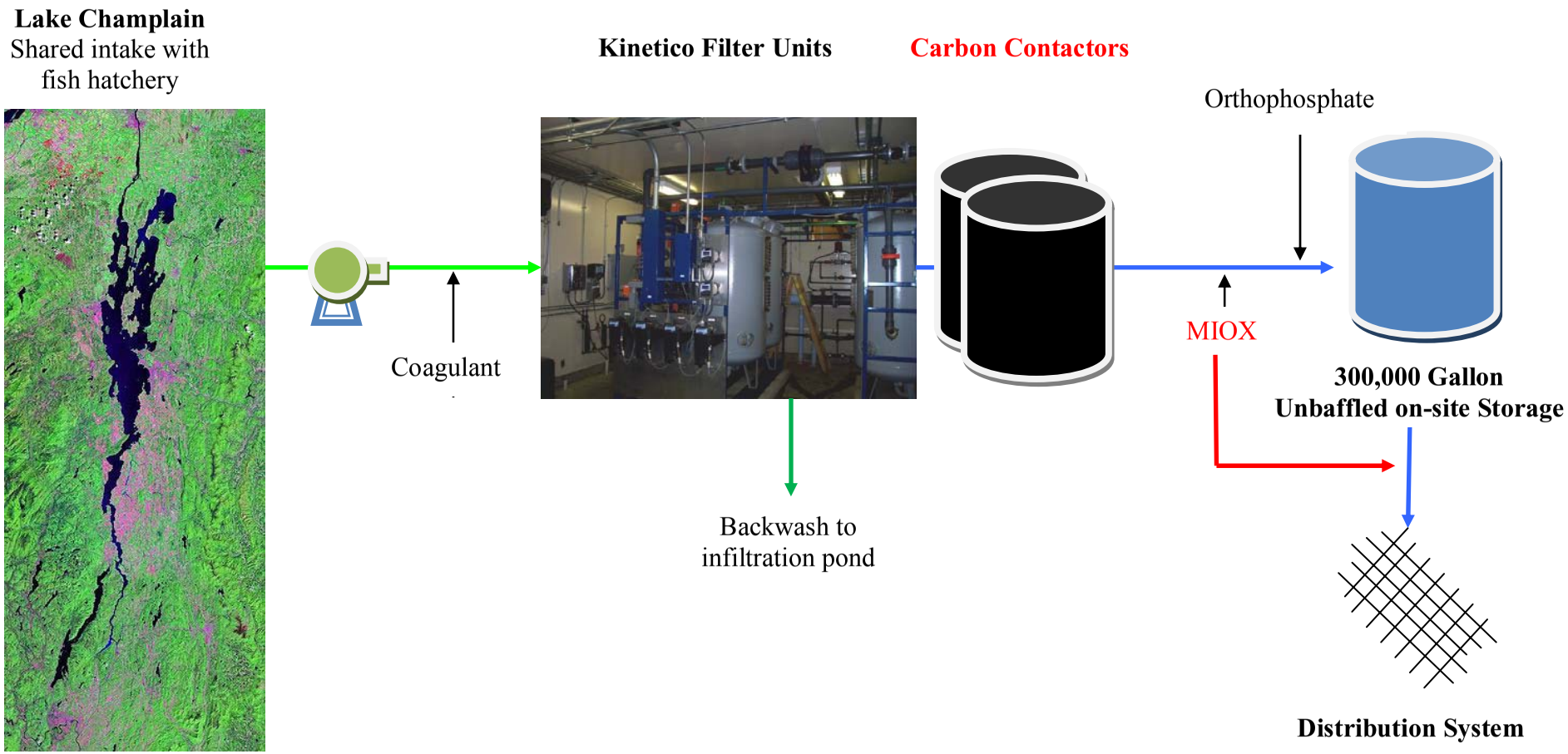


FIGURE 5-25C
GRAND ISLE CONSOLIDATED WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE
(UPDATES IN RED)

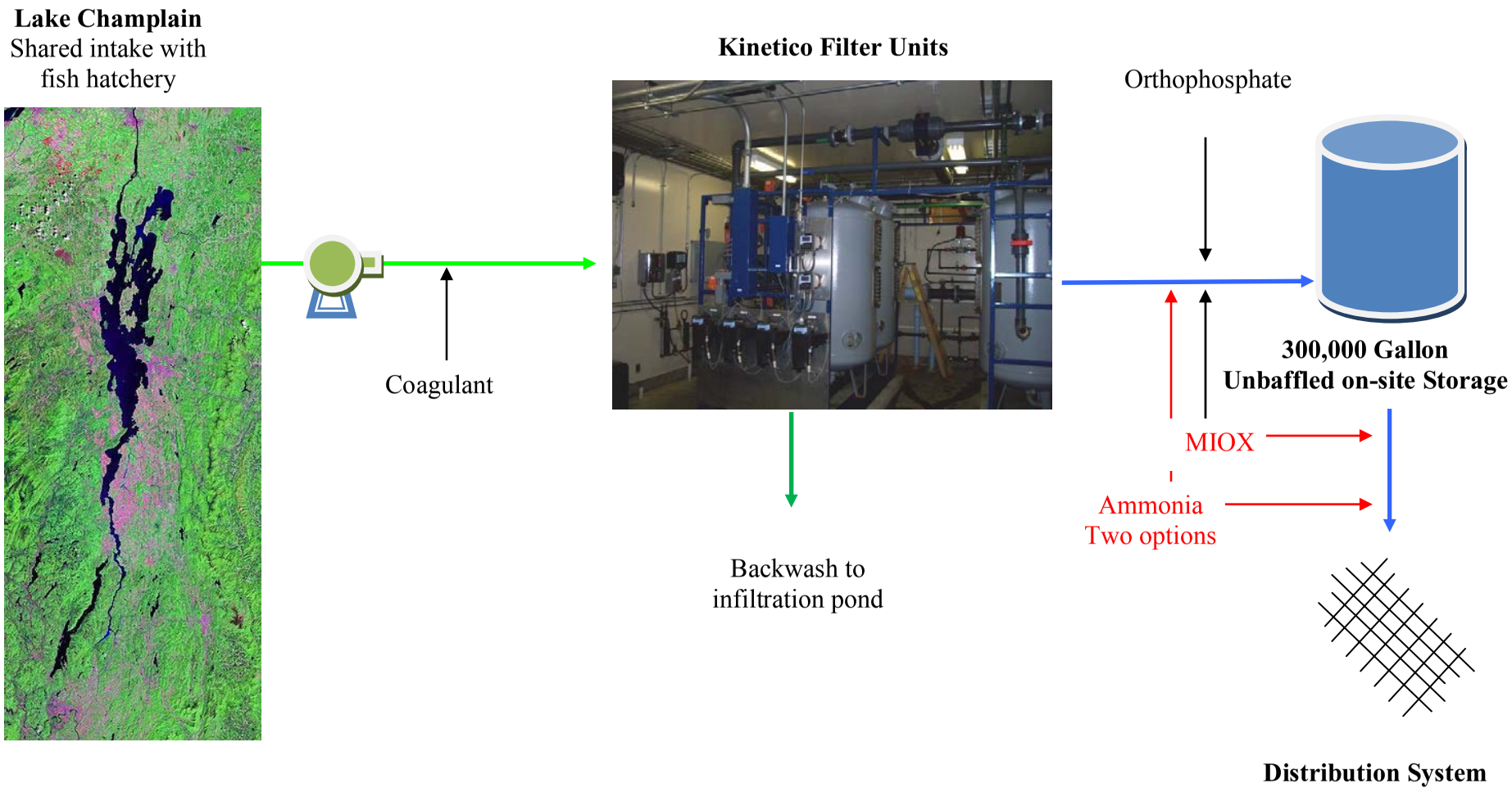
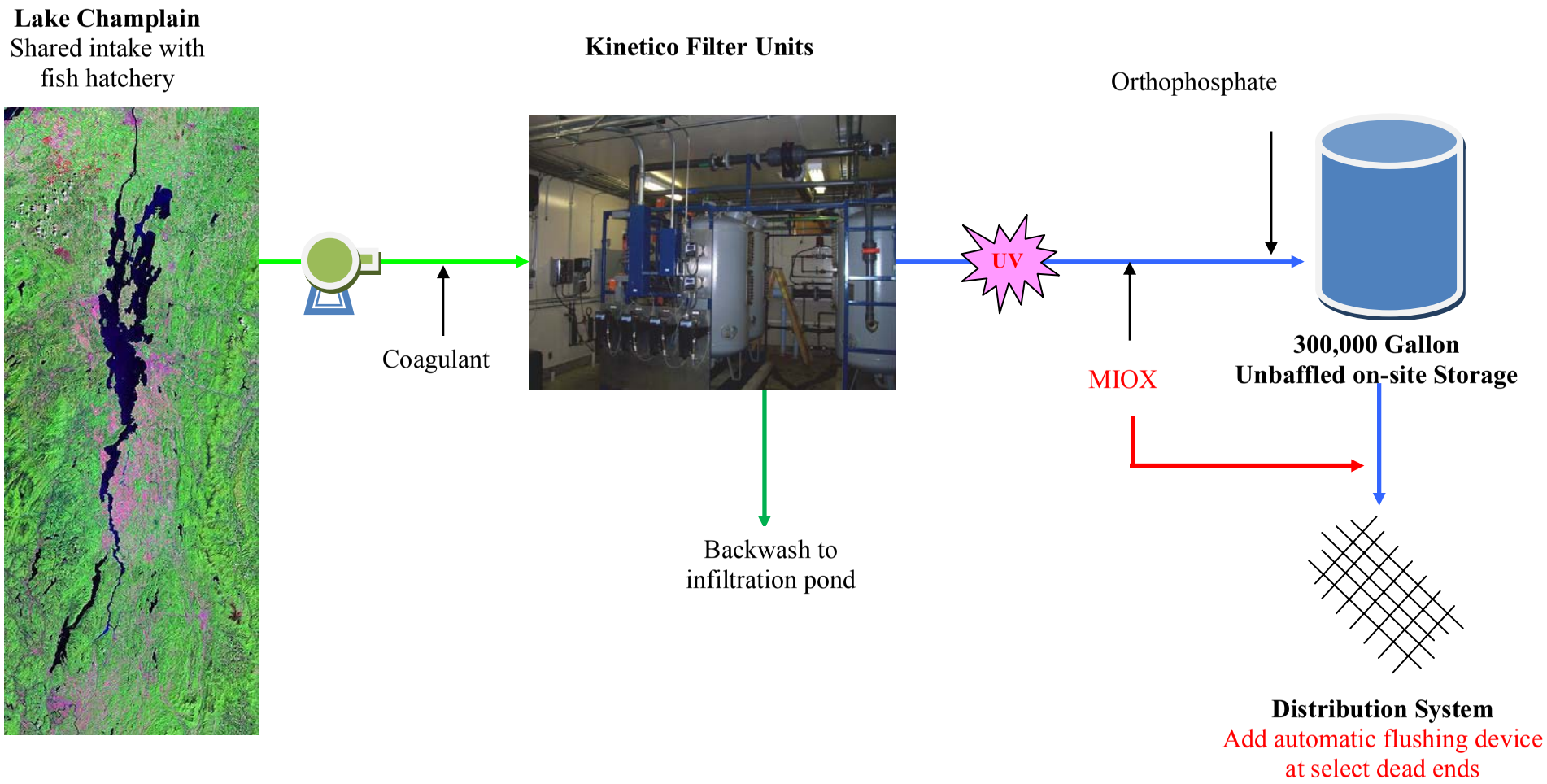


FIGURE 5-25D
GRAND ISLE CONSOLIDATED WATER DISTRICT PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS
(UPDATES IN RED)



5.2.3.19 North Hero - Existing System

The treatment summary for the existing system is provided below and in Table 5-27. Figure 5-28A provides a schematic of existing treatment. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source water for North Hero is from a shallow part of Lake Champlain. There exists only one intake.

Treatment: Lake Champlain provides flooded suction to the raw water pumps which provide low lift flow to the buoyant media clarifier. Sodium permanganate is mainly added for Zebra Mussel control and a coagulant and/or polymer is dosed prior to the clarifier. There is no static mixer (except largest filter tank which was recently added). The operator indicates that the static mixer did not help in floc formation. The buoyant media clarification provides partial removal of solids prior to mixed media filtration. Filtered water is treated with sodium hypochlorite and phosphoric acid and then enters an in-plant clearwell. Distribution pressure is maintained with variable speed pumps and a hydro-pneumatic tank.

Distribution: The distribution services the island and is fairly well looped. However, there are sections which service seasonal populations and can experience low demand. The distribution does not have a storage tank and pressure is maintained with a pneumatic tank. There are no consecutive systems.

Operator: One operator is full time and a second operator is part time filling weekends and vacations. The WTF operates "on demand" for 24 hours a day, seven days a week.

VT DEC Observations: None.

AECOM Observations: The raw and filtered TOC is not regularly measured. Thus overall performance is not known. It is expected with some more targeted jar testing, the plant could optimize TOC removal. In addition, the contact time reported for disinfection

purposes appears to be excessive. This is complicated by the lack of a distribution tank which would provide more flexibility to WTF operation. Lastly, seasonal water use on lines which are dead ends would require flushing to prevent stagnation of water in the distribution line.

TABLE 5-27
NORTH HERO SYSTEM SUMMARY

Population			
Value	600		
System Demand, MGD	February, 2008	April, 2008	August, 2008
Minimum	0.036	0.045	0.106
Average	0.043	0.067	0.127
Maximum	0.052	0.135	0.177
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	45		
-Estimated average diameter, inches	8 inch		
Approximate System Detention Time	10 days		
Number of Storage Tanks	None		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	No		
Preoxidation	25% Sodium Permanganate	Max. of 0.1 at clarifier	Dry
Coagulant	EC-462 Cationic Blended Polymer (Aluminum blended with DADMAC polymer and Polyamines polymer)	3.5	Wet
Alternative Coagulant	48.5% Alum	7.0	Dry
Chlorination	12.5% Sodium Hypochlorite	2.0 to 2.2 (summer) 1 to 1.5 (winter) Residual ~ 1.0	Dry
Corrosion Control	75% Phosphoric Acid	1.2 to 2.3 (Target 1.0 in dist. System)	Dry

Note: Coagulant and alternative coagulant can be blended as certain times of year.

5.2.3.20 North Hero - Proposed Alternatives

AECOM recommends that North Hero follow the steps provided in Table 5-28 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

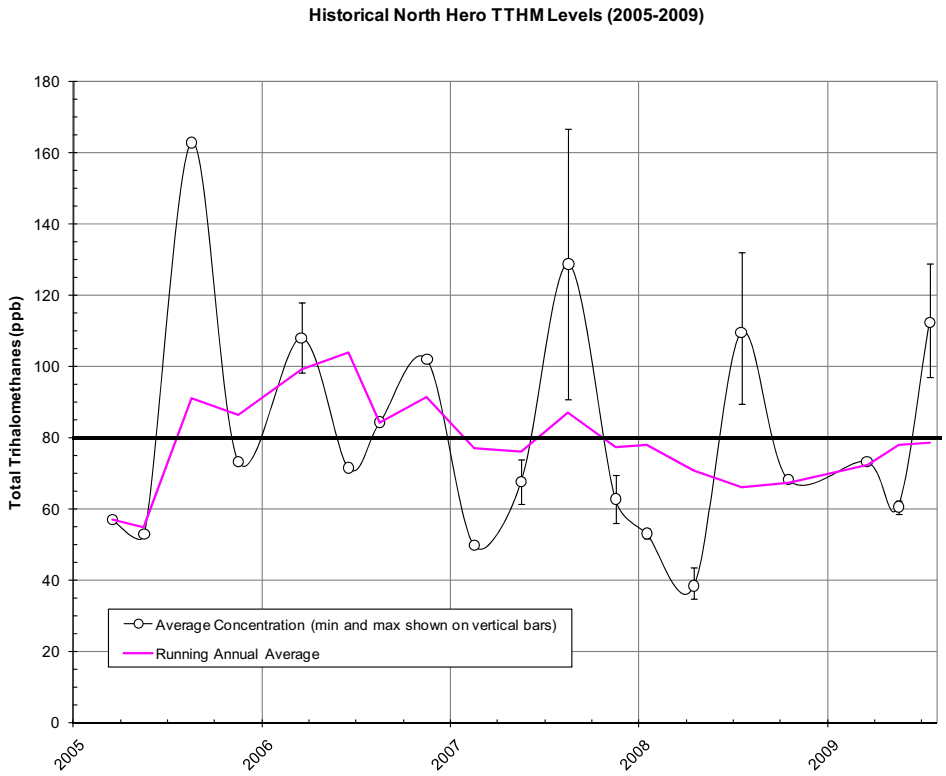
Figures 5-26 and 5-27 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that North Hero will be required to operate the recommended treatment for six months (i.e., during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP levels for chloramine can be found in Appendix 2 - Computer Model Output Summary under North Hero. The summer DBP levels for calibration were 100 ppb for TTHM and 62 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 90 ppb, HAA5 = 58 ppb
2. UV and post clearwell chloramination: TTHM = 79 ppb, HAA5 = 40 ppb
3. UV and pre clearwell chloramination: TTHM = 12 ppb, HAA5 = 5 ppb

Chlorine as distribution residual (Figure 5-28B)

North Hero has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Optimize the existing operations to remove TOC. This requires that more coagulant be dosed to remove TOC and/or an alternative coagulant scheme be implemented. This will also require modification of the clarifier run times (probably shorter) to remove the additional solids as well as increase the solids and volume loading to the existing residual handling systems. The high coagulant dose also will likely depress the pH and make chlorine more effective in the disinfection process, thus reducing the overall chlorine dose required.
2. Implement seasonal GAC absorber with a 20 minute EBCT for TOC removal after the rapid sand filters.



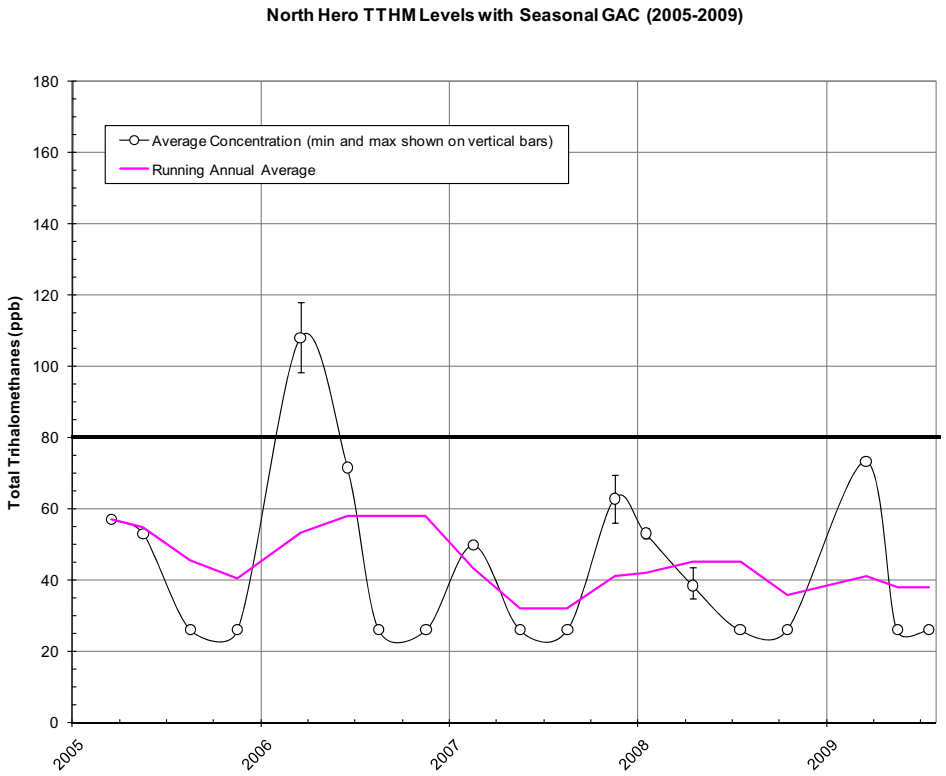
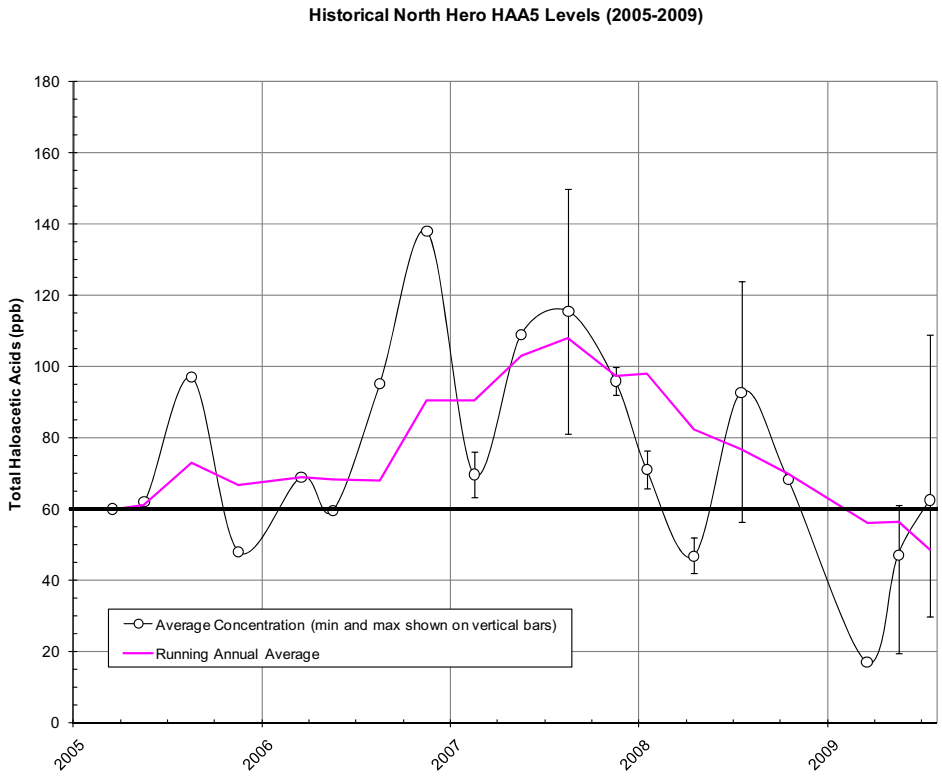


FIGURE 5-26
NORTH HERO TTHM LEVELS (2006-2009)



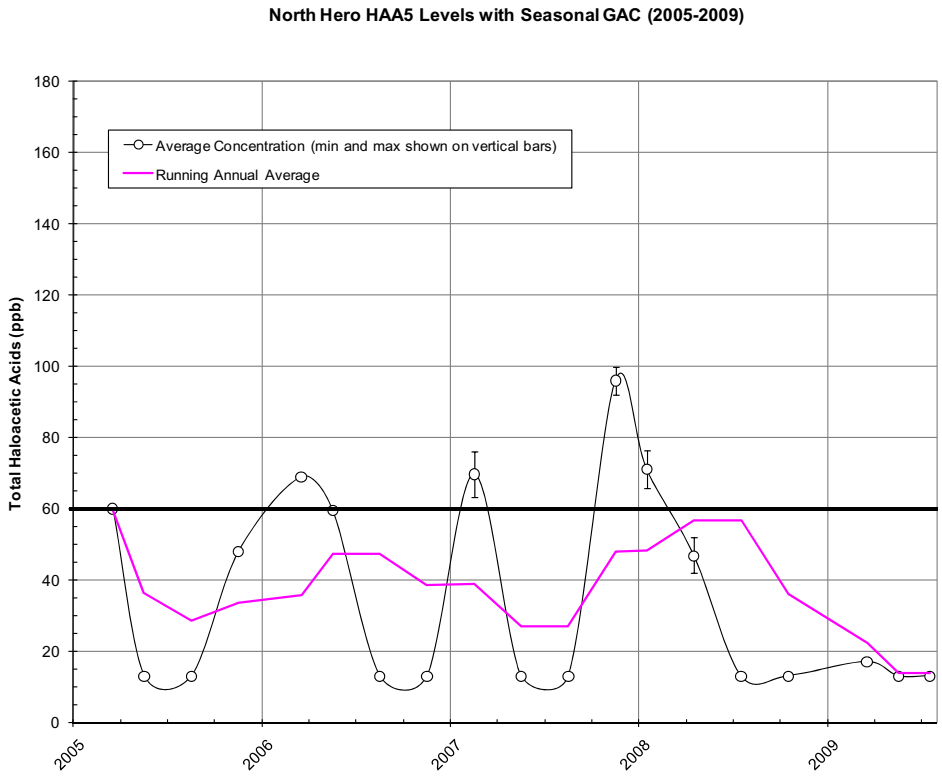


FIGURE 5-27
NORTH HERO HAA5 LEVELS (2006-2009)

Chloramine as the Distribution Residual (Figure 5-28C)

North Hero has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.
2. Installation of UV will reduce the reliance on chlorine as a disinfectant, or a possible alternative would be primary and secondary disinfection with chloramine only.

TABLE 5-28
PROPOSED ALTERNATIVES FOR NORTH HERO

Treatment- Existing System	Distribution
For Chlorine	
1.) Optimize coagulation to remove TOC	None proposed
2.) Seasonal post GAC absorber with 20 minute EBCT	
For Chloramine	
1.) Install UV disinfection to reduce chlorine for primary disinfection/ use chlorine for virus inactivation only	1.) Chloramine

5.2.3.21 North Hero - Other DBP Reduction Alternatives (Figure 5-28D)

A summary of other DBP reduction alternatives for North Hero is provided in Table 5-29. The following provides a brief discussion of each alternative.

1. Reported disinfection values are high, up to 10 times or more than what is required. This is somewhat complicated by the fact that the system does not have distribution tank and is required to maintain the clearwells as active status at all times. Installing a distribution tank will allow flexibility in WTF operation to reduce the contact time at the WTF. Coupled with this option would be separating of chlorine dosing for disinfection and chlorine required for the distribution system residual.

2. Since the system sees sections of the distribution system with seasonal summer use, then automatic flushing systems should be considered at the dead end to reduce residence time.
3. The WTF is at the end of the distribution system. Booster chlorination near the average time of the distribution can reduce the overall time chlorine is in contact with the finished water.

TABLE 5-29
OTHER DBP REDUCTION OPTIONS FOR NORTH HERO

Treatment- Existing System	Effect*	Distribution	Effect*
1.)Separate chlorine dosing for disinfection verses distribution residual	Low	1.) Automatic flushing	Low or Medium
2.) Install UV disinfection to reduce chlorine required for disinfection	Low	2.) Add distribution tank	Low or Medium
		3.) Booster chlorination	Low or Medium

*Effect is the anticipated relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-28A
NORTH HERO EXISTING PROCESS FLOW DIAGRAM

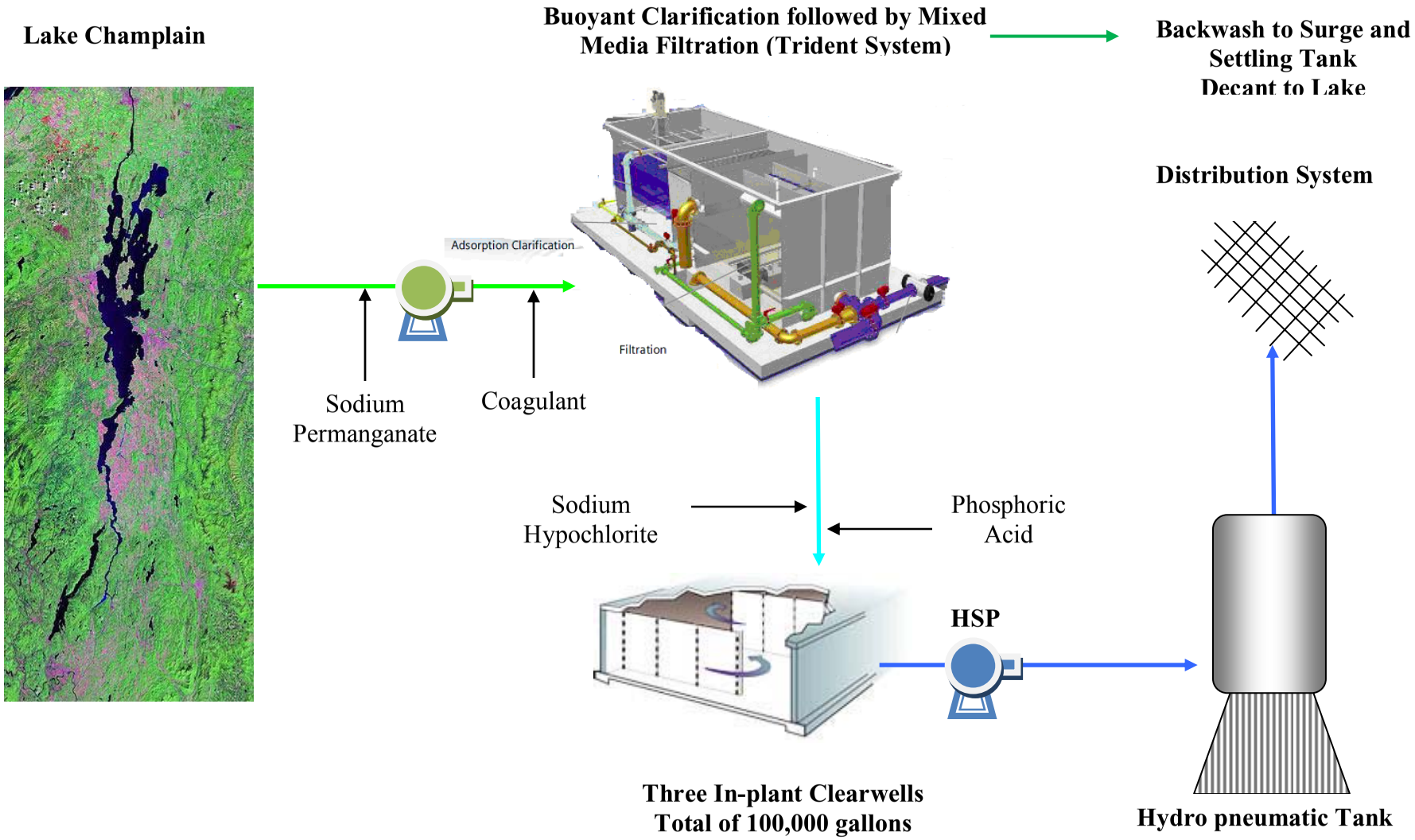


FIGURE 5-28B
NORTH HERO PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

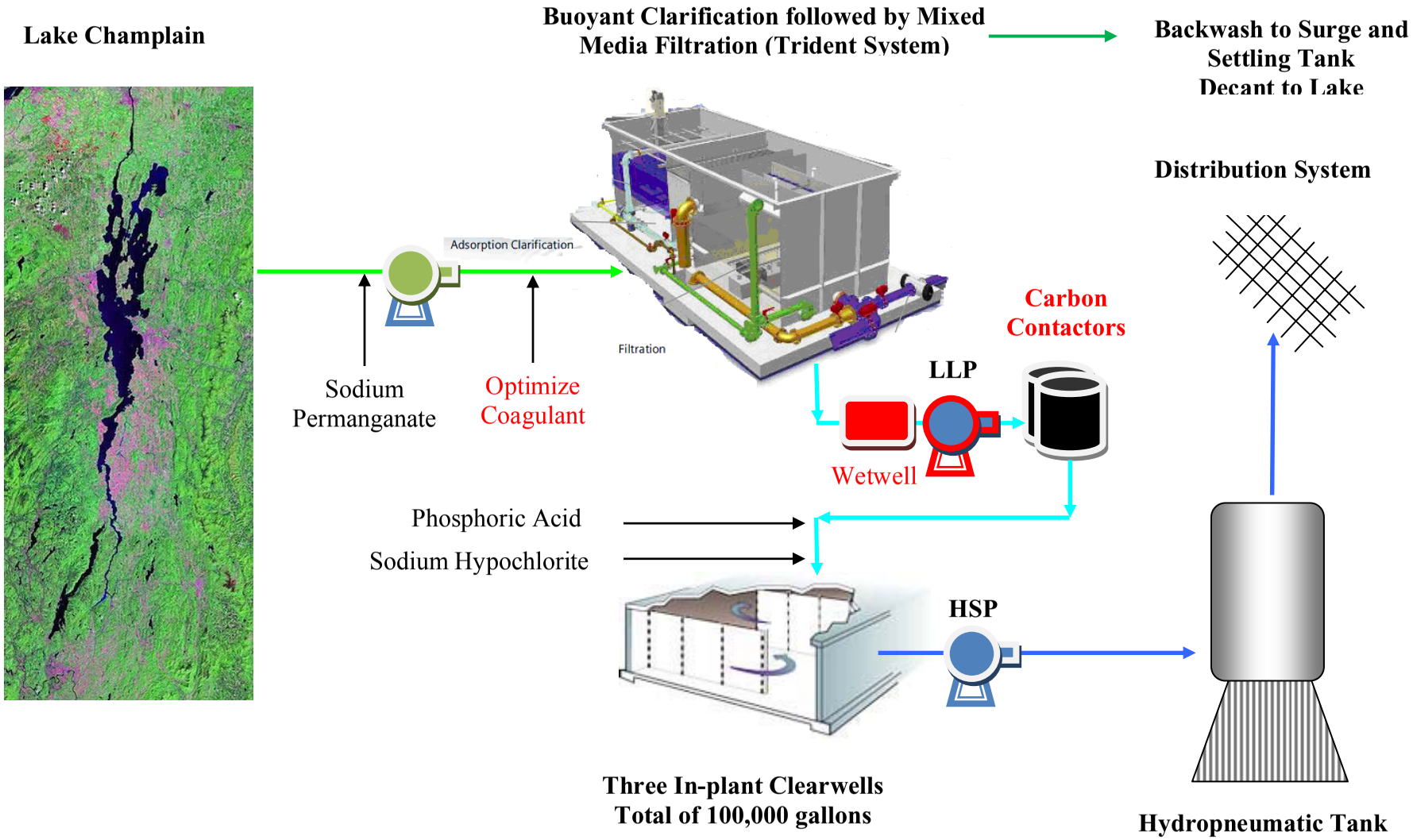


FIGURE 5-28C
NORTH HERO PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

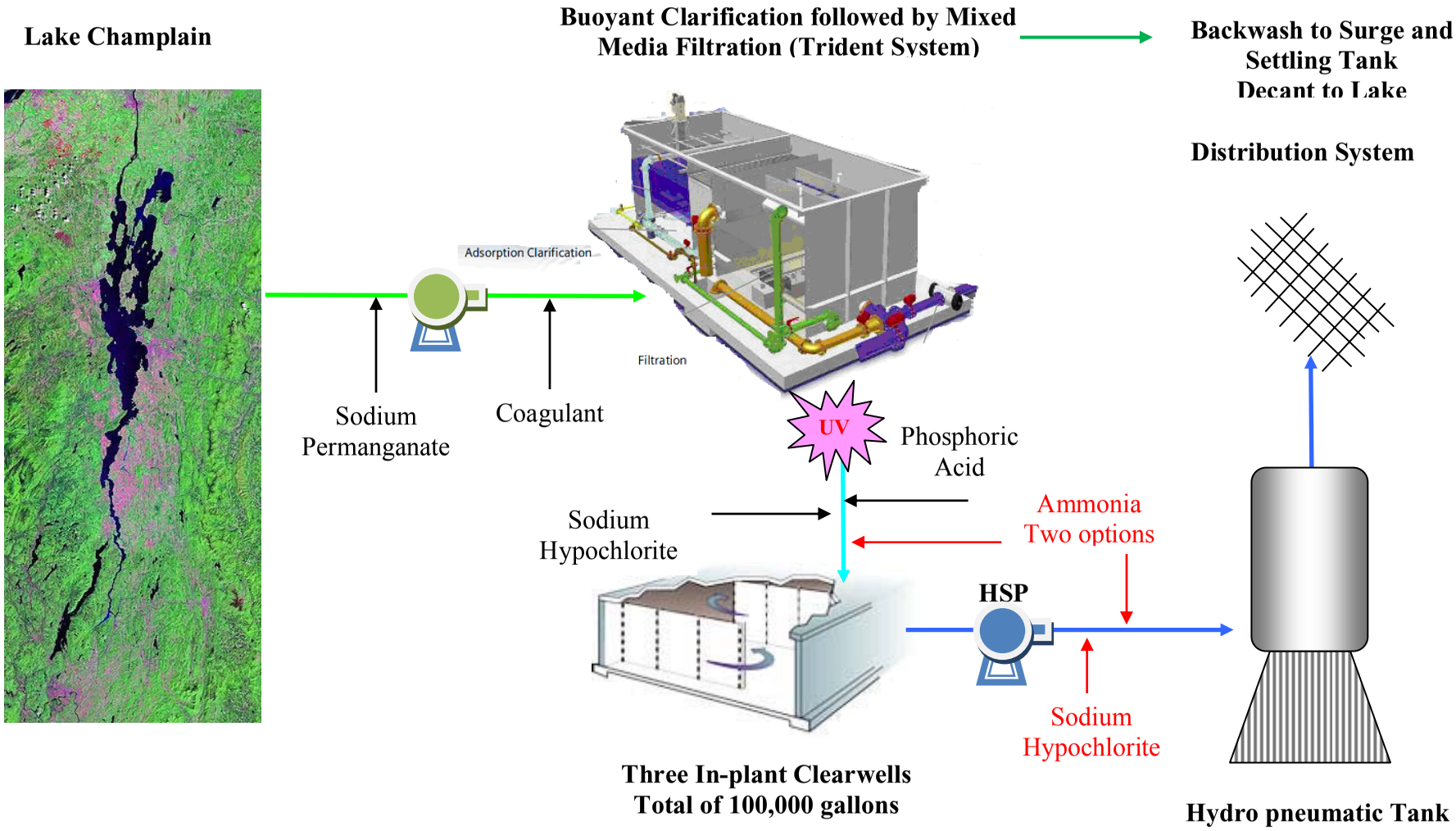
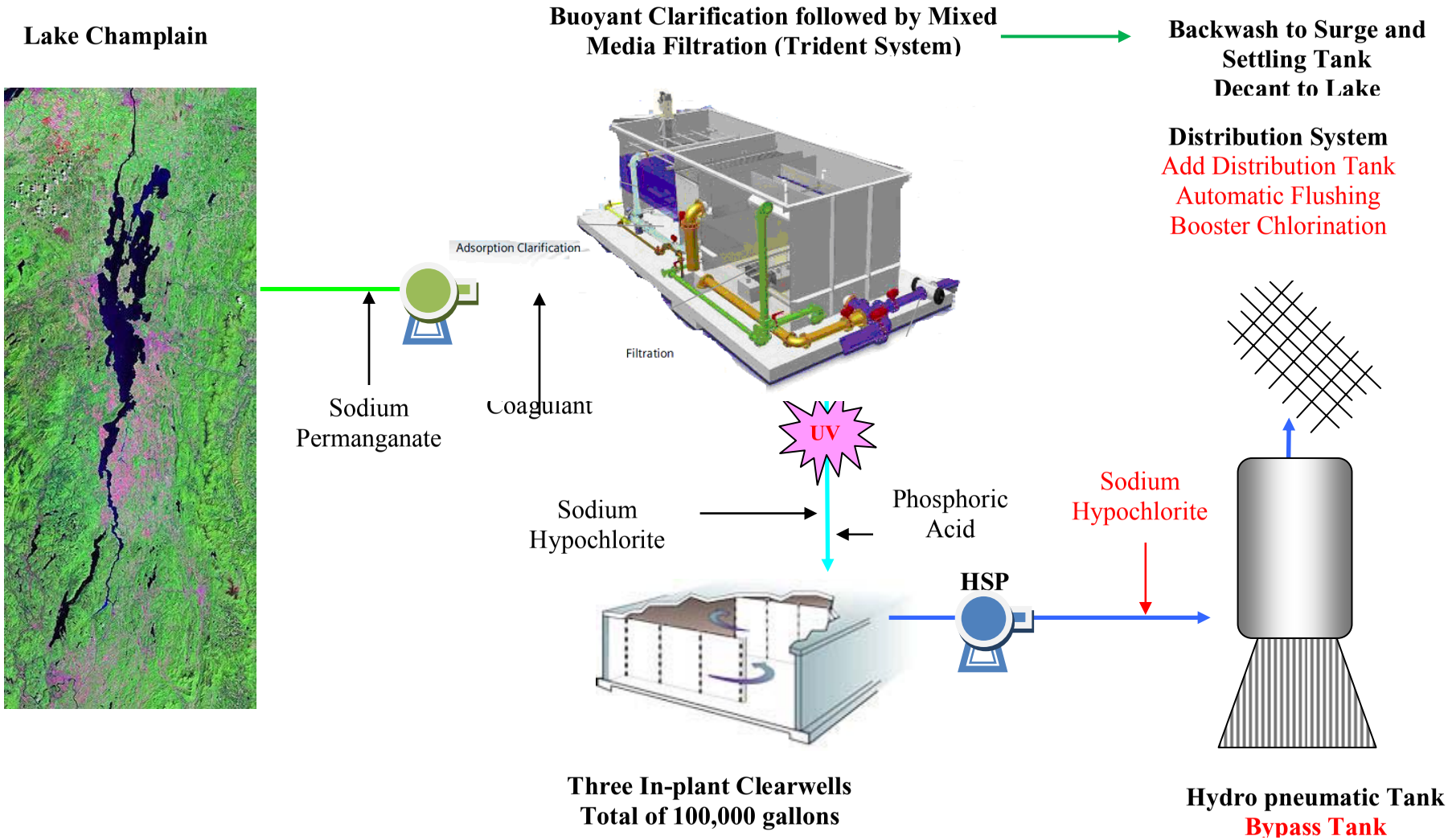


FIGURE 5-28D
NORTH HERO PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)
Stop Sodium Hypochlorite



5.2.3.22 Proctor - Existing System

The treatment summary for the existing system is provided below and in Table 5-30. The existing treatment is provided in Figures 5-31A. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source for the surface WTF is Kiln Brook, an upland mountain stream. The source can have turbidity events during rain storms which make it difficult to treat the water. The Town also has a well which also can supply water into the distribution system.

Treatment: The WTF consists of rapid mix, followed by flocculation and direct sand filtration. Water is treated with sodium hypochlorite, a set of coagulants, dosed with fluoride for dental health and finally soda ash for pH and alkalinity adjustment prior to filtration. Disinfection must be achieved within the WTF as there is no clearwell on site. Flow to remote distribution storage tanks is by gravity.

Distribution: The distribution is separated by a river and there are storage tanks on each side of the river. The system is flushed twice a year. There are no consecutive systems serviced.

Operator: The water treatment facility operates 24 hours a day, seven days a week. There are three full time operators.

VT DEC Observations: Turbidity can be flashy and dependent on storm event. Coagulant dose is considered a direct filtration dose and there is little settling in the floc basin. Disinfection must be achieved before filtration in the settling tank and above the filters. The system is currently exploring the option of abandoning the supply and utilizing an approved well.

AECOM Observations: The main problem with existing WTF setup is that the facility is required to pre-chlorinate in order to achieve disinfection of the source water. Table 5-30 provides an overview of DBP levels at the WTF and in the distribution system. Although it is difficult to make many conclusions as the date of sampling at the WTF is different than the distribution system sampling dates, it would appear that the bulk of the DBPs are formed at the WTF. The Town indicates that based on a limited coordinated testing done in 2007, about 20% to 40% of the DPB formation occurred in the distribution system.

TABLE 5-30
HISTORICAL DISINFECTION BYPRODUCT LEVELS AT WTF AND DISTRIBUTION SYSTEM

Sample Date Range and location	THM, ug/L	HAA5, ug/L
Average Stage 1 and 2 DBPR results from 2003 to 2009	93.7	88.8
WTF – 3/28/03	90.0	94.0

5.2.3.23 Proctor - Proposed Alternatives

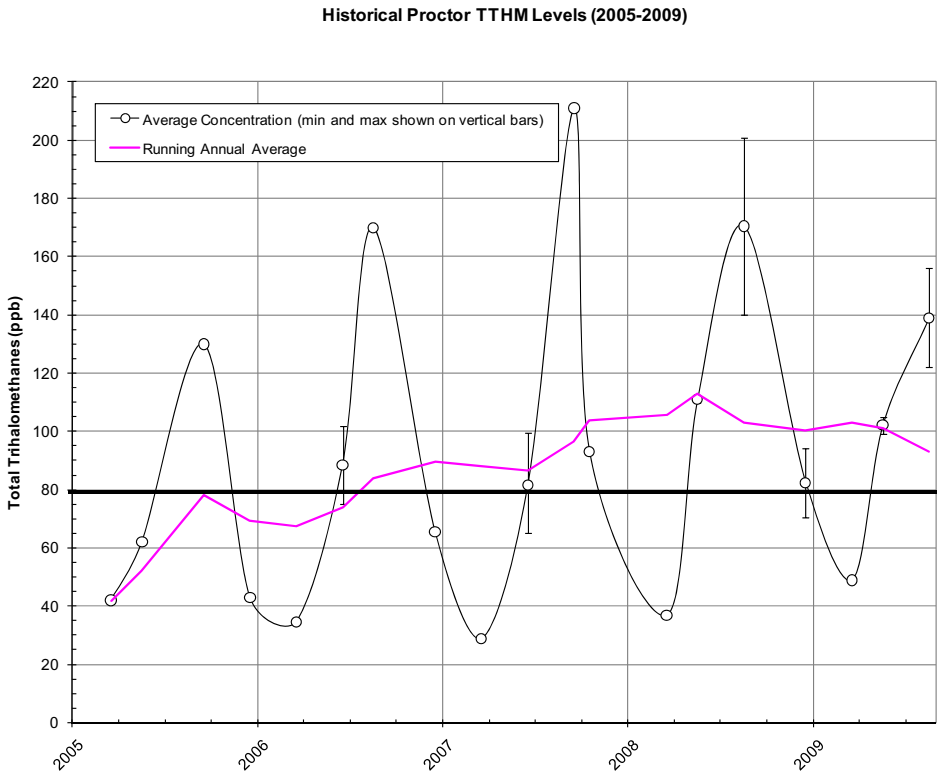
AECOM recommends that Proctor follow the steps provided in Table 5-31 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-29 and 5-30 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Proctor will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Proctor. The summer DBP levels for calibration were 121 ppb for THM, and 102 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 73 ppb, HAA5 = 61 ppb
2. UV and post clearwell chloramination: TTHM = 42 ppb, HAA5 = 27 ppb

TABLE 5-31
PROCTOR SYSTEM SUMMARY

Population			
Value	2,200		
System Demand, MGD			
	January, 2008		
Minimum	0.355		
Average	0.387		
Maximum	0.407		
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	22		
-Estimated average diameter, inches	~6 Inches		
Approximate System Detention Time	Generally less than 50 hours. Some dead ends at greater than 150 hours.	See Distribution Water Age Map	
Number of Storage Tanks	2		
-Provide Size of each, MG	0.10 (East Side) and 0.625 (West Side)	A new 0.5 MG East side tank proposed	A new 0.03 MG Bluff St tank proposed.
-Tank Type	Standpipe		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	Yes		
Preoxidation	12.5% Sodium Hypochlorite	(see disinfection)	Dry
Coagulant 1	EC-462	5.2 (4 to 12.5)	Wet
Coagulant 2	Aluminum chlorohydrate (ACH)	4 mg/L	Wet
Disinfection	12.5% Sodium Hypochlorite	3.2 (Range 1.4 to 8.2)	Dry
Corrosion Control	Soda Ash	Target pH of 7.8 to 8.0 in dist. System	Dry
Dental Health	Fluoride	1.0	Dry



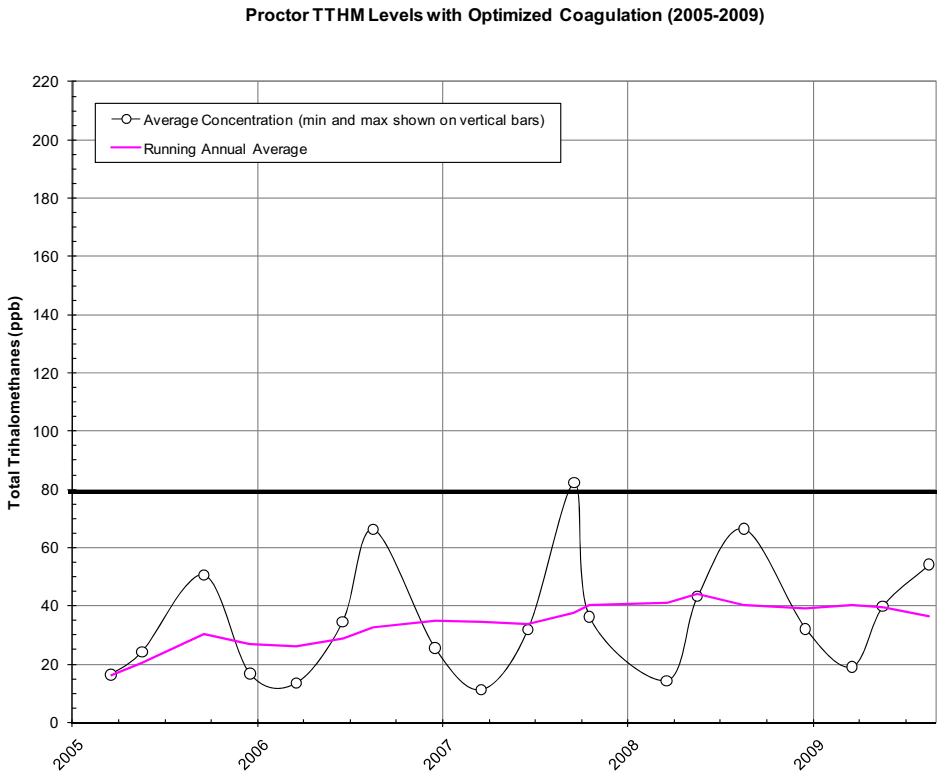
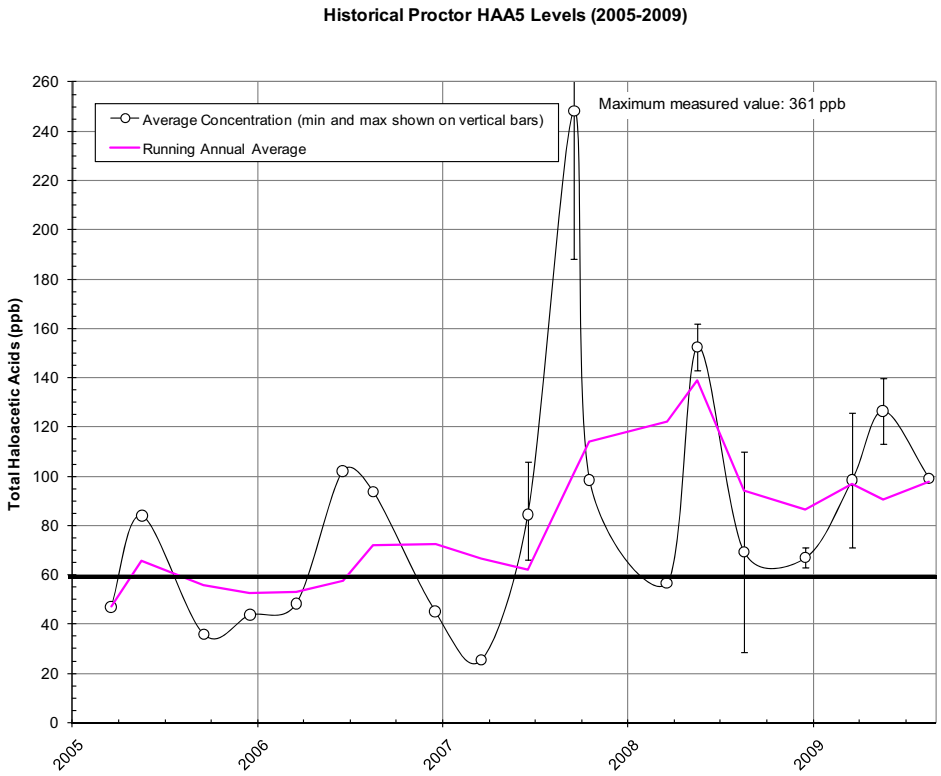


FIGURE 5-29
PROCTOR TTHM LEVELS (2006-2009)



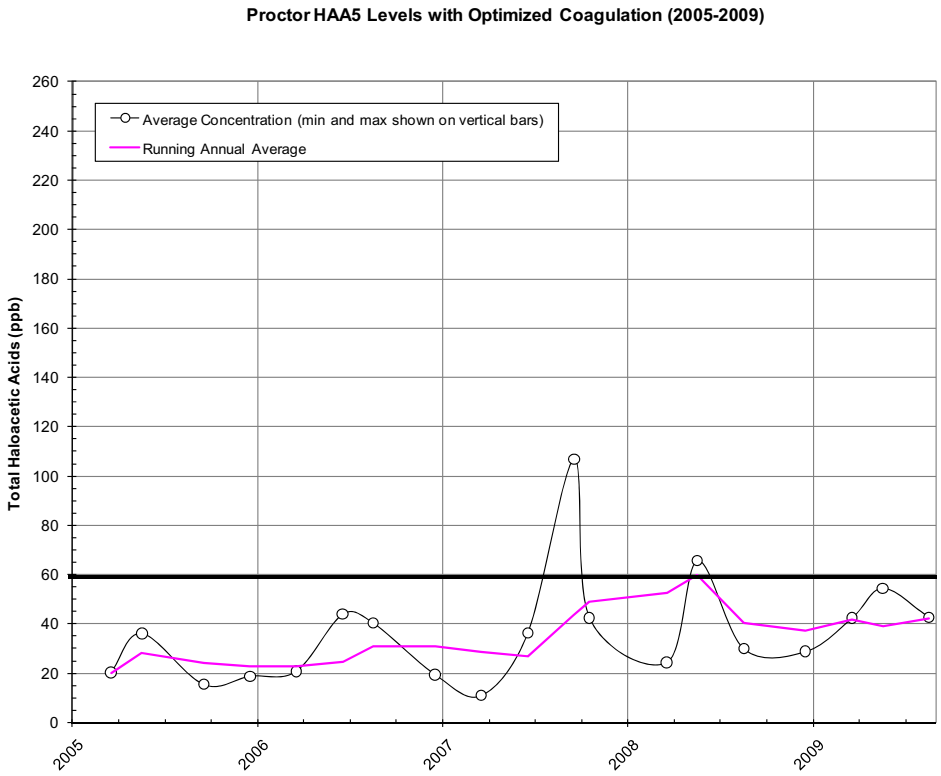


FIGURE 5-30
PROCTOR HAA5 LEVELS (2006-2009)

Chlorine as Distribution Residual (Figures 5-31B and 5-31C)

The Town is in the process of evaluating the condition of the water treatment facility. Currently, the Town' engineer (SVE Engineers) has estimated $1.4M in repairs and updates to the existing WTF. The Town is considering abandoning the surface water source in favor of wells due to the cost of the WTF updates and potential repairs associated with the 5 mile finished water supply pipeline to the Town. In view of this, two options remain for the Town:

1. Replace the existing WTF with a new water treatment facility with a clarification process which allows flexibility in coagulant dosing. Based on the nature of the source being flashy at time as well as hard to coagulate, the following treatment technologies would be likely candidates for the source:
   a. Ballasted clarification (Actiflo by Krüger) followed by filtration.
   b. Trident HS tube settlers, buoyant media clarification, and filtration.
   c. Degrémont Technologies DensaDeg followed by filtration.
   d. Dissolved Air Flotation (DAF).
2. Develop wells to replace the existing surface supply. Developing additional wells to provide the necessary supply would allow the surface source to be discontinued or used as emergency basis. The wells would likely have less TOC which is less reactive to chlorine. However, the wells require electricity to pump the water which is not needed with the surface source as well as pH adjustment for corrosion control. In addition, wells tend to have hard water compared to the surface water which can cause aesthetic complaints.

Chloramine as the Distribution Residual (Figure 5-31D)

It is believed that if a new treatment or source is developed then it will be unlikely that chloramine would be implemented. If chloramination is required, the following would be anticipated steps required.

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to a level below MCLs.

2. Installation of UV will reduce the reliance on chlorine as a disinfectant.
3. Move fluoride dosing point.
4. Add contact tank for virus inactivation.

TABLE 5-32
PROPOSED ALTERNATIVES FOR PROCTOR

Treatment- Existing System	Distribution
For Chlorine	
1.) Replace existing WTF	None proposed
2.)Develop new well sources	
For Chloramine	
1.) Install UV disinfection to stop pre-chlorine. Chlorination required for virus inactivation only	1.) Chloramine
2.) Move fluoride dosing point	
3.) Add contact tank for virus inactivation	

5.2.3.24 Proctor - Other DBP Reduction Alternatives (Figure 5-31E)

A summary of other DBP reduction alternatives for Proctor is provided in Table 5-33. The following provides a brief discussion of each alternative.

1. The system can be retrofitted with UV disinfection on the filtered water as well as a small contact basin for 2.0 log virus disinfection. This will allow for chlorine to be dosed after filtration and limit contact with TOC.
2. Continue to optimize the existing treatment operations to remove TOC. This requires that more coagulant be dosed to remove TOC and/or an alternative coagulant scheme be implemented. It is noted that VT DEC has one TOC dataset on record on 9/10/2007 for the distribution system. Average distribution TOC was 1.75 mg C/L with a range of 1.4 to 2.2 mg C/L for 6 samples. Assuming that the raw water is similar to Table 5-1 for a stream/brook, then the removal through the existing water treatment facility is fair.
    a. The fluoride should be dosed after the filter as fluoride can associate with aluminum and increase the solubility of aluminum, thus preventing good floc formation.

3. Minimize the disinfectant in the 5 mile transmission main. Booster chlorinate at the end of the main perhaps at the field street well area. Also consider booster chlorination at the distribution tanks.
4. Since the system is long and contains several dead ends, the utility should consider looping the system so that water has an opportunity to circulate and not site at a dead end. If this is not possible, then automatic flushing systems should be considered at the dead end.
5. Seasonal use of the well in the summer. Testing is required to confirm the following, but well water tends to have lower TOC levels which are less reactive to chlorine than surface water.

TABLE 5-33
OTHER DBP REDUCTION ALTERNATIVES FOR PROCTOR

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Add UV disinfection at the WTF with virus chlorine contact tank to stop prechlorination.	Medium	1.) Booster chlorination	Medium
2.) Optimize coagulation to remove TOC. Move fluoride dosing point.	Medium	2.) Looping dead ends and/or seasonal flushing	Low or Medium
3.) Seasonal use of well source.	Ranges		

*Effect is the anticipated relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-31A
PROCTOR EXISTING PROCESS FLOW DIAGRAM

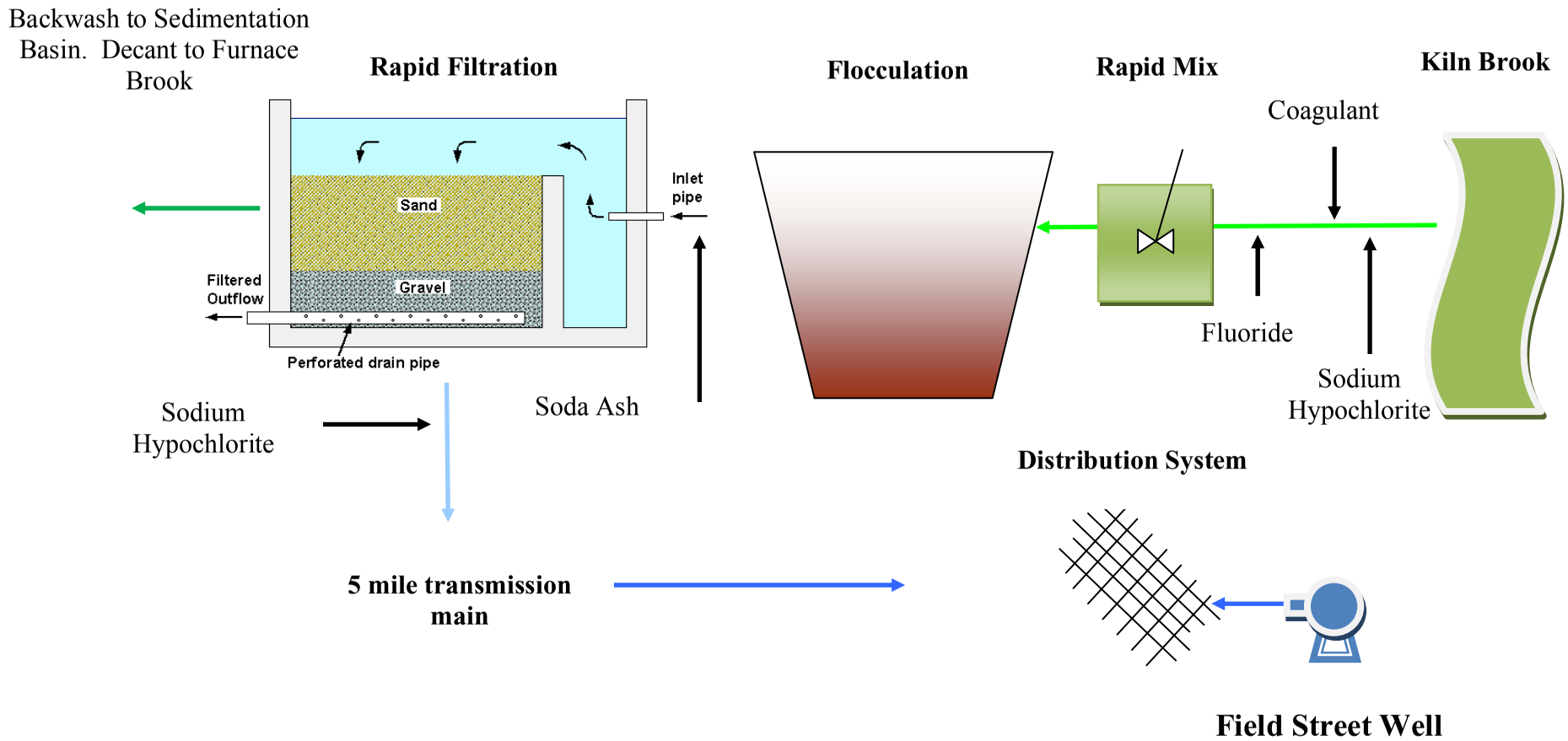


FIGURE 5-31B
PROCTOR ALTERNATIVE NO. 1 PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

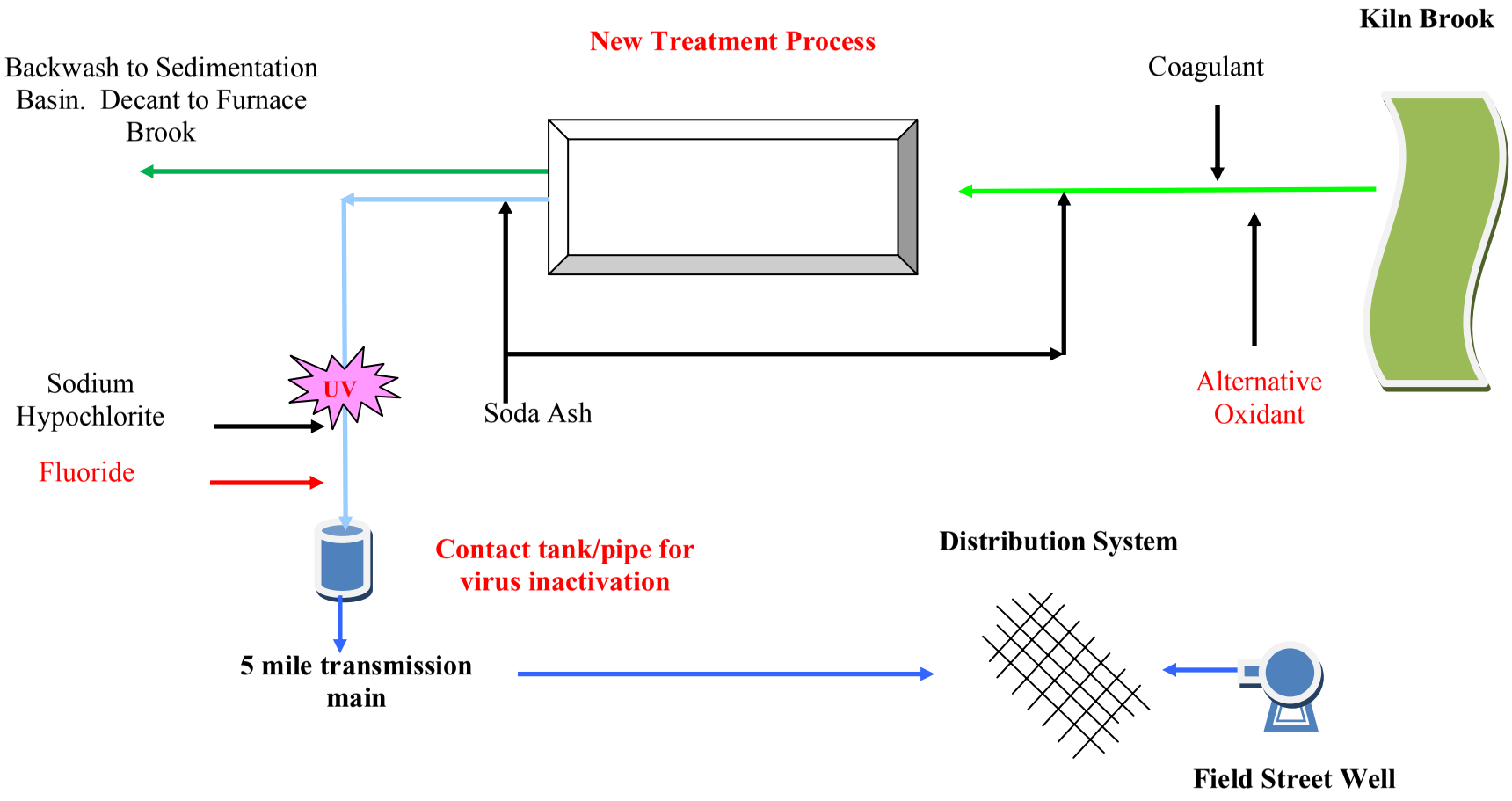


FIGURE 5-31C
PROCTOR ALTERNATIVE NO. 2 PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

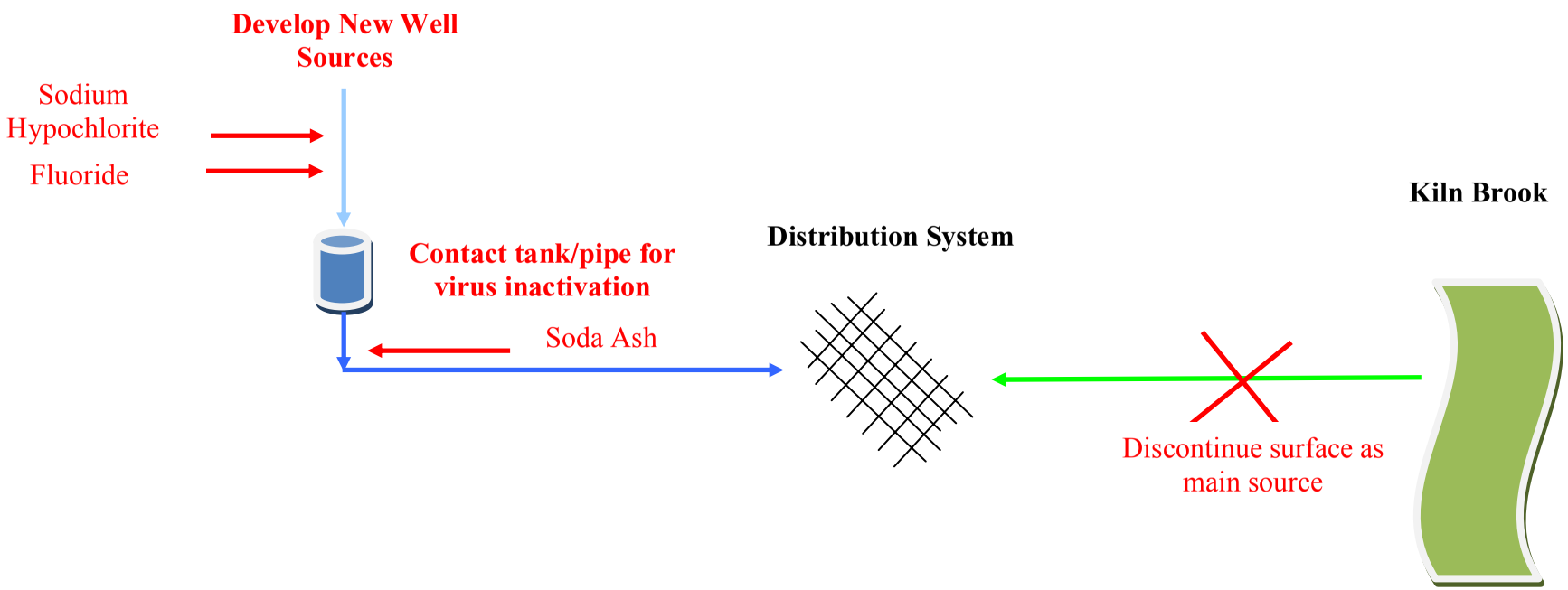


FIGURE 5-31D
PROCTOR PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

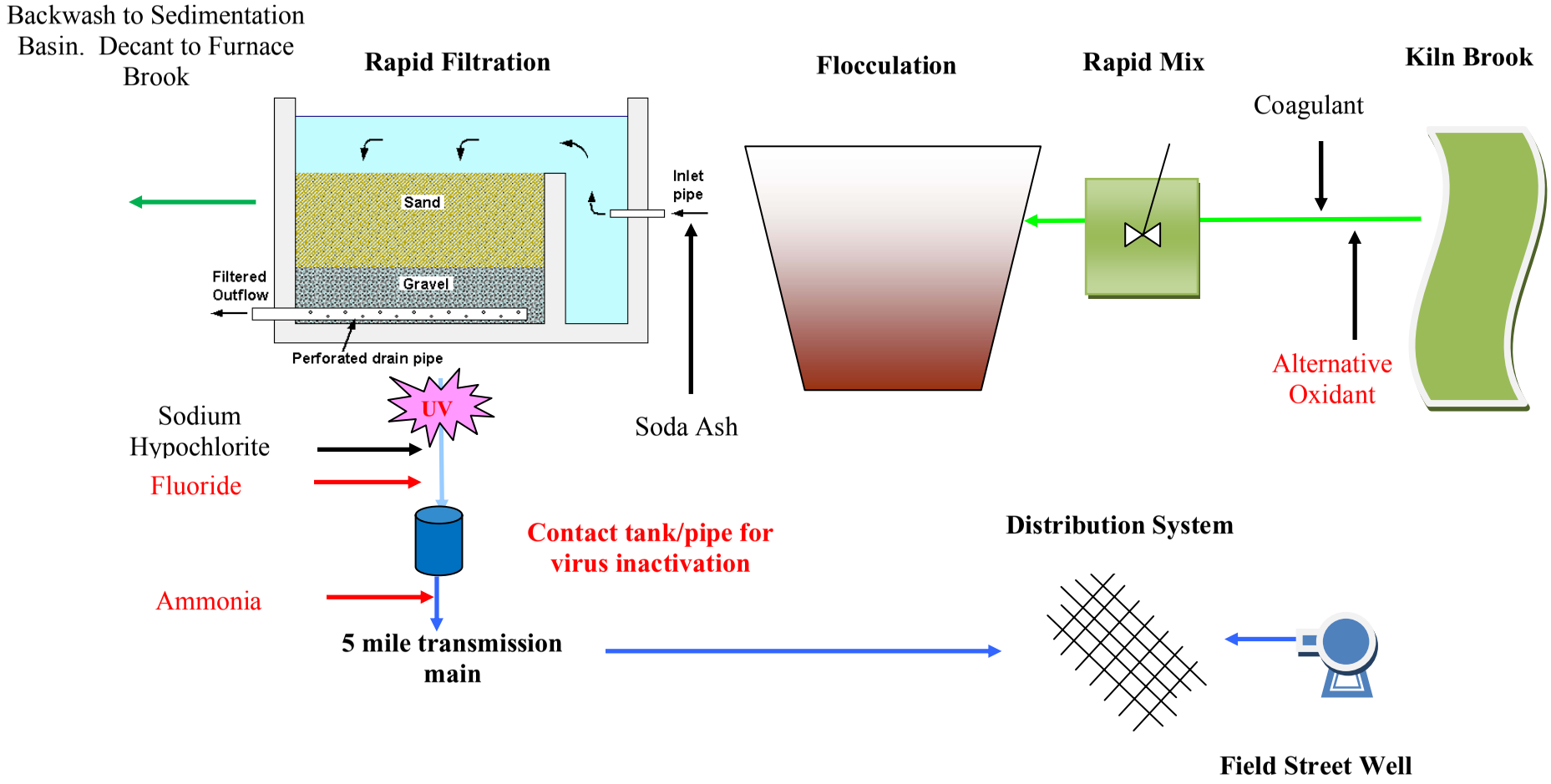
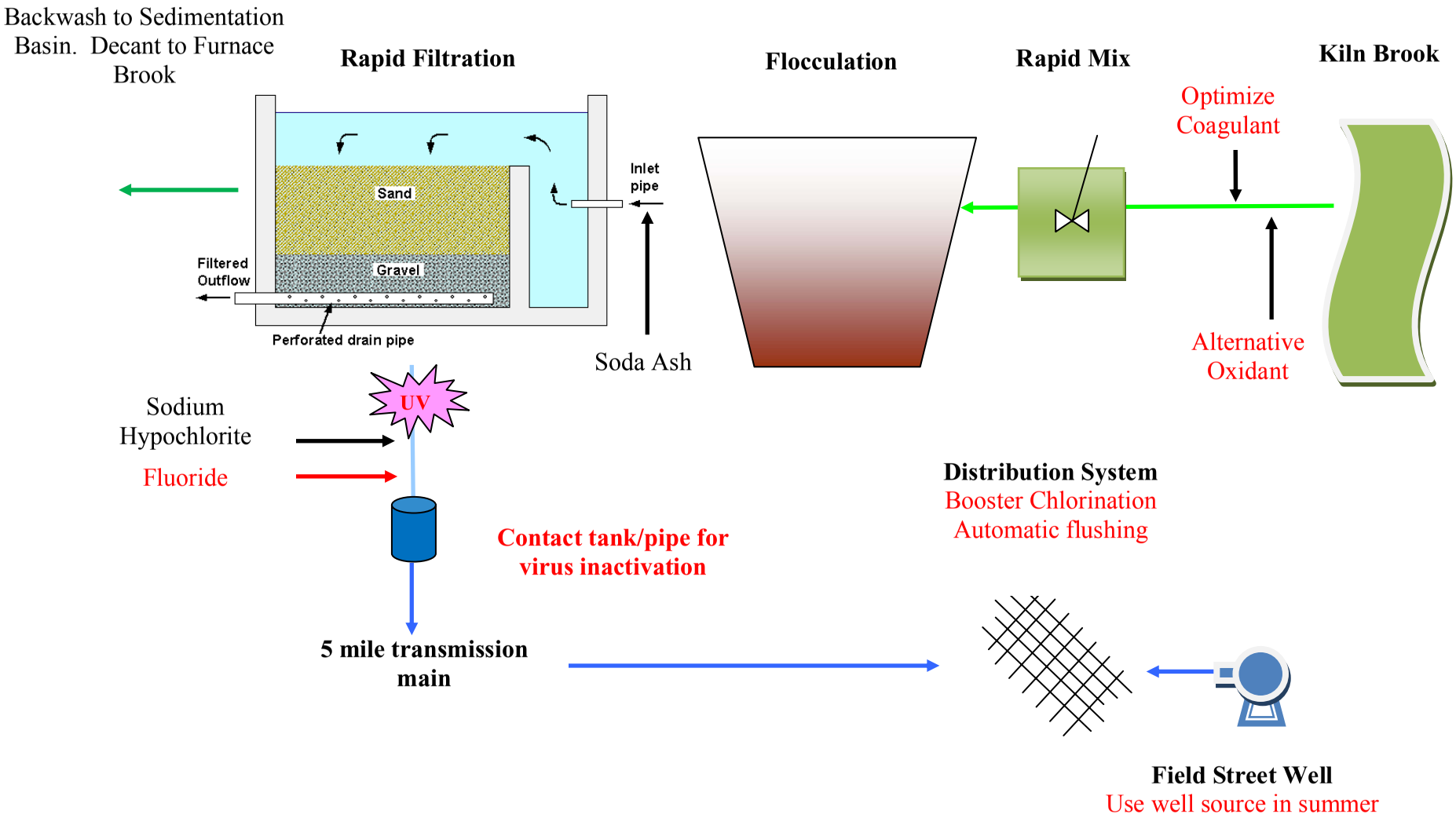


FIGURE 5-31E
PROCTOR PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTION (UPDATES IN RED)



5.2.3.25 Readsboro Village - Existing System

The treatment summary for the existing system is provided below and in Table 5-34. Figure 5-34A provides a schematic of the existing system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source for Readsboro is a low alkalinity upland stream out of Howe Pond. A single intake is used for tapping the source.

Treatment: A coagulant is added to the raw water followed by direct filtration using Culligan brand filters (roughing filter followed by a polishing filter). Disinfection is achieved via an onsite contact tank and water flows out by gravity to remote storage. The system has a difficult time treating the source water when turbidity spikes to 2.0 NTU. The chemicals currently are not flow paced and are manually set by the operator. The building is in good condition. The electronics are outdated and are due to be replaced. The filtration equipment and pipe show signs of outside rust, but appear to be in relatively good shape. Internal inspection is required to determine system condition and necessary treatment and/or replacement.

Distribution: The distribution consists of cast iron and ductile iron pipe. The cast iron pipe is over 100 years old. The ductile iron pipe was installed in the 1960's. The system is not looped and is flushed two times a year. There are no consecutive systems.

Operator: There is a part time operator at the WTF. The WTF is operated four days a week (Monday-Thursday). The WTF is shut down on Thursday night. If needed, the WTF is run for partial day on Friday. The WTF is off Saturday and Sunday. The goal is to fill the distribution tank for weekend when WTF is not in operation.

VT DEC Observations: None

AECOM Observations: The water treatment facility is not operated seven days a week, and thus tends to overdose chlorine so that chlorine levels remain detectable over the weekend. In addition, the WTF is not designed to remove TOC.

TABLE 5-34
READSBORO SYSTEM SUMMARY

Population			
Value	440		
System Demand, MGD			
	Typical		
Minimum	---		
Average	0.037		
Maximum	----		
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	Few Miles		
-Estimated average diameter, inches	6 inch		
Approximate System Detention Time	Not know		
Number of Storage Tanks	1		
-Provide Size of each, MG	0.275		
-Tank Type	Two concrete chambers each approx. 38’3” W x 37’8” L make up larger tank		

TABLE 5-34 (Continued)
READSBORO SYSTEM SUMMARY

Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	Yes		
Coagulant	EC-462 Cationic Blended Polymer (Aluminum blended with DADMAC polymer and Polyamines polymer)	4-6	Wet
Chlorination	12.5% Sodium Hypochlorite	5 to 6* (1.0 to 1. 4 Residual)	Dry
Corrosion Control	Sodium Bicarbonate	21	
Corrosion Control	27% Orthophosphate	2.5	dry

* Based on Operator estimate of 1.5 gallons of 12.5% bleach per day.

5.2.3.26 Readsboro - Proposed Alternatives

AECOM recommends that Readsboro follow the steps provided in Table 5-35 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-32 and 5-33 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Readsboro will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Readsboro. The summer DBP levels for calibration were 73 ppb for TTHM and 72 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 37 ppb, HAA5 = 47 ppb
2. UV and post clearwell chloramination: TTHM = 26 ppb, HAA5 = 20 ppb

Chlorine as Distribution Residual (Figure 5-34B)

Readsboro has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Consider operation of the WTF seven days a week. Consistent operation of the WTF would reduce the requirements to "boost" the chlorine levels during weekend periods when the WTF was not in operation. The chlorine should be flow placed to provide efficient dosing of the bleach. This will require that the electronics be upgraded so that the system can be flow paced and truly provide unmanned operation.
2. Seasonal use of GAC with EBCT of 20 minutes. The seasonal use of GAC will remove TOC such that the DBP formation would be reduced. The GAC typically has a life of 3 to 6 months and would be required to be replaced yearly.
3. Develop a well source. A high water quality well will reduce the disinfection requirements and DBPs formed. However, the well source could have higher electrical requirements compared to existing source and may also require treatment or iron, manganese, etc.

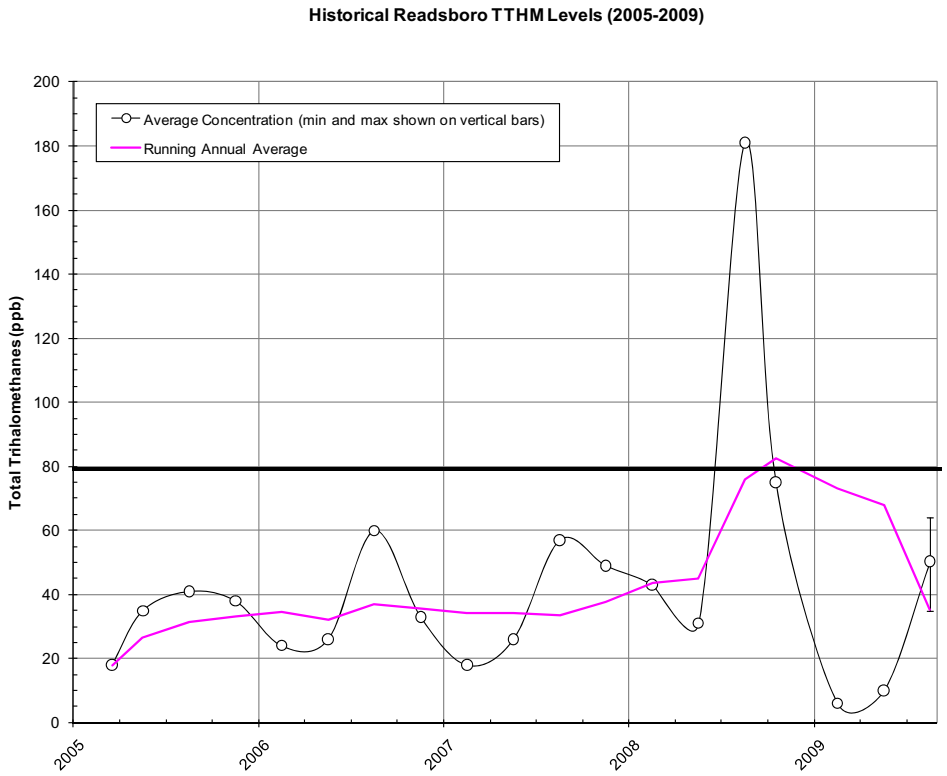
Chloramine as the Distribution Residual (Figure 5-34C)

Readsboro has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs below MCLs.
2. Update SCADA and associate controls

TABLE 5-35
PROPOSED ALTERNATIVES FOR READSBORO

Treatment- Existing System	Distribution
For Chlorine	
1.) Seven day operation of WTF. Update SCADA	None proposed
2.) Seasonal use of GAC absorber	
3.) Develop a well source	
For Chloramine	
1.) Update SCADA	1.)Chloramine



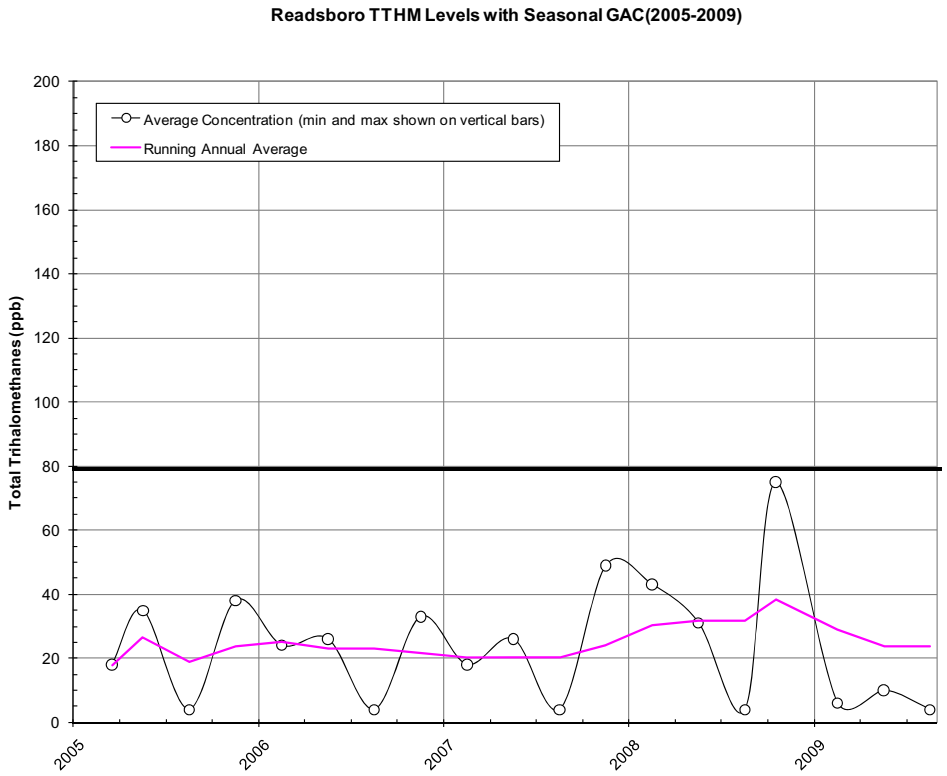
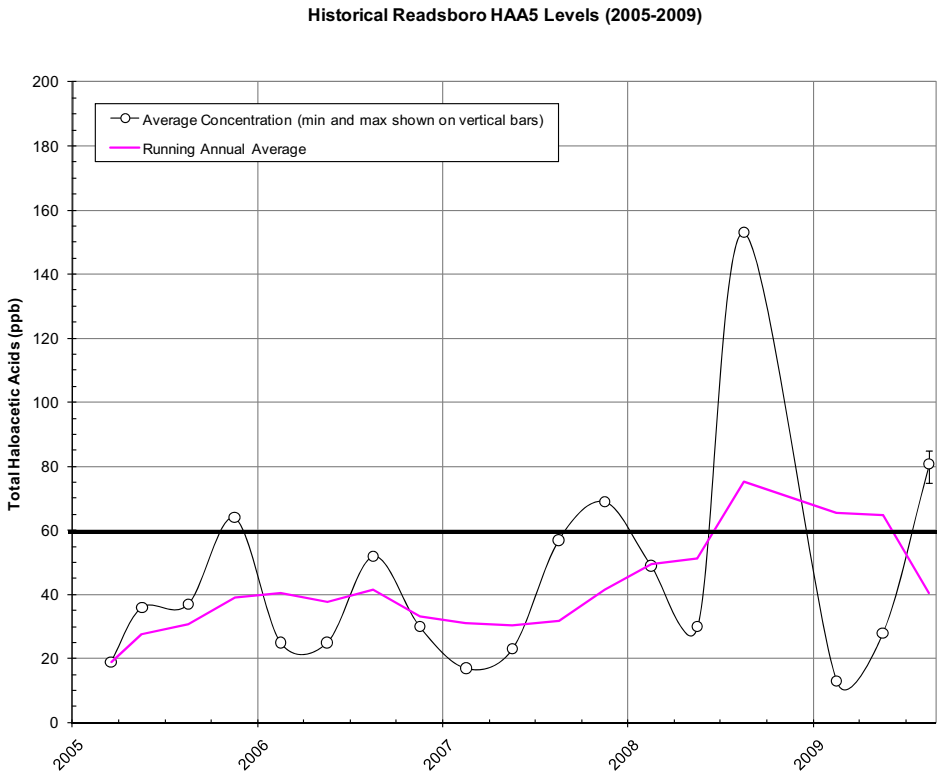


FIGURE 5-32
READSBORO TTHM LEVELS (2006-2009)



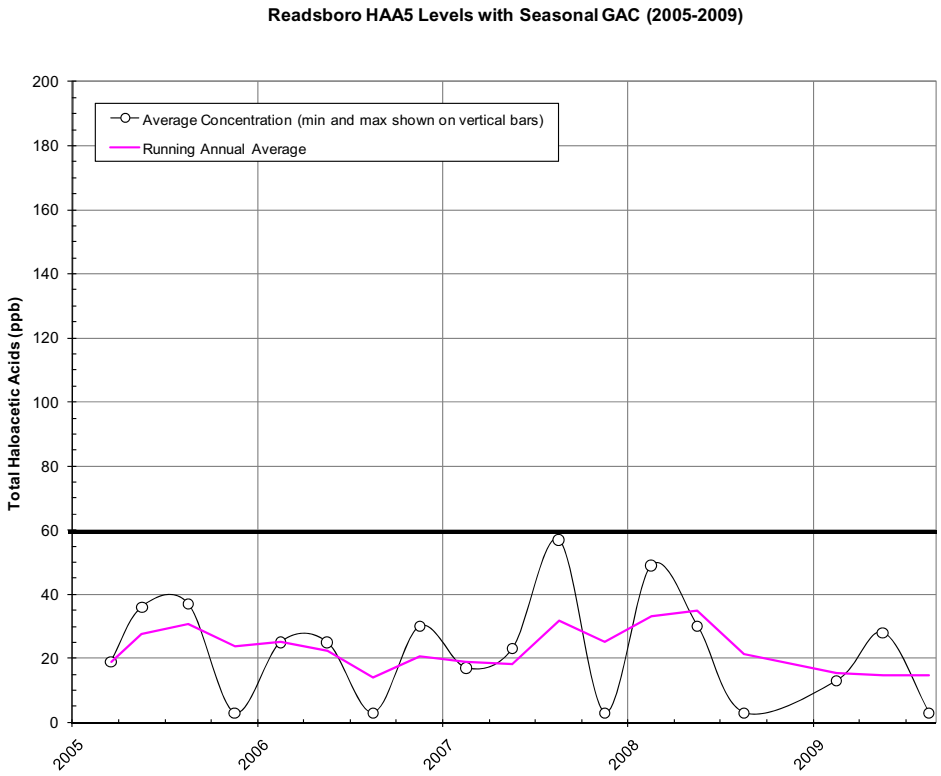


FIGURE 5-33
READSBORO HAA5 LEVELS (2006-2009)

5.2.3.27 Readsboro - Other DBP Reduction Alternatives (Figure 5-34D)

A summary of other DBP reduction alternatives for Readsboro is provided in Table 5-36. The following provides a brief discussion of each alternative.

1. Currently, the chemicals are not flow paced using a flowmeter. Flow pacing will provide a tighter range in which chemicals are dosed and could reduce the amount of sodium hypochlorite dosed.
2. Booster chlorination at the distribution tank. This would reduce the required chlorine dosed to maintain a chlorine residual over the weekend. The tanks do not have power lines. Thus either lines need to be constructed or a solar system needs to be installed and associated building.
3. Since the system contains several dead ends, and it will not be easy to loop these lines. Automatic flushing systems should be considered at the dead ends to reduce water age. Flushing however does not take advantage of consuming the water and could be considered a waste.
4. Switch to chlorine dioxide as residual disinfectant. A system could be rented or chemical purchased and tested on a short term basis as discussed in Section 4.4.4: Chlorine Dioxide (Implementation). Other benefits and drawbacks are provided in this section. The switch to chlorine dioxide would likely necessitate seven day operation of the WTF as chlorine dioxide generally has a shorter half life compared to chlorine.

TABLE 5-36
READSBORO - OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Distribution	Effect*
None proposed	1.) Flow Pace Chemical	Low or Medium
	2.) Booster chlorination	Medium
	3.) Automatic flushing device	Low
	4.) Switch to chlorine dioxide	High

*Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-34A
READSBORO EXISTING PROCESS FLOW DIAGRAM

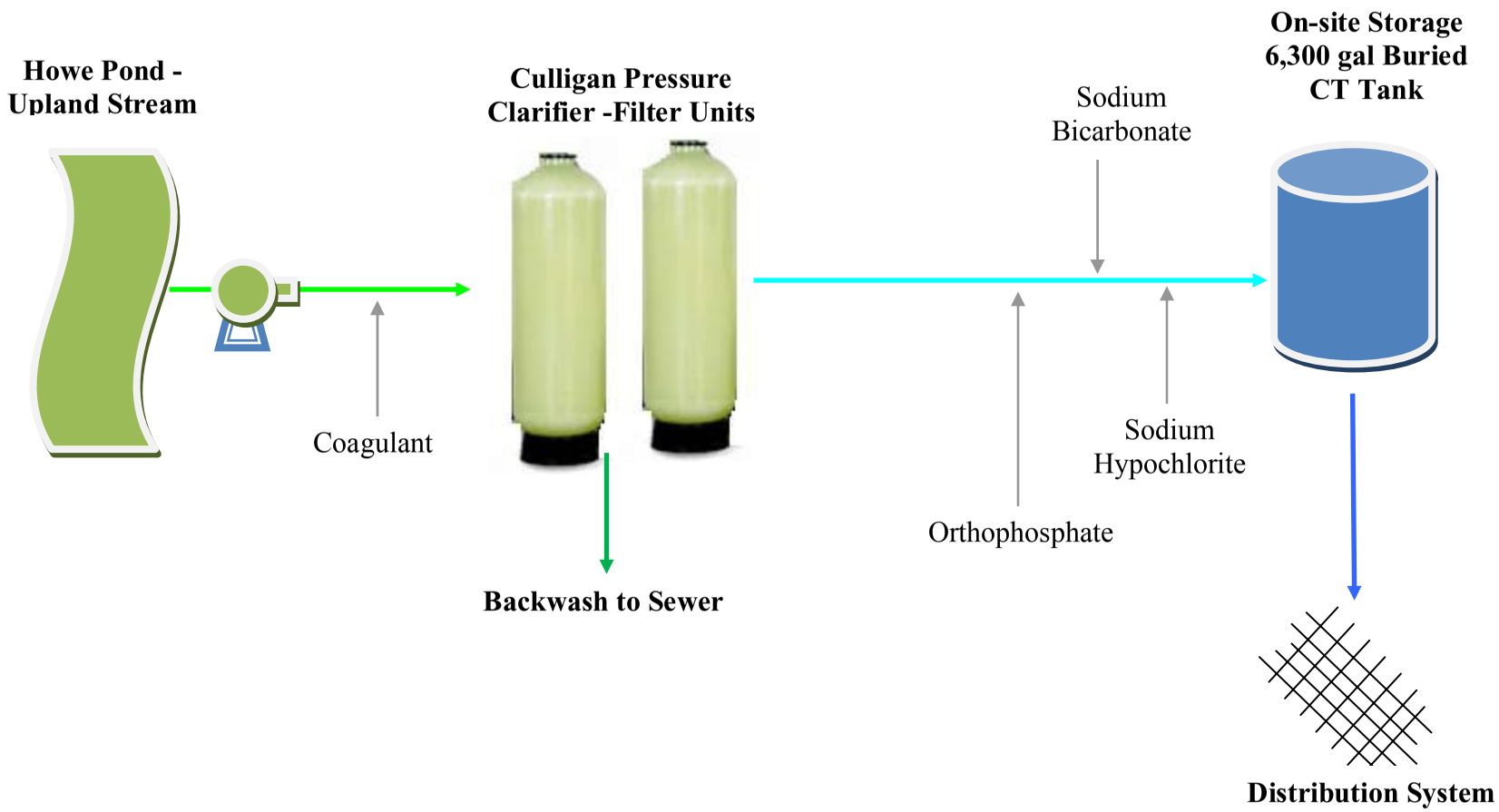


FIGURE 5-34B
READSBORO PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

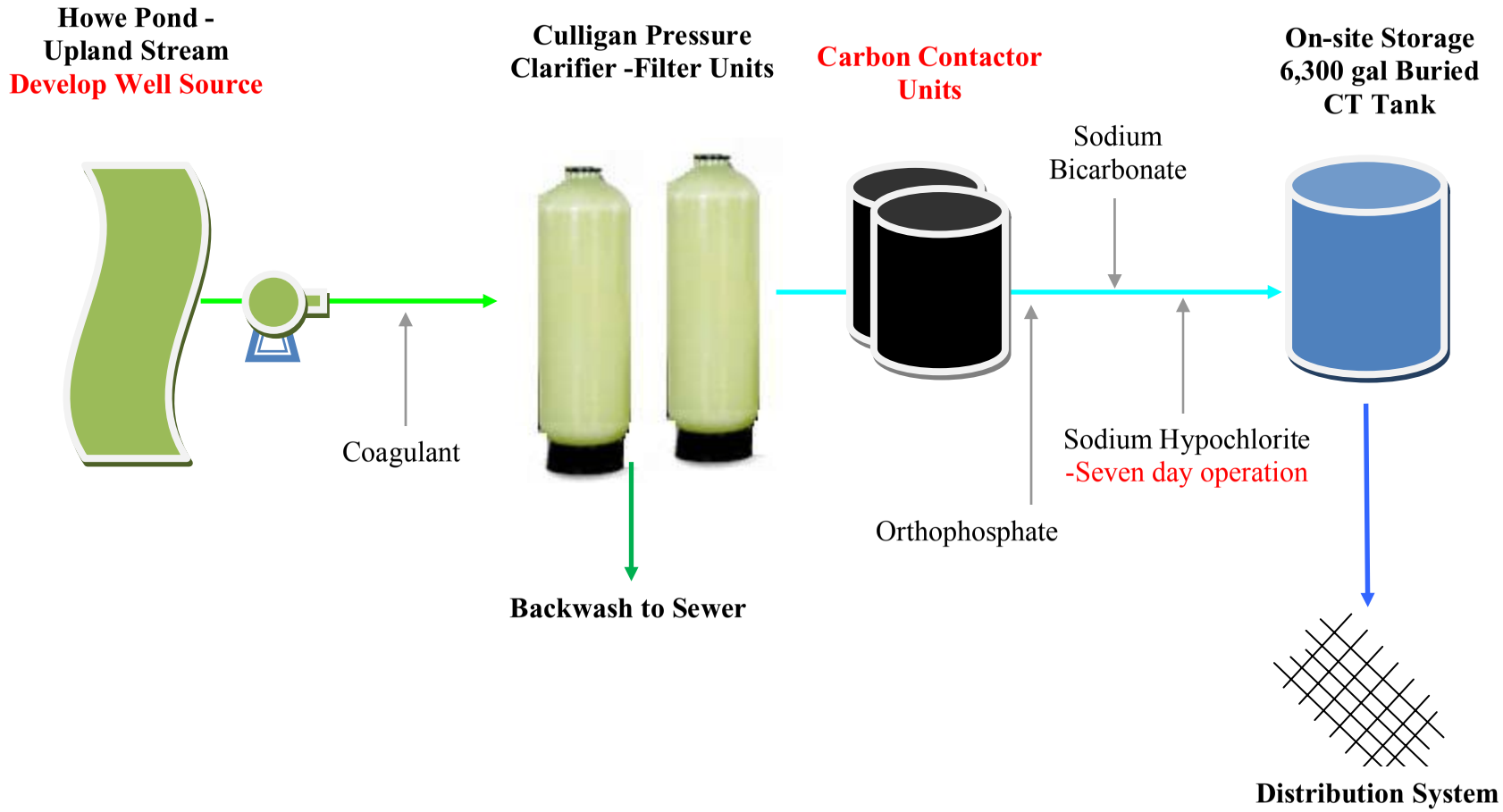


FIGURE 5-34C
READSBORO PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

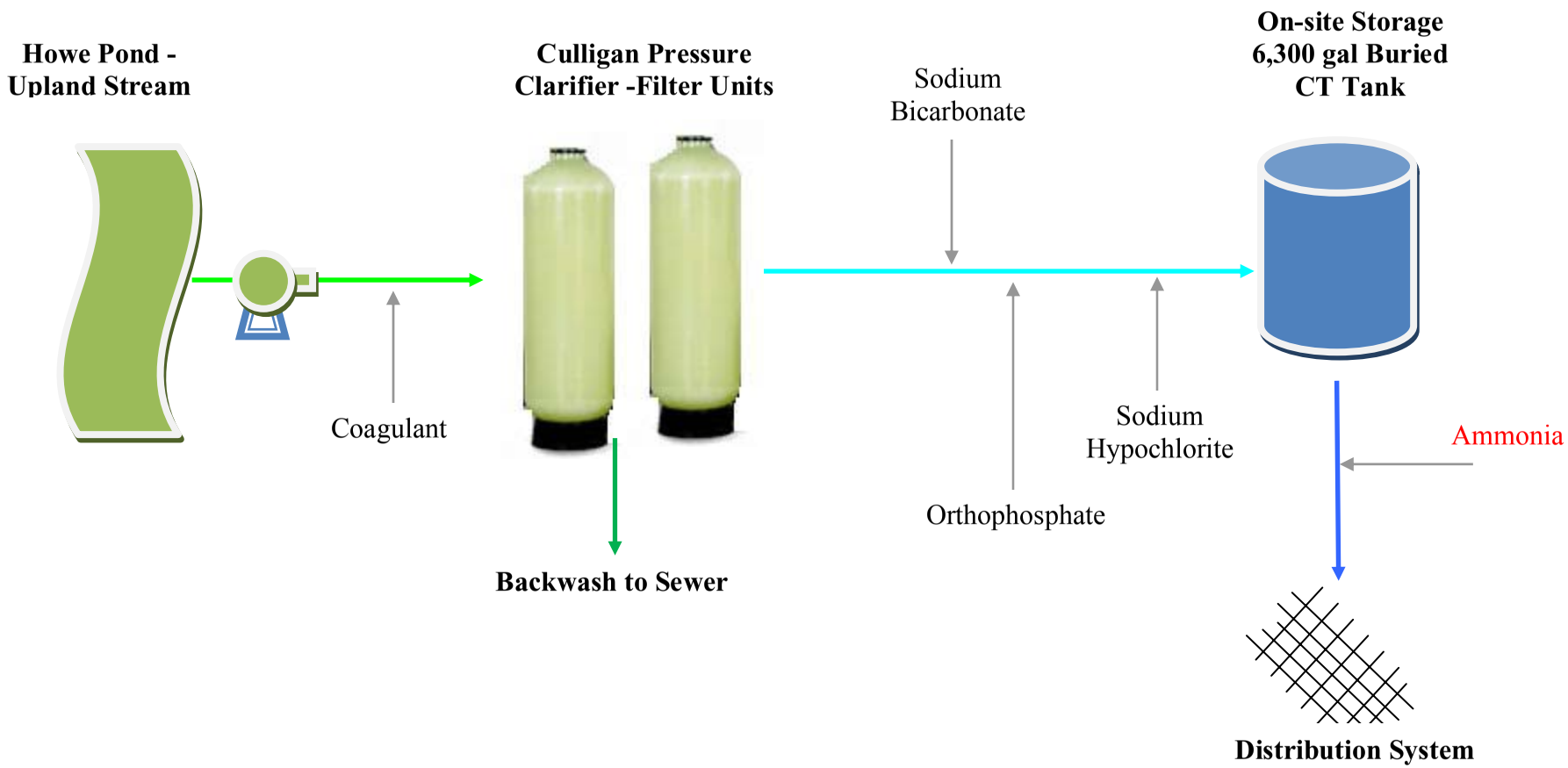
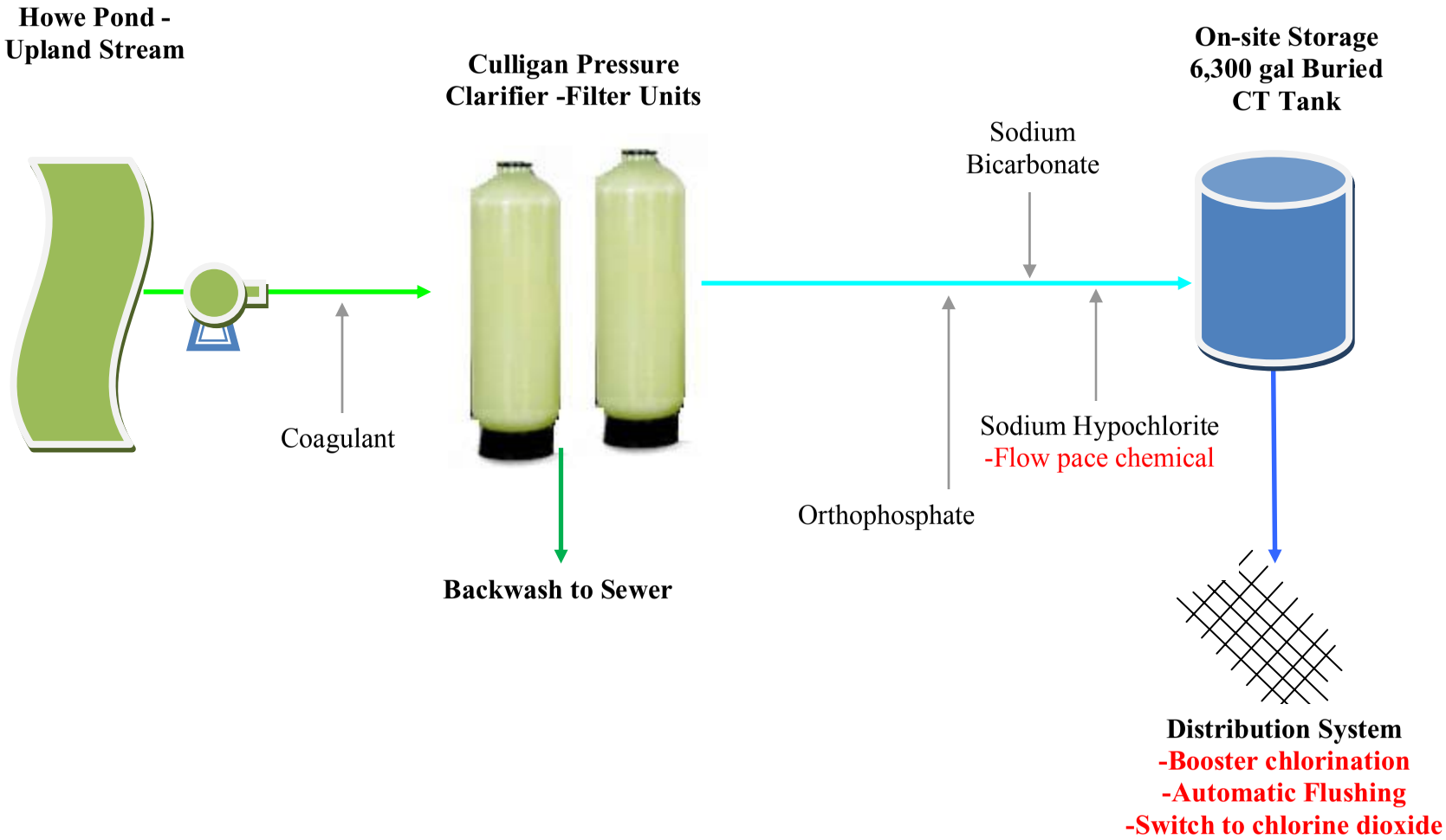


FIGURE 5-34D
READSBORO PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTION S(UPDATES IN RED)



5.2.2.28 Richford - Existing System

The treatment summary for the existing system is provided below and in Table 5-37. Figure 5-37A provides a general treatment schematic of the existing system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source is a low alkalinity upland stream.

Treatment: Treatment consists of slow sand filtration.

Distribution: There are no consecutive systems.

Operator: There is a part time operator at WTF.

VT DEC Observations: Rain events provide poorer quality water.

AECOM Observations: Richford was unable to make a date for an on-site visit with AECOM. As such, the recommendations here are based on limited data which was provided by the Vermont Department of Environmental Conservation.

TABLE 5-37
RICHFORD SYSTEM SUMMARY

Population			
Value	1,700		
System Demand, MGD			
	Typical		
Average	0.159		
Distribution System Characteristics			
Parameter	Information not provided		
Miles of Pipe	---		
Approximate System Detention Time	---		
Number of Storage Tanks	---		
-Provide Size of each, MG	---		
-Tank Type	---		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	No		
Chlorination	12.5% Sodium hypochlorite	1.04 to 1.30 Residual	Dry
Corrosion Control	Soda ash	Not provided	

5.2.3.29 Richford - Proposed Toolbox System

AECOM recommends that Richford follow the steps provided in Table 5-38 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-35 and 5-36 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Richford will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Richford. The summer DBP levels for calibration were 82 ppb for TTHM and 42 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 69 ppb, HAA5 = 38 ppb
2. UV and post clearwell chloramination: TTHM = 55 ppb, HAA5 = 22 ppb
3. UV and pre clearwell chloramination: TTHM = 15 ppb, HAA5 = 6 ppb

Chlorine as Distribution Residual (Figure 5-37B)

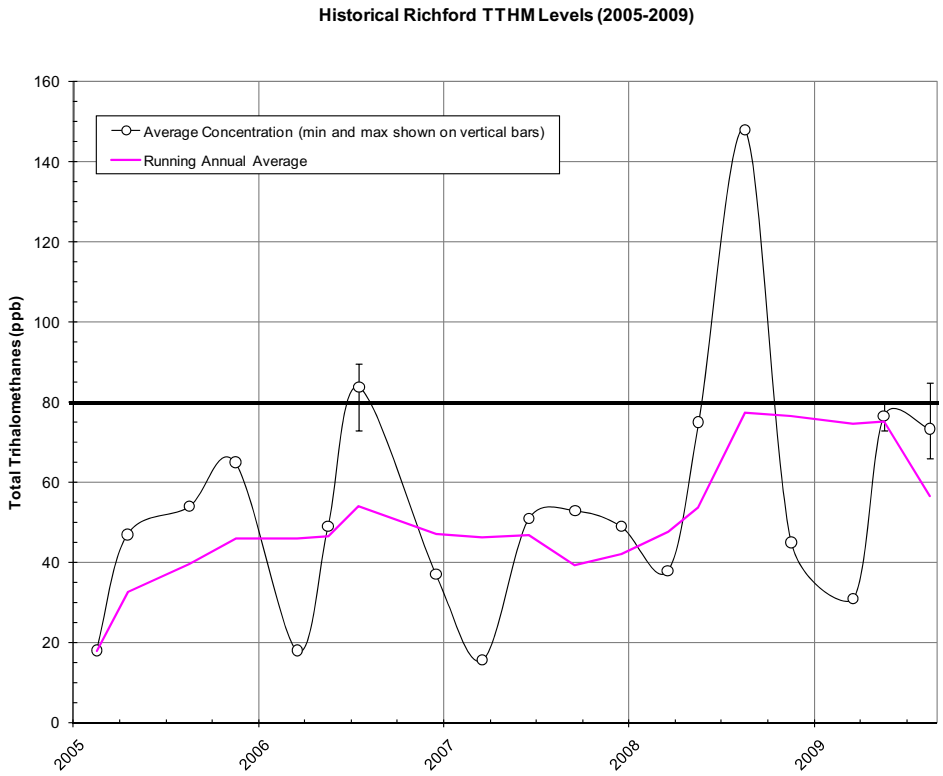
Richford has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Currently, all the sodium hypochlorite is dosed prior to a 0.95MG, two cell tank. This tank alone can provide up to 5 days of stage at average daily demands. Richford should explore splitting the chlorine dose so that the amount of chlorine required for distribution is dosed after the onsite storage tank.
2. Seasonal use of GAC absorbers after filtration. Typically, Richford sees the distribution DBPs spike in the warmer, summer months. Seasonal use of GAC will allow Richford to target TOC removal and ultimately reduce the amount of DBPs which form.

Chloramine as the Distribution Residual (Figure 5-37C)

Richford has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to a level below MCLs.
2. Installation of UV to reduce chlorine required for primary disinfection purposes.



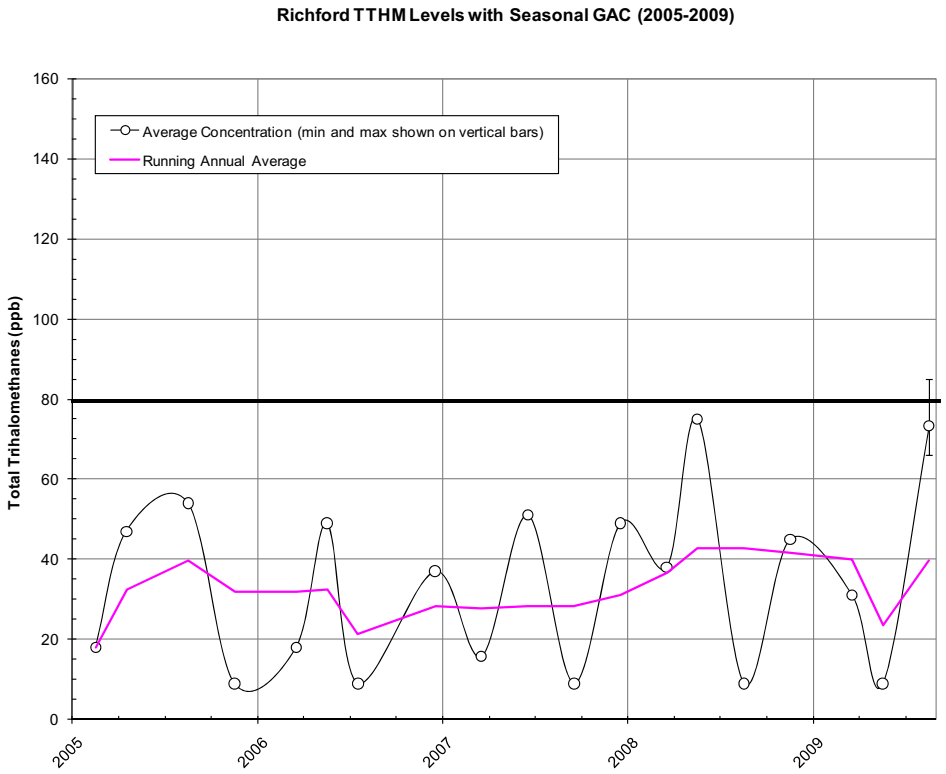
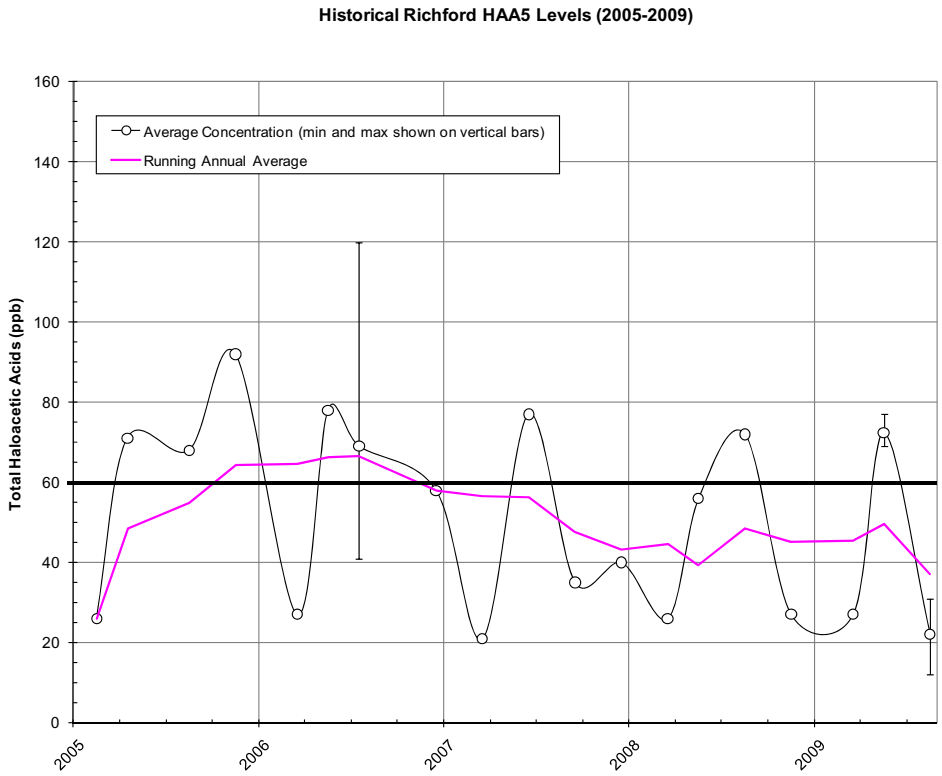


FIGURE 5-35
RICHFORD TTHM LEVELS (2006-2009)



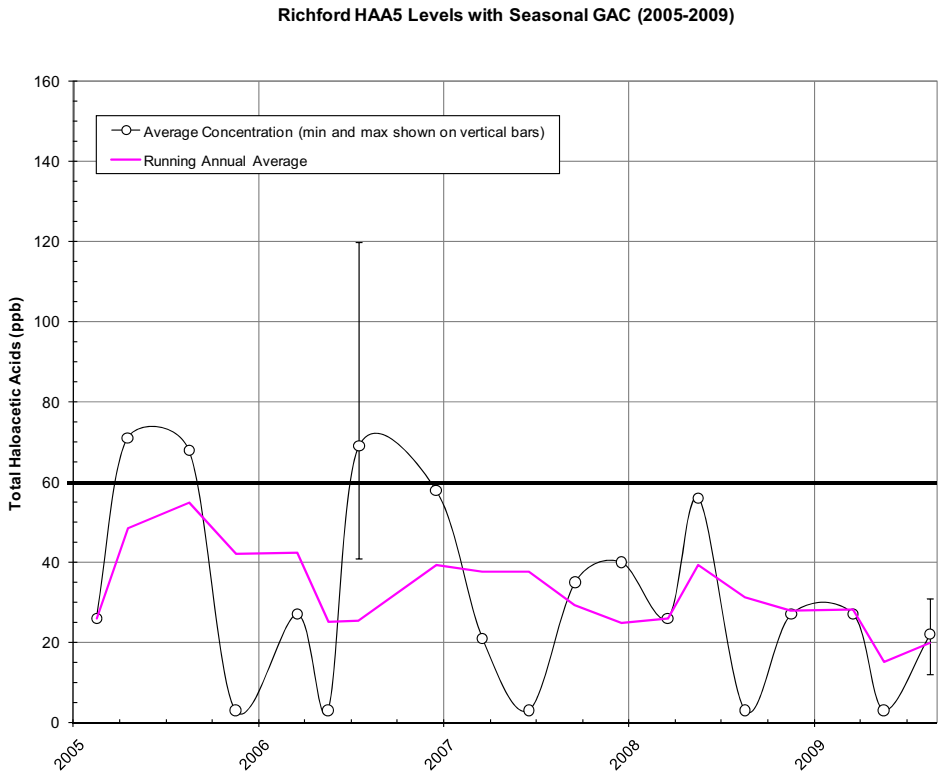


FIGURE 5-36
RICHFORD HAA5 LEVELS (2006-2009)

TABLE 5-38
PROPOSED ALTERNATIVES FOR RICHFORD

Treatment- Existing System	Distribution
For Chlorine	
1.) Break up chlorination between disinfectant and distribution residual	None proposed
2.) Seasonal use of GAC absorber with an EBCT of 20 minutes	
For Chloramine	
1) Install UV disinfection after filtration	1.) Chloramines

5.2.3.30 Richford - Other DBP Reduction Alternatives (Figure 5-37D)

A summary of other DBP reduction alternatives for Richard is provided in Table 5-39. Richford can manage the age in the onsite tank to produce lower age in the over system. This can be done by lowering water levels in the tank or operating only one cell of the tank.

TABLE 5-39
RICHFORD OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Distribution
1.) Reduce onsite storage by managing water levels or cells in operation	None proposed

FIGURE 5-37A
RICHFORD EXISTING PROCESS FLOW DIAGRAM

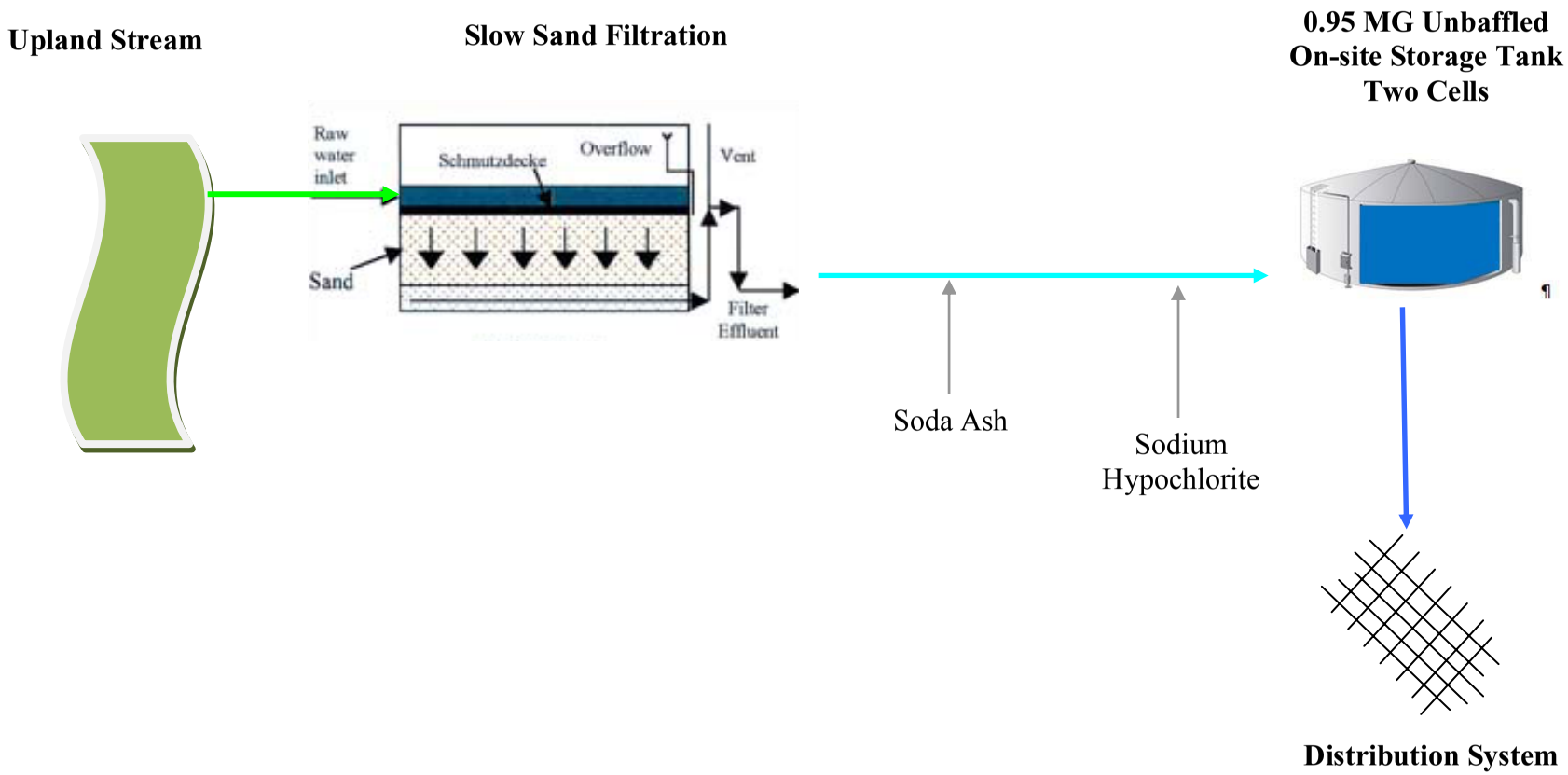


FIGURE 5-37B
RICHFORD PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

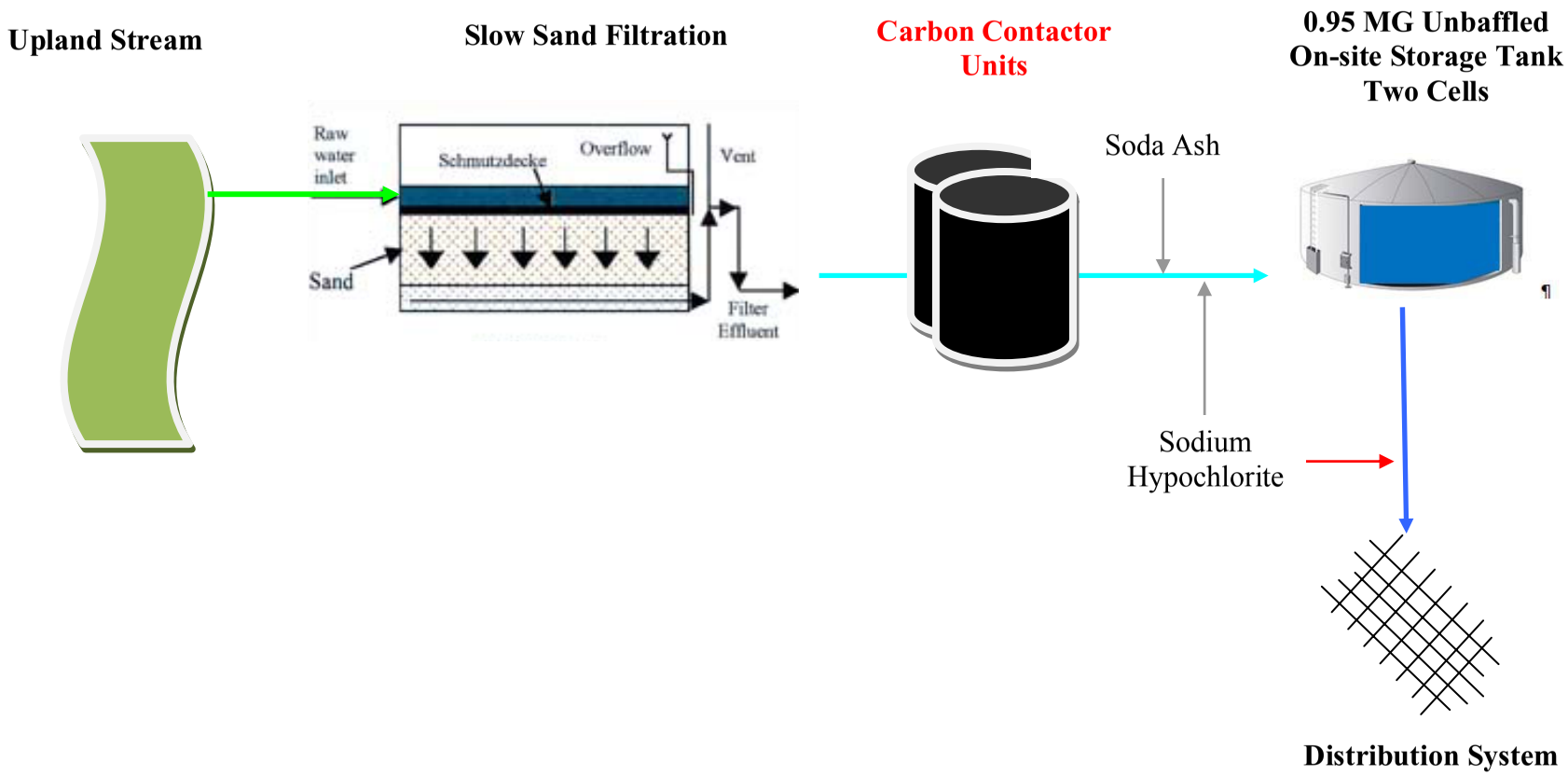


FIGURE 5-37C
RICHFORD PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

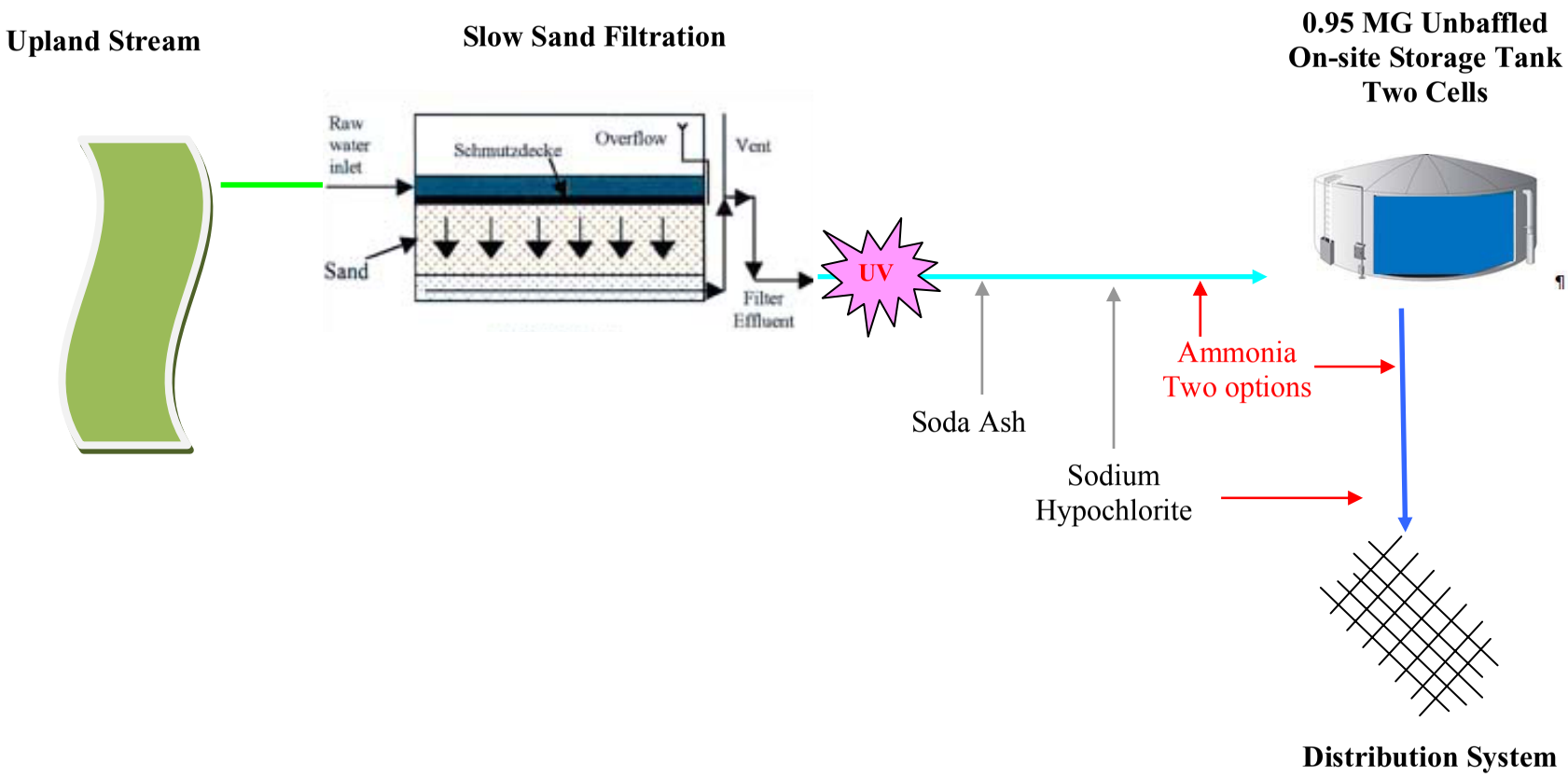
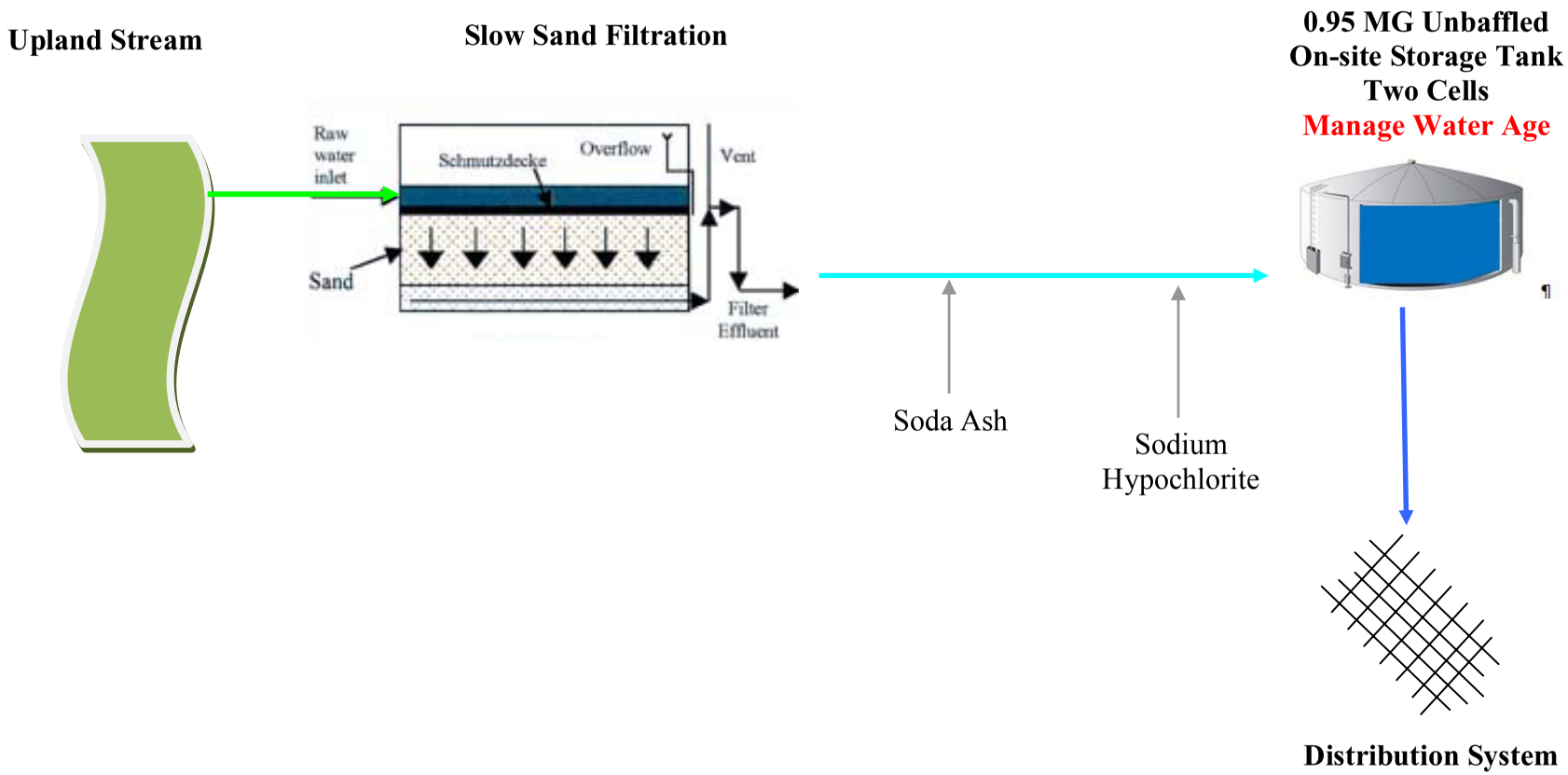


FIGURE 5-37D
RICHFORD PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.2.3.31 Rutland City - Existing System

The treatment summary for the existing system is provided below and Table 5-40. Figure 5-40A provides a schematic of the existing treatment system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The City obtains their source from an open reservoir which is fed by Mendon Brook. The City has enough capacity in the reservoir that Mendon Brook can bypass the reservoir during poor water quality events.

Treatment: Water flows by gravity from the reservoir to the slow sand filtration. Water from the sand filters flows to a wetwell and from there to two 2.5 MG concrete reservoirs followed by gravity flow to distribution system. Sodium hypochlorite, fluoride, and orthophosphate are added prior to the reservoirs. The slow sand filters are wet and dry harrowed every 2-3 months during warmer temperatures and less frequent in the cooler, winter periods. The top foot of sand is harrowed. Sand replacement is rare, with only one filter having the top foot of sand replaced in 15 years. The top foot of sand is harrowed. Sand replacement is rare, with only one filter having the top foot of sand replaced over 15 years. Water during the filter ripening period is sent to an old unlined reservoir. There is no three phase power at the WTF.

Distribution: The system is moderate in size and in general, well looped. The system is flushed twice a year. In addition, during the winter months, water is bled off to keep services from freezing. There are 9 consecutive systems ranging in population of 29 to 1,500.

Operator: Two full time operators run the WTF. The WTF runs 24 hours a day, seven dates a week. The WTF runs unmanned during specified periods of the day.

VT DEC Observations: Reservoir water quality is consistently good.

AECOM Observations: There is an excessive amount of water in storage at the WTF. It is noted that greater than two-thirds of the overall DBPs form at the WTF. In addition, the specific DBPR sampling points, such as the High School, suffer from low flows during late summer.

TABLE 5-40
RUTLAND SYSTEM SUMMARY

Population			
Value	18,500		
System Demand, MGD			
	October, 2008	February, 2009	August, 2009
Minimum	1.3	1.4	1.4
Average	2.2	2.2	2.3
Maximum	7.1	4.6	5.6
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	110		
-Estimated average diameter, inches	8 inch Range 3 inch to 20 inch	2/3 is Cast Iron Remaining is AC or DI pipe.	
Approximate System Detention Time	2 to 4 days		
Number of Storage Tanks	None		
-Provide Size of each, MG	N/A (Located on site)		
-Tank Type	N/A		
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Chlorination	12.5% Sodium hypochlorite	1.0 to 1.3 (Residual)	Dry
Corrosion Control	30% Orthophosphate	0.36 to 0.47	Dry

5.2.3.32 Rutland - Proposed Alternatives

AECOM recommends that Rutland follow the steps provided in Table 5-41 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

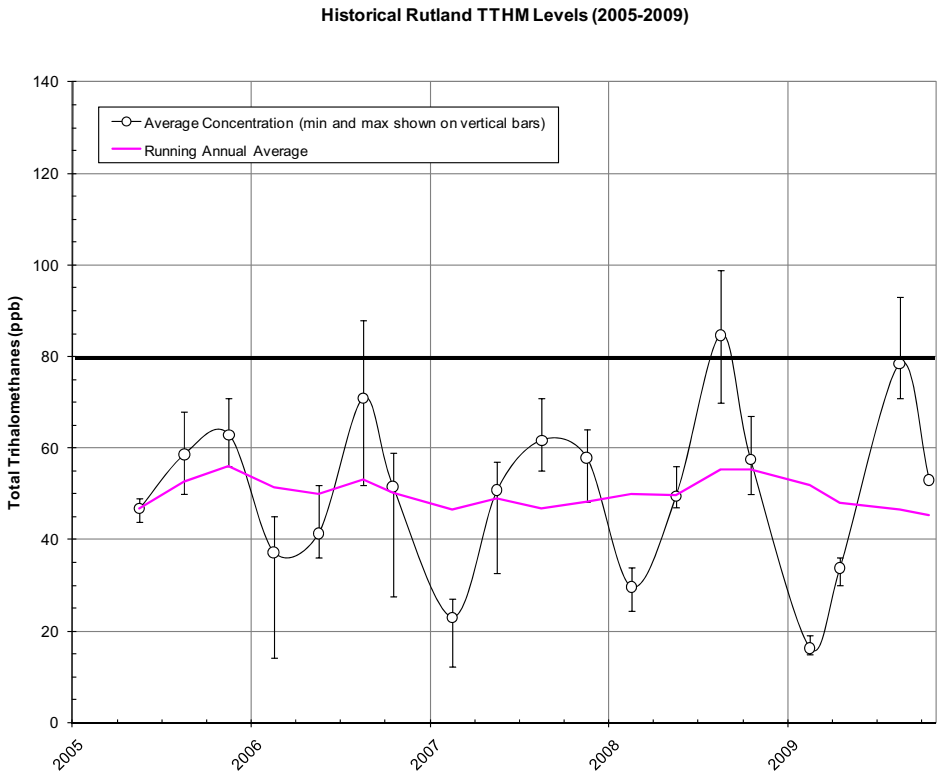
Figures 5-38 and 5-39 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Rutland will be required to operate the recommended treatment for 6 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Rutland. The summer DBP levels for calibration were 90 ppb for TTHM and 101 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 72 ppb, HAA5 = 87 ppb
2. UV and post clearwell chloramination: TTHM = 63 ppb, HAA5 = 65 ppb
3. UV and pre clearwell chloramination: TTHM = 15 ppb, HAA5 = 12 ppb

Chlorine as Distribution Residual (Figure 5-40B)

Rutland has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Seasonal use of GAC with an EBCT of 20 minute after the sand filters. Since the sampling sites see a peak DBP formation in warmer months of the year (e.g. July-November), the facility can implement filtration after the slow sand filters to remove TOC prior to the addition of chlorine.
2. Install MIEX to remove a portion of the raw TOC. Install a wellfield to lower TOC. The site on which the water treatment facility is located is mainly sand. Installing wells next to the reservoir may allow for a reduction in raw TOC levels and also may produce TOC which is less reactive with chlorine due to the natural action of filtering and biological activity.



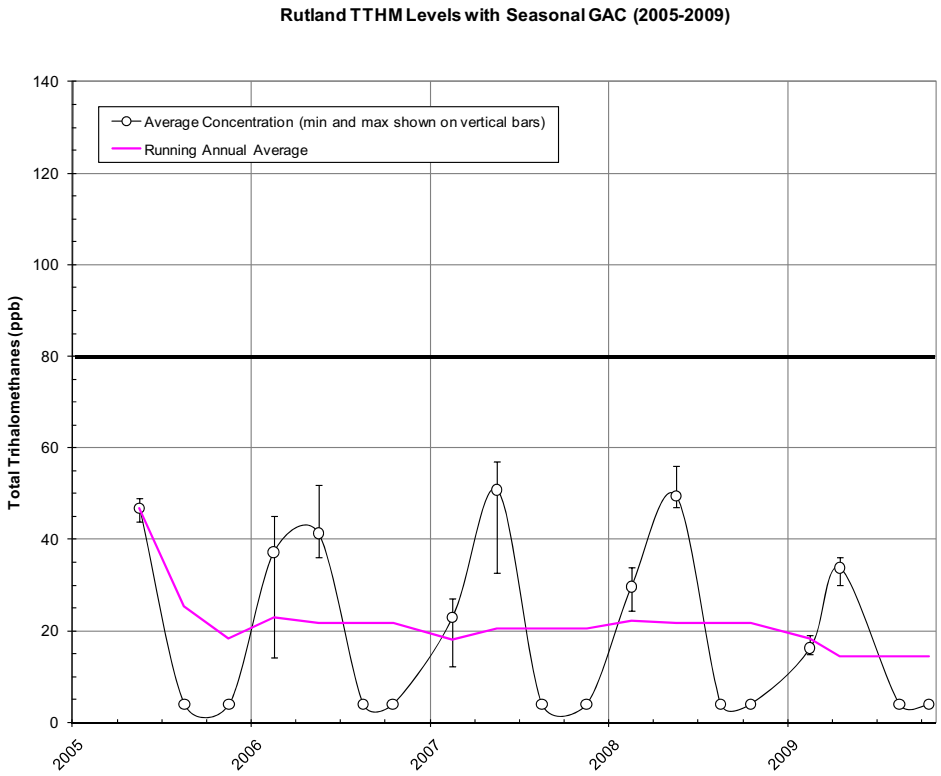
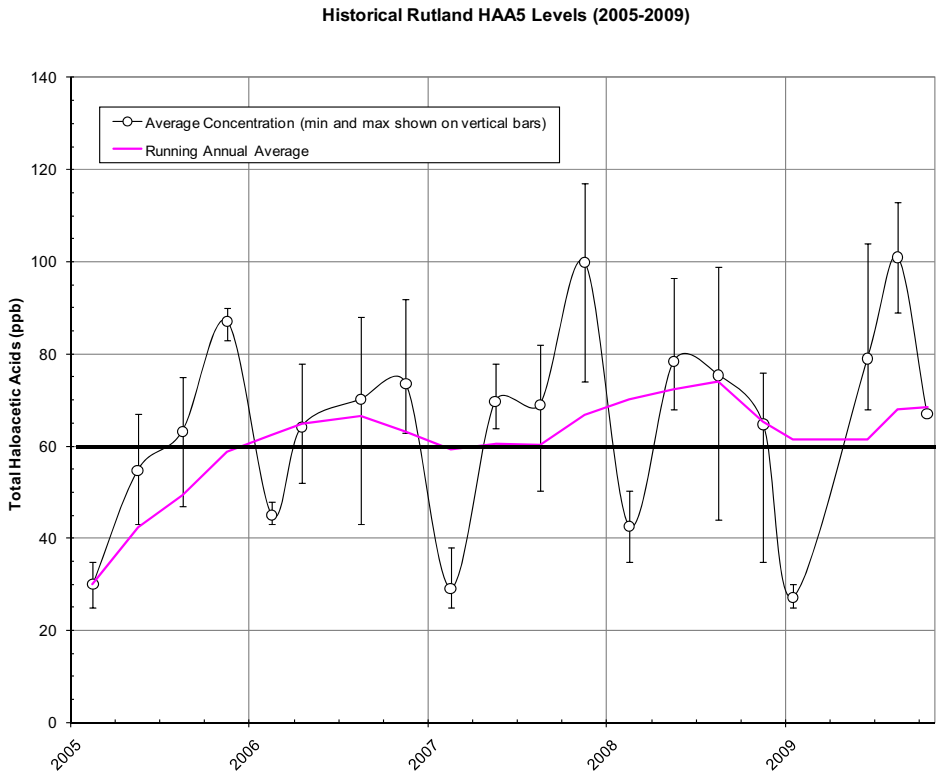


FIGURE 5-38
RUTLAND CITY TTHM LEVELS (2006-2009)



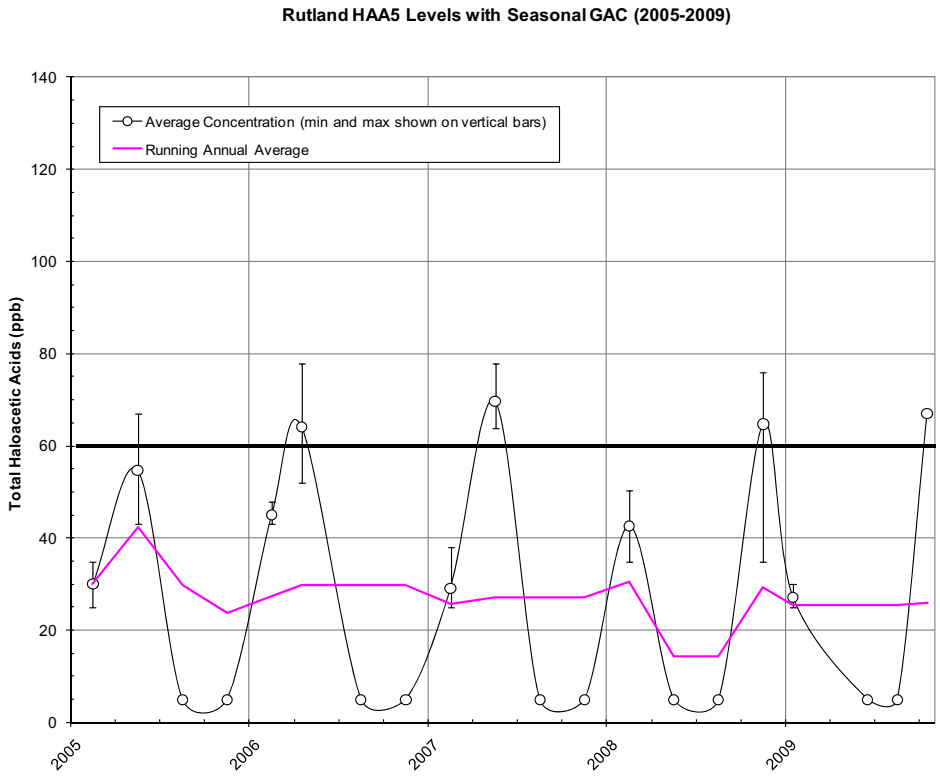


FIGURE 5-39
RUTLAND CITY HAA5 LEVELS (2006-2009)

Chloramine as the Distribution Residual (Figure 5-40C)

Rutland has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.
2. Installation of UV to reduce chloramine required for primary disinfection purposes. Add chloramine as secondary disinfectant.

TABLE 5-41
PROPOSED ALTERNATIVES FOR RUTLAND

Treatment- Existing System	Distribution
For Chlorine	
1.) Seasonal GAC absorption with an EBCT of 20 minutes after slow sand filters	None proposed
2.) Wells or MIEX	
For Chloramine	
1) Install UV disinfection after filtration to reduce disinfection requirements for chloramines.	1.) Chloramine

5.2.3.33 Rutland - Other DBP Reduction Alternatives (Figure 5-40D)

A summary of other DBP reduction alternatives for Rutland is provided in Table 5-42. The following provides a brief discussion of each alternative.

1. There is substantial water age on site. The current operation doses chlorine after filtration and prior to the two 2.5 MG storage tanks. The disinfection sampling point is after the 2.5 MG storage tanks. In addition, enough chlorine is dosed so that not only can disinfection be achieved on site, but that a residual can be achieved at distant points in the distribution system. Implementing a second chlorination injection point after the onsite tanks will allow an overall lower time in which chlorine is in contact with TOC. Similar to this, water levels or tank in service can be varied to reduce water age.

a. Another method of reducing the overall time in which chlorine is in contact with TOC is to target booster chlorination in the system. For example, booster chlorine at the entrance to a consecutive system can lower the chlorine dose which is applied at the WTF.

2. Specific DBPR sampling points, such as the High School, suffer from low flows during later summer and as a result tend to have high DBP level in comparison to other sampling sites. Implementing an automatic flushing program will aid in preventing water from stagnating in this area and reduce the time chlorine is in contact with TOC.

TABLE 5-42
RUTLAND - OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Add chlorine injection point after onsite tanks	Medium	1.) Booster chlorination	Low

*Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

** Site specific

FIGURE 5-40A
RUTLAND CITY EXISTING PROCESS FLOW DIAGRAM

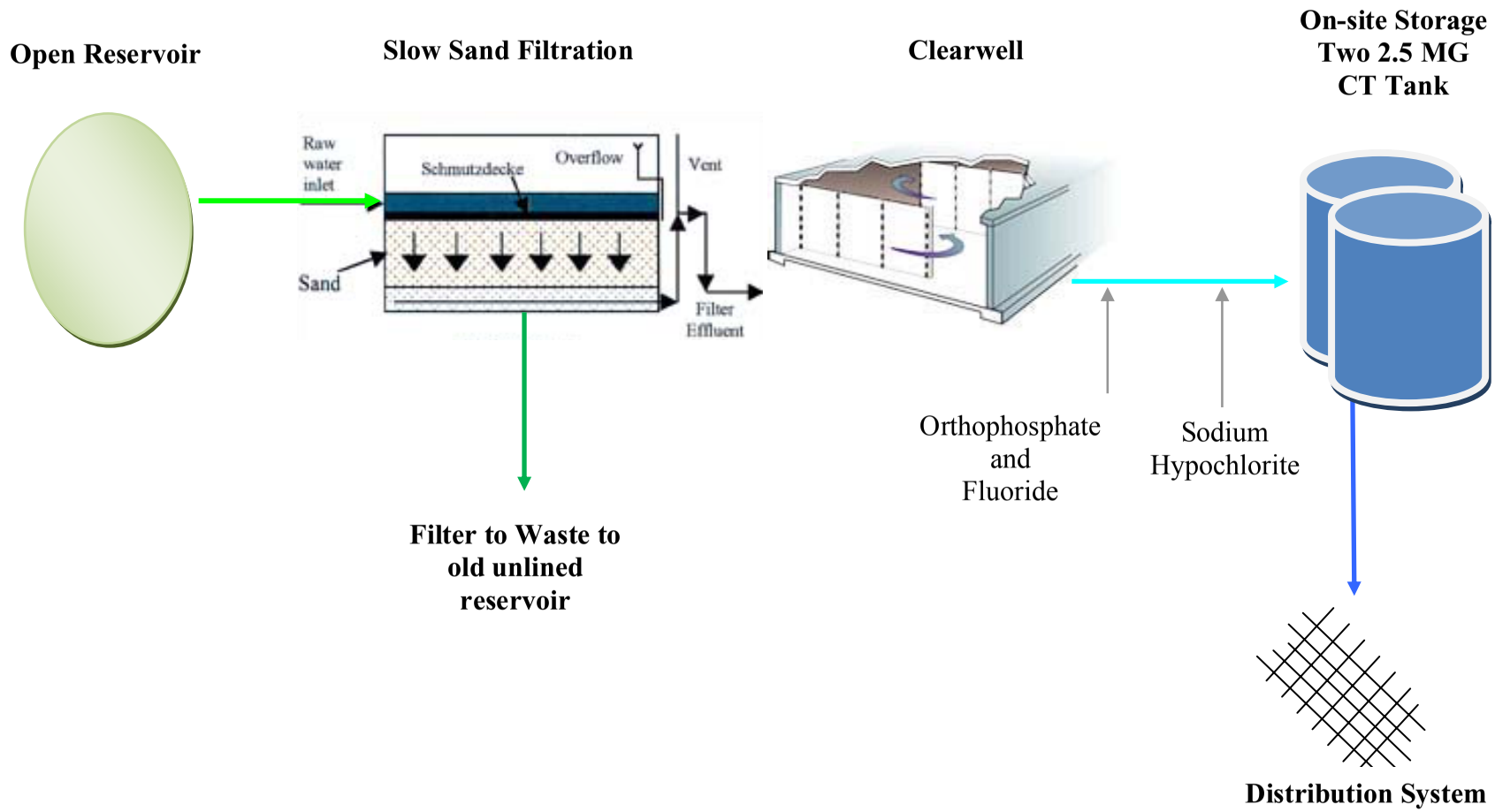


FIGURE 5-40B
RUTLAND CITY PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

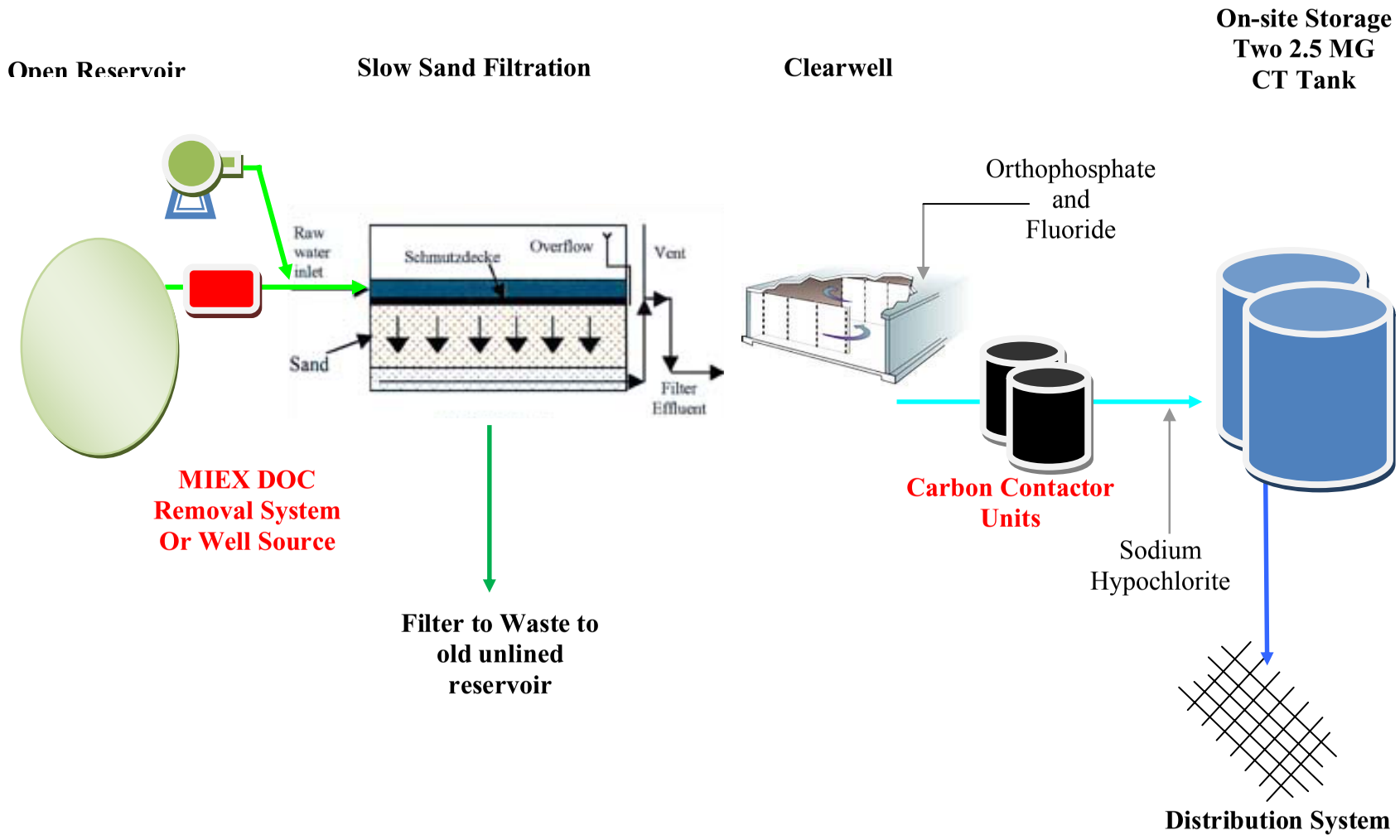


FIGURE 5-40C
RUTLAND CITY PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

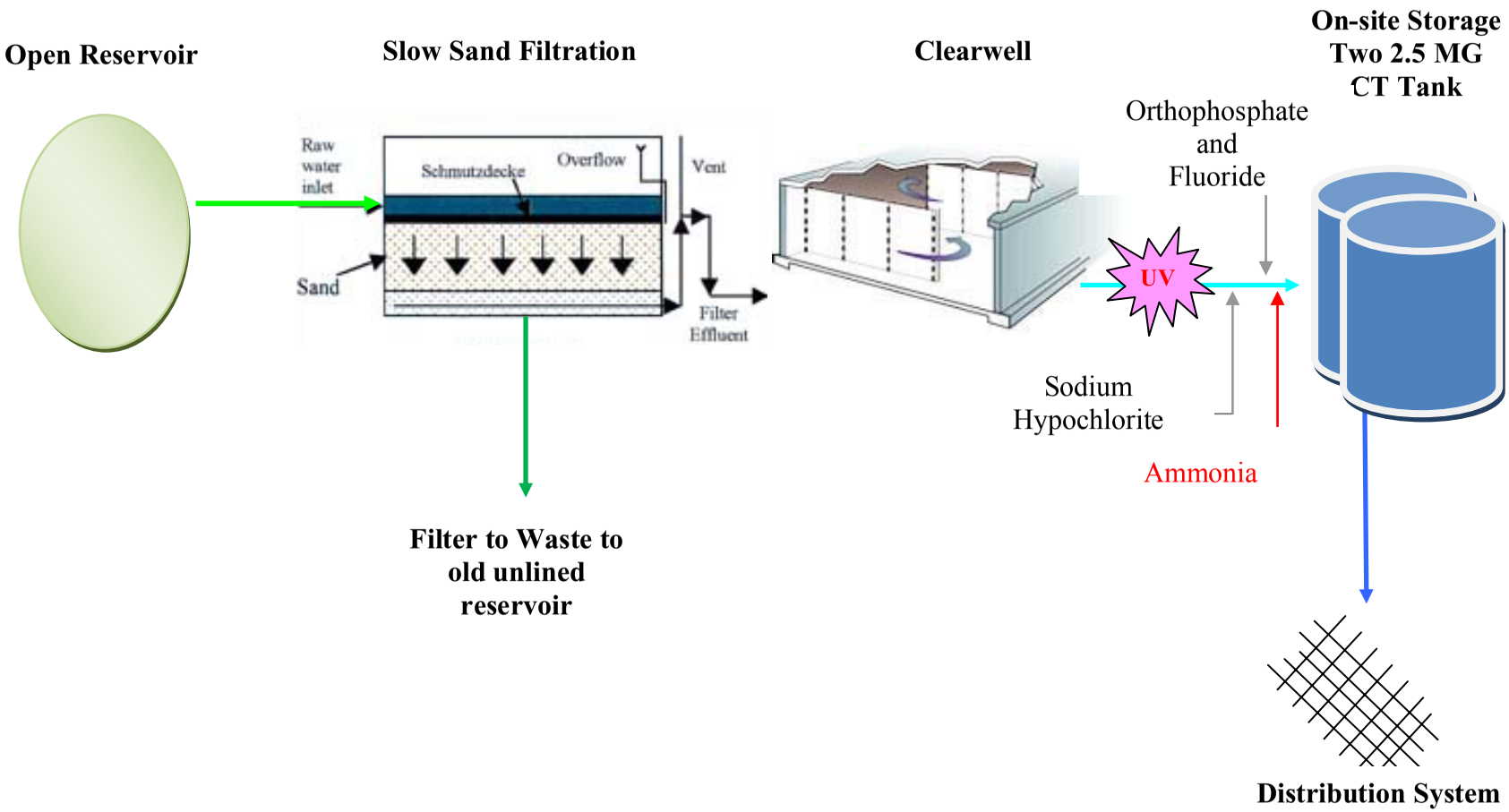
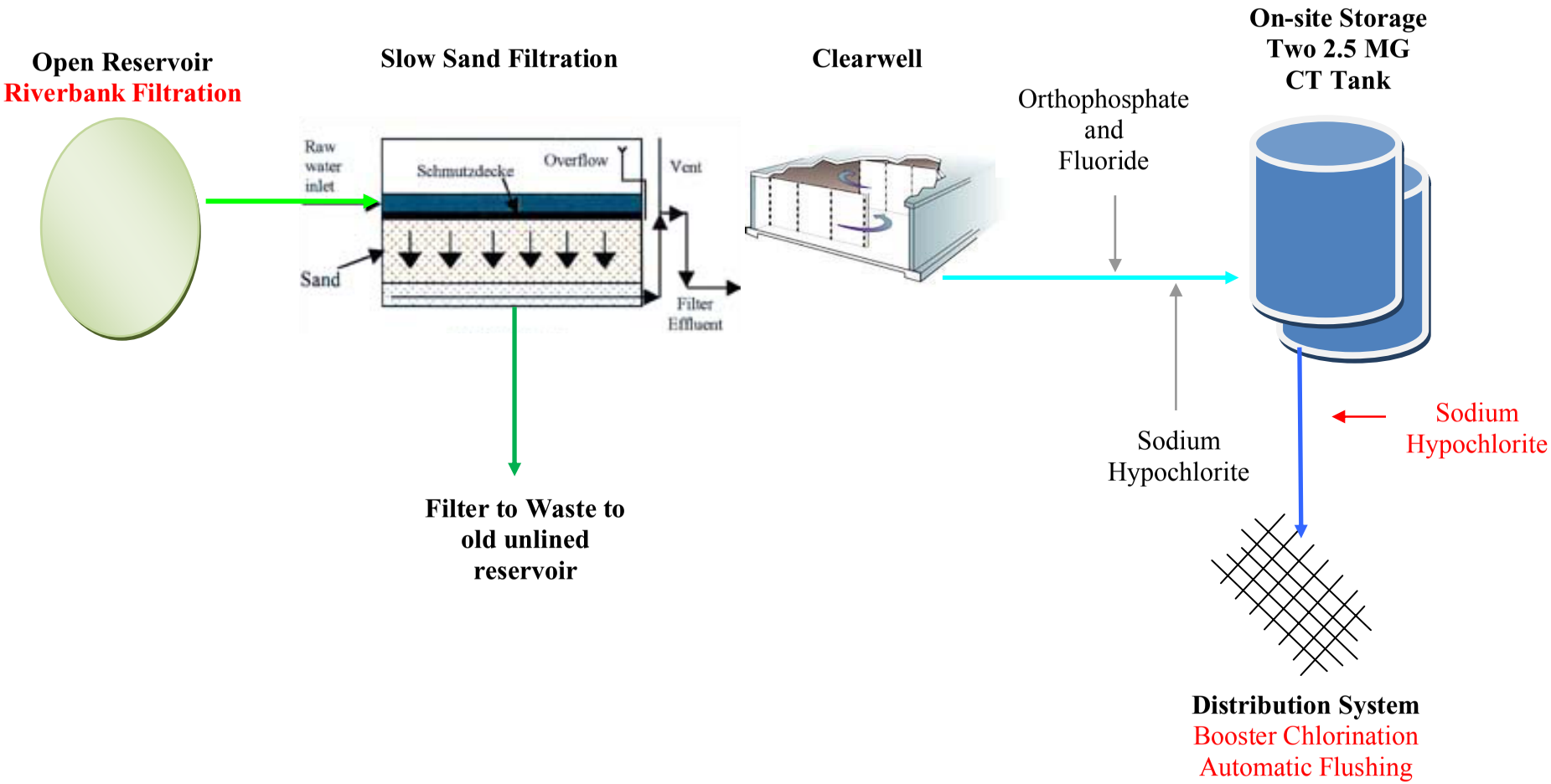


FIGURE 5-40D
RUTLAND PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.2.3.34 St. Johnsbury - Existing System

The treatment summary for the existing system is provided below and in Table 5-43. Figure 5-41A provides a schematic of the existing process system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source for St. Johnsbury is an open reservoir, Stiles Pond. The intake is about 30 feet off shore and in about 10.75 feet deep water. The water experiences significant increases in temperature during the summer. Algae blooms have not been experienced.

There is the Oak Street well in the distribution system which is an emergency source. The well tends to be hard water with near neutral pH (4/23/09 sample). Iron has not been measured in the source, but sitting water tends to precipitate an orange substance which is believed to be iron. Radon is measured in one sample as 668 pCi/L (5/11/09 sample).

Treatment: The current treatment process is buoyant media clarification followed by mixed media filtration (Trident system). Filtered water is received in plant clearwell. St. Johnsbury was required to recycle a portion of the backwash water (~90,000 gallons/day) due to restrictions in the receiving sewer line. This was discontinued in September, 2009. St. Johnsbury's engineer (Dufresne and Associates) has proposed a new 0.5 MG tank after the clearwell and an emergency lagoon system for filter backwash. The onsite storage tank is to provide more flexibility in providing disinfection time as well as backwash water for the filters. In addition, the existing paddle flow meters are to be replaced with magnetic flow meters. The chemical pacing is constant feed and currently not flow paced. In addition, the WTF operates in cyclic fashion based on a 1.5 foot drop in clearwell.

Distribution: The distribution system mainly consists of cast iron or ductile iron pipe. There are 4 consecutive systems ranging in population from 140 to 370.

St. Johnsbury flushes the distribution system once a year. During the winter periods, St. Johnsbury runs bleeders in the system to prevent lines from freezing. The system is generally well looped except for a few fingers which run out of town. Currently, three chlorine booster stations are being installed to address the long fingers of the distribution system.

Operator: The WTF operates 24 hours a day, seven days a week. There are two certified operators, one full time, and one part time. There are no laborers.

VT DEC Observations: Reservoir water quality is good, but there is a seasonal manganese problem. The system is currently undergoing engineering improvements to provide additional onsite filtered water storage, potassium permanganate feed at the intake and recent provisions to send plant residuals to the wastewater treatment plant.

TABLE 5-43
ST. JOHNSBURY SYSTEM SUMMARY

Population			
Value	3,800		
System Demand, MGD			
	Typical		
Minimum	0.873		
Average	1.11		
Maximum	1.92		
Distribution System Characteristics			
Parameter	Value	Notes	
Miles of Pipe	53.06 miles		
-Estimated average diameter, inches	8.8 inch Dia.		
Approximate System Detention Time	0-499		
Number of Storage Tanks	5		
-Provide Size of each, MG	0.5 MG each		
-Tank Type	Standpipe	Glass Lined Steel	Steel
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	Yes		
Preoxidation*	23% Potassium Permanganate	0.8 (Slight Pink in Clarifier)	Dry
Preoxidation	12.5% Sodium Hypochlorite	Target residual < 1.0 (See disinfection: total Cl_2 dose)	Dry
Coagulant	48.5% Alum	30 - Summer 25 - Winter	Dry
Polymer	Nonionic	0.7 to 1.0	Wet
Disinfection chlorine**	12.5% Sodium hypochlorite	Dose: 5.5 - Summer, 3.5 – Winter. Residual: 2.0 (Range1.1 - 2.5)	Dry
Corrosion Control	25% Sodium Hydroxide	Target pH 7.2 (range 7.2 to 8.0)	
Dental health	26% Fluoride	1.0	Dry

*Based on a usage of 55 gallons over two weeks for summer and 55 gallons over three weeks in the winter at average flow.
**Based on a usage of 30 gallons per day in the summer and 20 gallons per day in the winter at average flow.

AECOM Observations: The water treatment facility is obtaining a hardy floc and appears to be enhanced coagulating. The main problem is that the facility is prechlorinating.

5.2.3.35 St. Johnsbury - Proposed Alternatives

AECOM recommends that St. Johnsbury follow the steps provided in Table 5-44 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

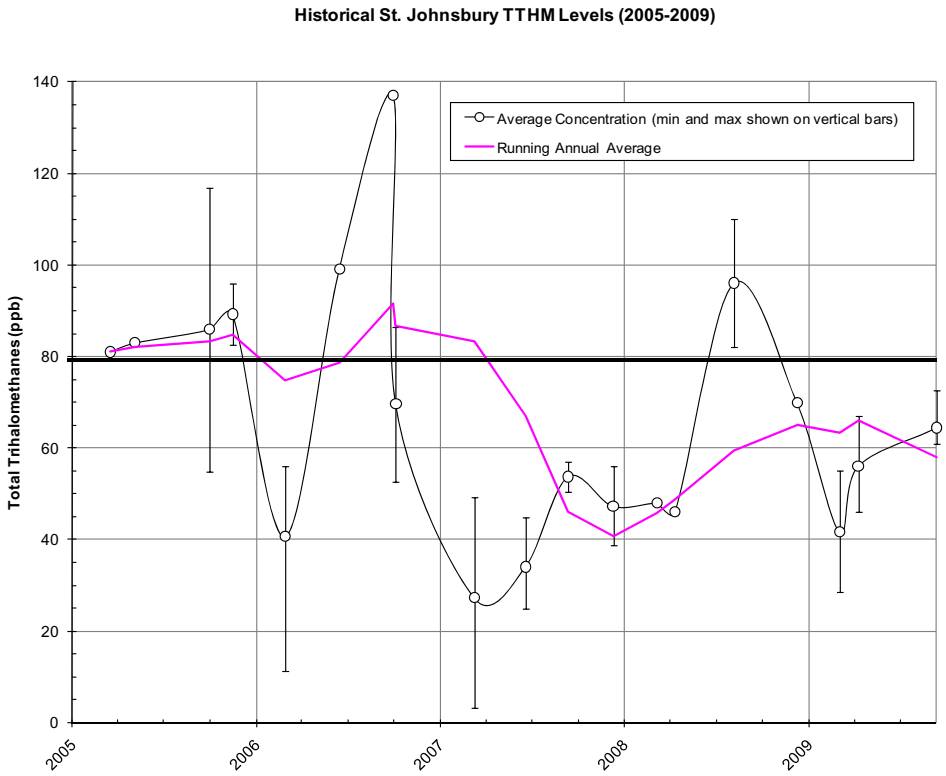
Figures 5-41 and 5-42 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that St. Johnsbury will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under St. Johnsbury. The summer DBP levels for calibration were 66 ppb for TTHM and 58 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 58 ppb, HAA5 = 54 ppb
2. UV and post clearwell chloramination: TTHM = 19 ppb, HAA5 = 12 ppb

Chlorine as Distribution Residual (Figure 5-43)

St. Johnsbury has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Stop prechlorination. Adding chlorine to the clarifier maximizes the contact of chlorine with TOC. Stopping prechlorination and substituting with another oxidant will reduce the amount of DBPs formed at the WTF.
2. Seasonal use of GAC with an EBCT of 20 minute after the sand filters. Since the sampling sites see a peak DBP formation in warmer months of the year (e.g. July-November), the facility can implement GAC contactors after the sand filters to remove TOC prior to the addition of chlorine.



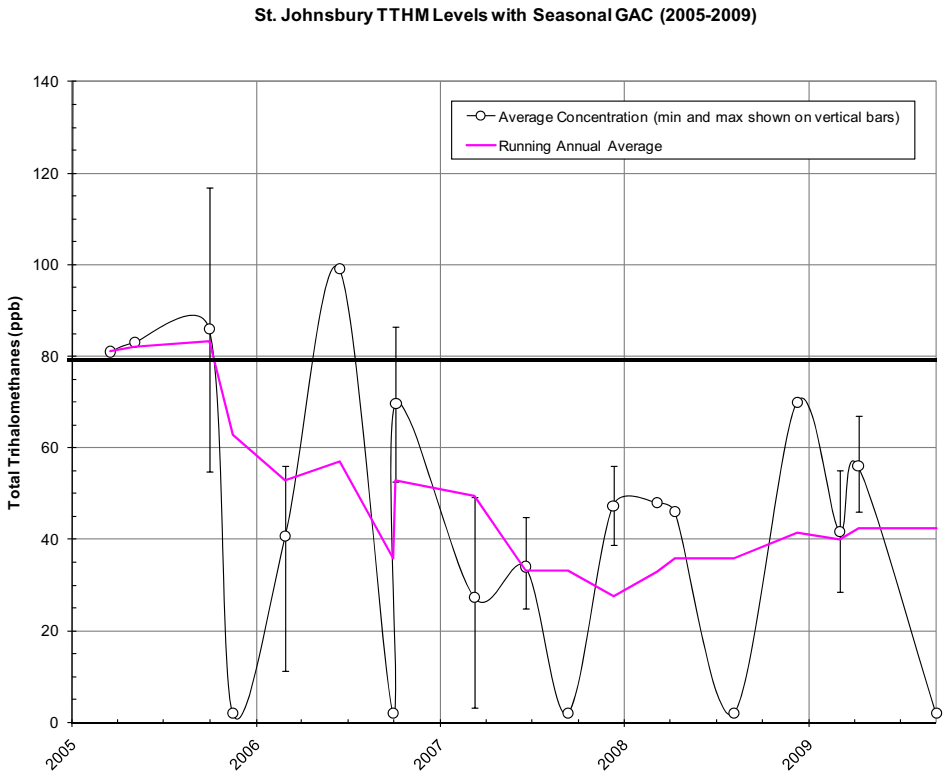
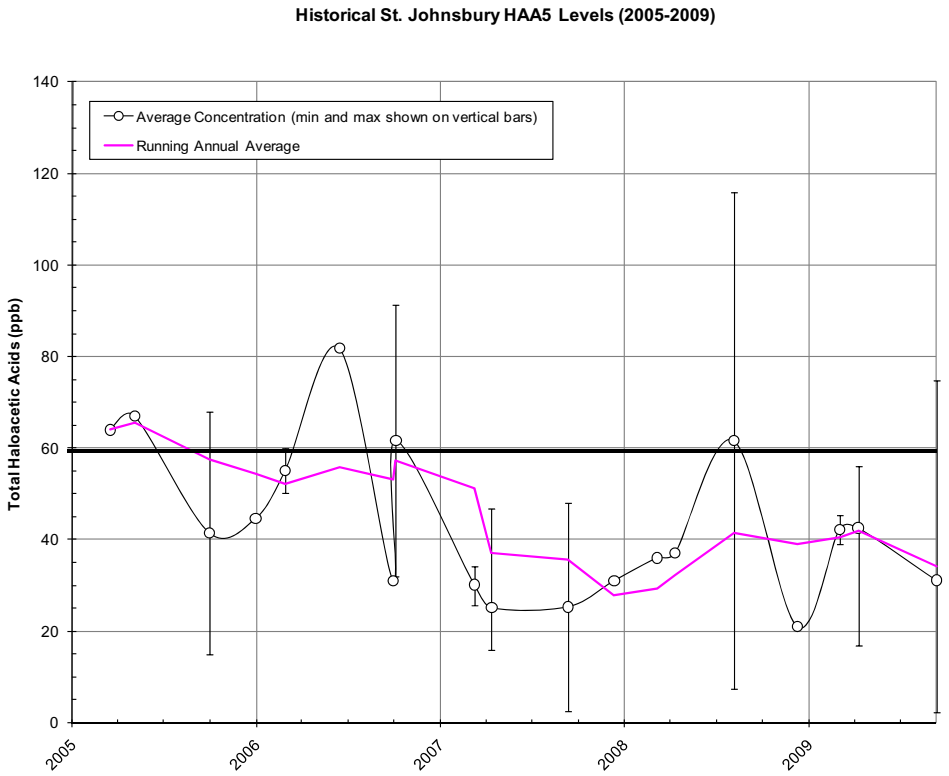


FIGURE 5-41
ST. JOHNSBURY TTHM LEVELS (2006-2009)



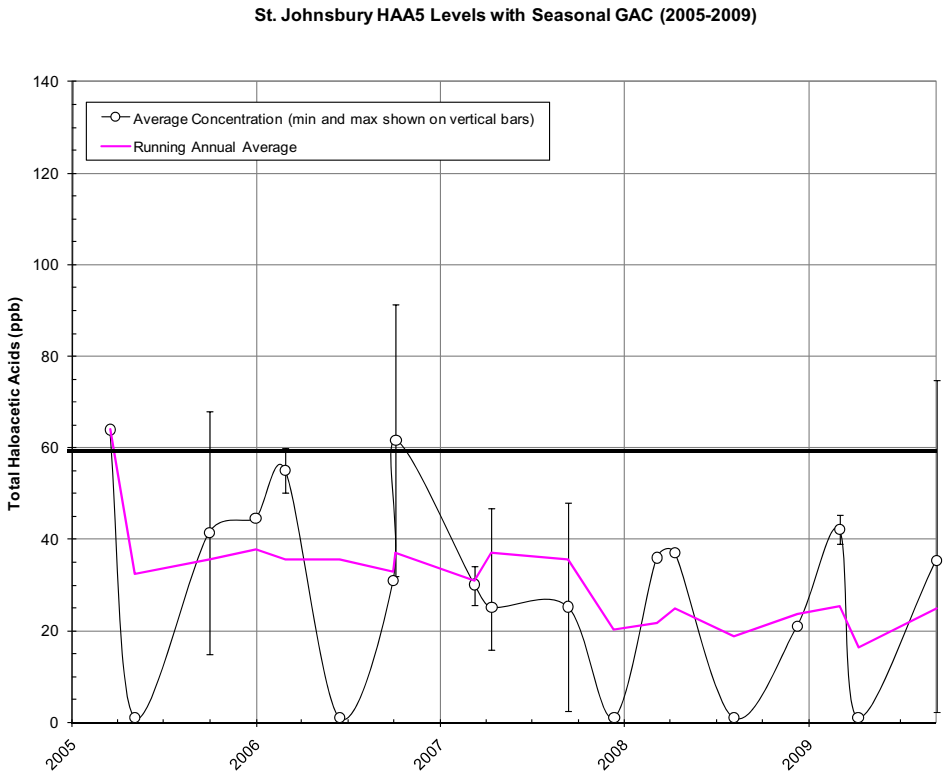


FIGURE 5-42
ST. JOHNSBURY HAA5 LEVELS (2006-2009)

Chloramine as the Distribution Residual (Figure 5-43C)

St. Johnsbury has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.
2. Installation of UV to reduce chlorine required for primary disinfection purposes and further reduces DBPs.

TABLE 5-44
PROPOSED ALTERNATIVES FOR ST. JOHNSBURY

Treatment- Existing System	Distribution
For Chlorine	
1.) Stop prechlorination. Use Alternative pre-oxidant to chlorine	None proposed
2.) Seasonal (summer) post GAC absorber with 20 minute EBCT	
For Chloramine	
1.) Install UV disinfection to reduce chlorine primary disinfection requirements to virus only	1.) Chloramine

5.2.3.36 St. Johnsbury - Other DBP Reduction Alternatives (Figure 5-43D)

A summary of other DBP reduction alternatives for St. Johnsbury is provided in Table 5-45. The following provides a brief discussion of each alternative.

1. Optimize the existing operations to remove TOC. St. Johnsbury should take periodical raw, effluent clarifier and filtered TOC to determine performance of the system as well as making changes to the coagulant dosage as needed.
2. Moving the pH adjustment point after the new 0.5 MG tank will allow the utility to take advantage of a lower pH during disinfection prior to adjustment for corrosion control.
3. Moving the distribution chlorine adjustment point after the new 0.5 MG tank will allow the utility to reduce the overall time that the same mass of chlorine is in contact the treated water.

4. Automatic flushing will reduce the water age in specific areas of the distribution system, similar to the bleeding program in the winter. However, water is not consumed, and could be considered wasted water.
5. Pump well water into the distribution system during the summer months. In reviewing the surficial geologic map of St. Johnsbury, AECOM finds several promising areas along the Passumpsic River, especially where there are mapped eskers deposits. Eskers are generally very coarse grained and highly transmissive features, and their proximity to the river means that there should be ample recharge. We count nine eskers along the river from the very northern boundary to the southern boundary. This will provide a source which is cooler than the existing surface source and presumably, lower TOC and less reactive TOC. However, the following would need to be considered prior to implementation:
   a. Measure source TOC levels and reactivity to chlorine.
   b. Address the iron levels either by sequestering or treatment.
   c. Address the radon levels with either treatment or multi-mitigation management program.
   d. Harder water, while beneficial for corrosion control, may cause some consumer complaints.
   e. Potentially higher electrical costs compared to surface water treatment source.

TABLE 5-45
ST. JOHNSBURY - OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Regular TOC measurements	Medium	1.) Automatic flushing	Low
2.) Move pH adjustment application point to after 0.5 MG onsite tank	Low		
3.) Move distribution chlorination point to after 0.5 MG onsite tank	Low		
2.) Develop well for use during warmer periods	Ranges		

*Effect is the anticipated relative effect the action has on reducing overall DBPs.
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-43A
ST. JOHNSBURY EXISTING PROCESS FLOW DIAGRAM

Note: Proposed improved by St. Johnsbury engineer are shown as dashed line

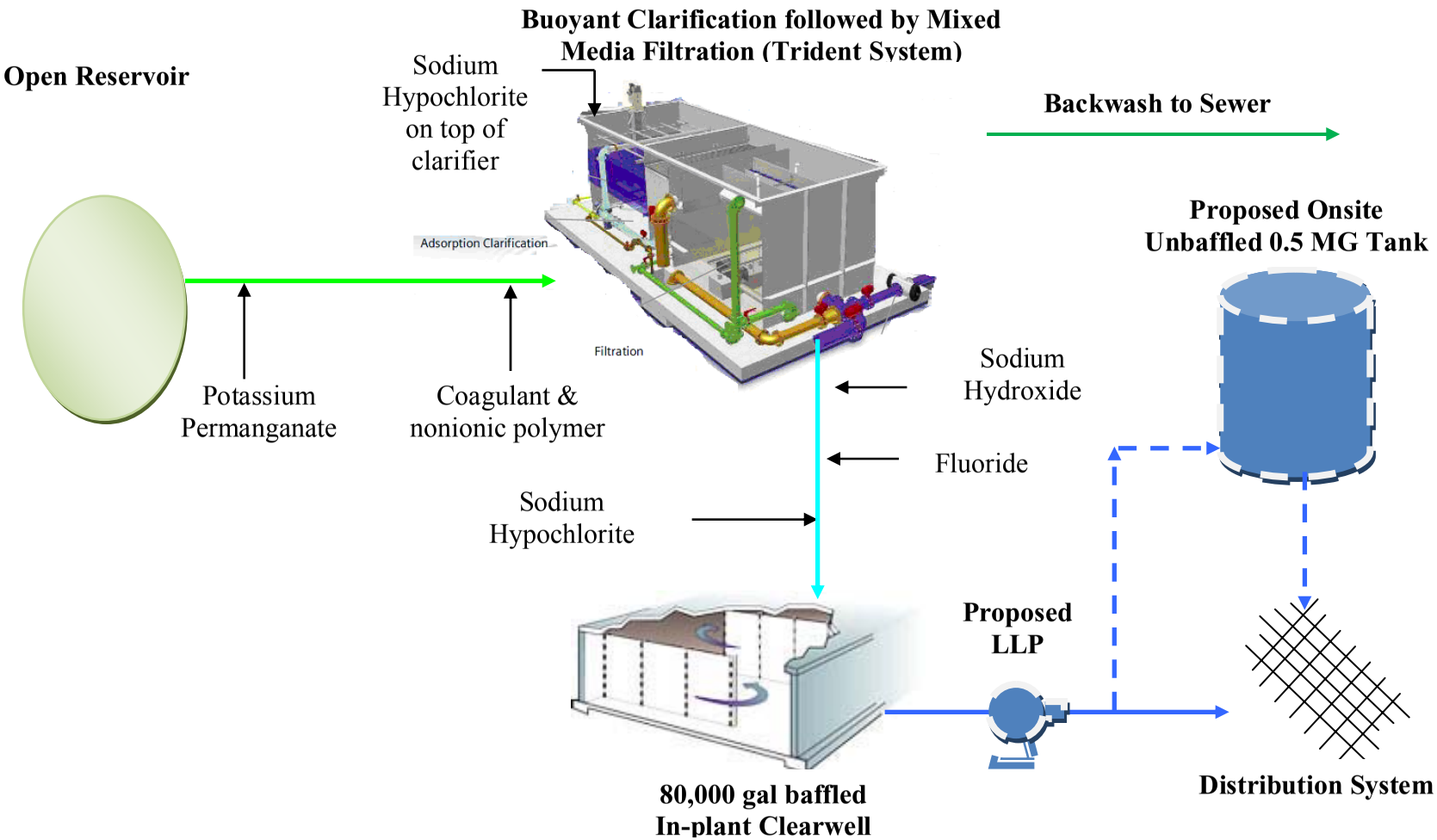


FIGURE 5-43B
ST. JOHNSBURY PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

Note: Proposed improved by St. Johnsbury engineer are shown as dashed line

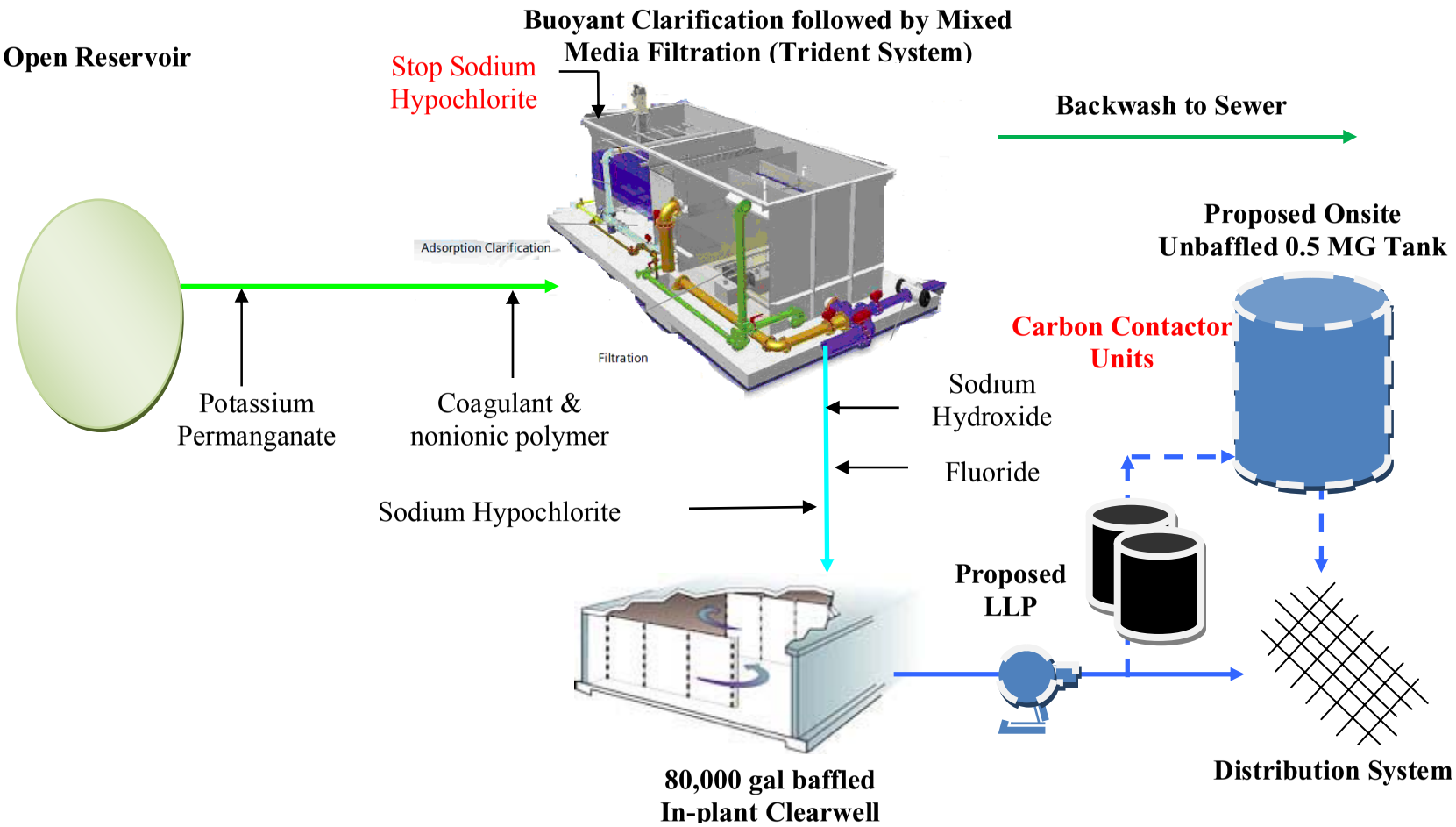


FIGURE 5-43C
ST. JOHNSBURY PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

Note: Proposed improved by St. Johnsbury engineer are shown as dashed line

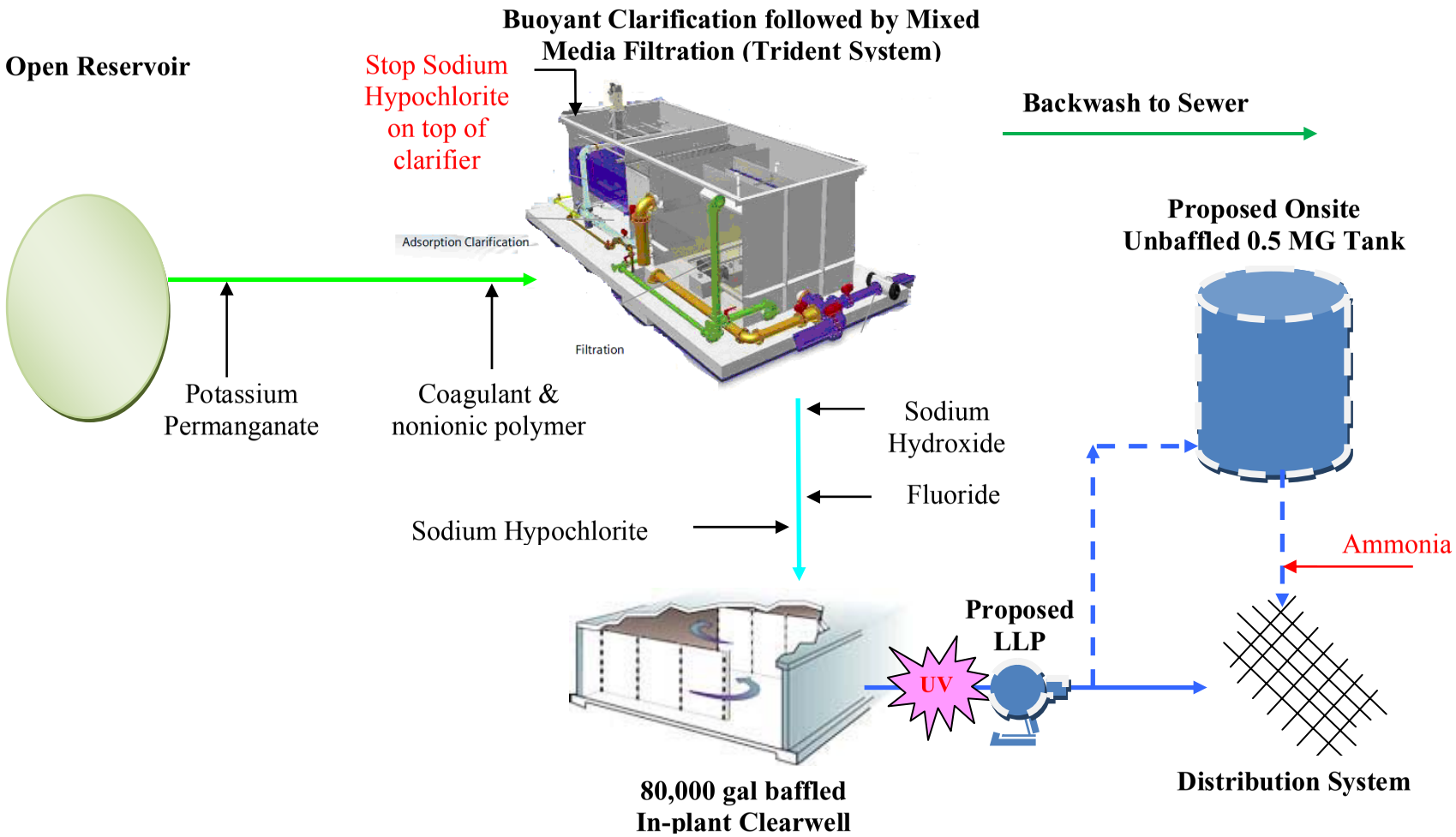
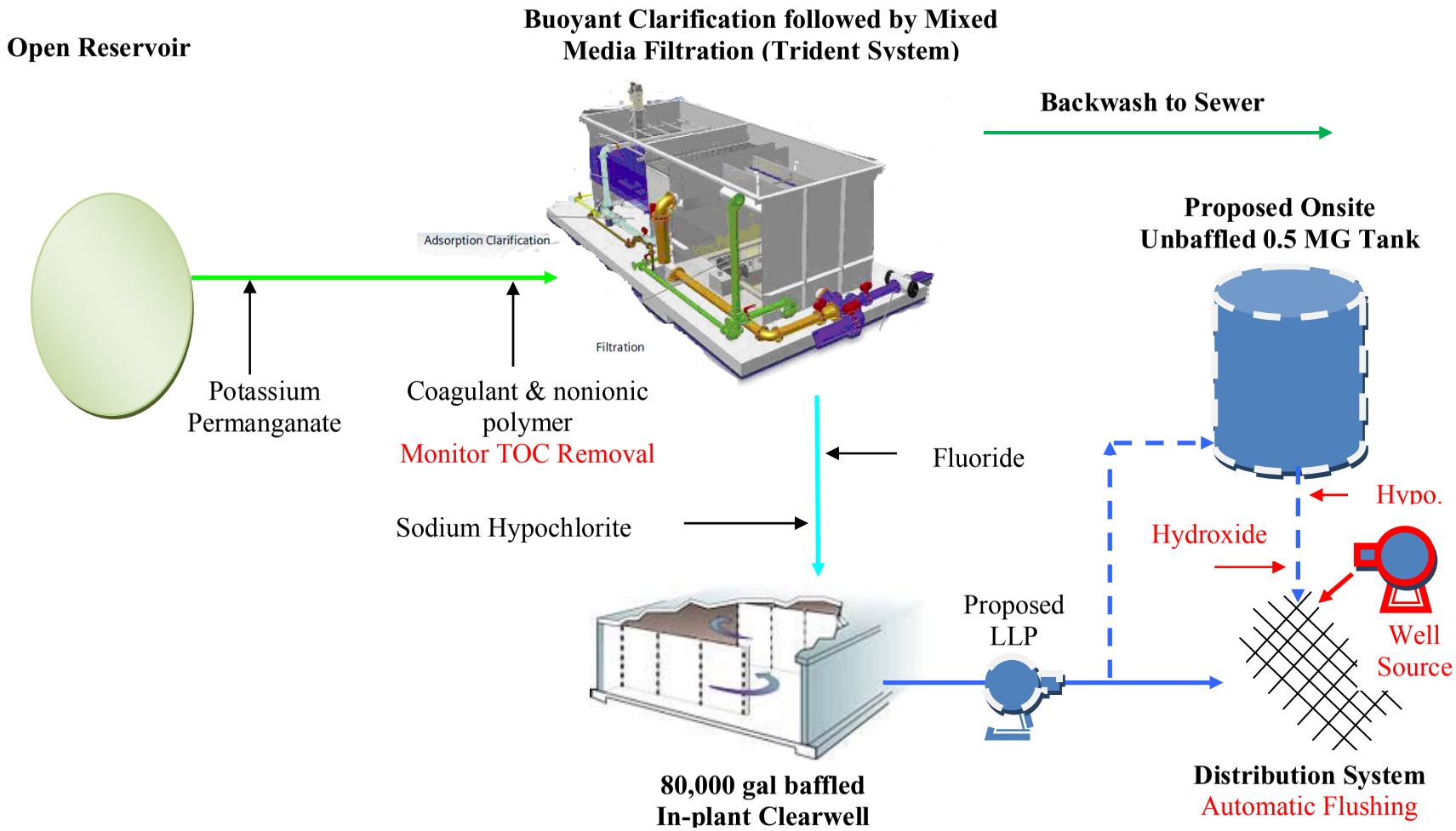


FIGURE 5-43D
ST. JOHNSBURY PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)

Note: Proposed improved by St. Johnsbury engineer are shown as dashed line



5.2.3.37 Swanton - Existing System

The treatment summary for the existing system is provided below and in Table 5-46. Figure 5-46A provides a schematic of the existing treatment system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source is a single intake on Lake Champlain. The intake is located at the bottom of the lake.

Treatment: The original WTF was constructed in 1979. Low lift pumps provide water to the WTF. Permanganate can be added for Zebra Mussel control. Alum is added as a coagulant and a substantial floc is realized. No rapid mix or static mixer is used to aid in the mixing of the alum. Limited sampling of TOC removed through the filter during early winter period (temperature ~5.0 C) indicates that the WTF is removing 45% of the raw TOC (3.1 down to 1.7 mg C/L on 12/7/09). Due to the large coagulant dose, filter run times are shortened.

Treatment consists of Modular Aquarius filtration with flocculation, tube settlers and gravity mixed media filtration. A nonionic polymer is added prior to the filters to aid in filtration. An in-plant clearwell provides disinfection credits. All chemicals are flow paced, but the WTF has limited SCADA.

Water is pumped from the plant to a remote storage tank at the end of the distribution system. In 2006, sodium hypochlorite addition prior to the filter was stopped. The distribution reservoir water level was fluctuated more in an attempt to mix the water in tank. These steps resulted in a reduction in DBPs detected in the distribution system.

The demands can reach close to system capacity, and with only two filters, can prevent service on the filters. In addition, when the largest high service pump is turned on, the time available in the clearwell is greatly reduced, necessitating a higher chlorine dose. Due to the age of the WTF and demands, the filtration equipment is due to be replaced.

Distribution: The distribution mainly consists of cast iron pipe, most of which is unlined. The distribution is highly looped and is flushed in the spring and fall of each year. There are no consecutive systems.

Operator: There is a full time plant operator and one part time operator to cover for weekends and vacations. The WTF runs 24 hours a day, seven days a week.

VT DEC Observations: The coagulant dose is sufficient to generate a settleable floc.

AECOM Observations: Swanton is one of the few utilities practicing enhanced coagulation (i.e. targeting TOC removal beyond turbidity removal). The main issues associated with the DBP formation is the flexibility of WTF operation as well as chlorine demand within the distribution system.

TABLE 5-46
SWANTON SYSTEM SUMMARY

Population			
Value	3,500		
System Demand, MGD			
	Typical		
Minimum	0.40		
Average	0.57		
Maximum	0.70		
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	~34		
-Estimated average diameter, inches	6 inch (2 inch to 12 inch)		
Approximate System Detention Time	Not known		
Number of Storage Tanks	1		
-Provide Size of each, MG	1.5		
-Tank Type	Square cement tank. Water depth typically 16. 75 ft.	Split for service and partially underground	
Chemical Addition			
Process	**Chemical**	**Average Concentration, mg/L**	**Wet/Dry weight**
Prechlorination	No		
Preoxidation	Potassium Permanganate	Pink color to filter	Dry
Coagulant	48.5 % Alum	45	Dry
Filter Aid	Nonionic Polymer	≤ 0.10	Wet
Disinfection	12.5% Sodium Hypochlorite	1.5 to 1.7 Residual	Dry
Corrosion Control	25% Sodium Hydroxide	Target 7.5 to 7.8	Dry

5.2.3.38 Swanton - Proposed Alternatives

AECOM recommends that Swanton follow the steps provided in Table 5-47 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-44 and 5-45 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Swanton will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Swanton. The summer DBP levels for calibration were 75 ppb for TTHM and 45 ppob for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 52 ppb, HAA5 = 35 ppb
2. UV and post clearwell chloramination: TTHM = 44 ppb, HAA5 = 21 ppb

Chlorine as Distribution Residual (Figure 5-46B)

Swanton has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. The facility is replacing the existing filter technology. AECOM would recommend dissolved air flotation followed by GAC (pressure filter).

Chloramine as the Distribution Residual (Figure 5-46C)

Swanton has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below the MCLs.

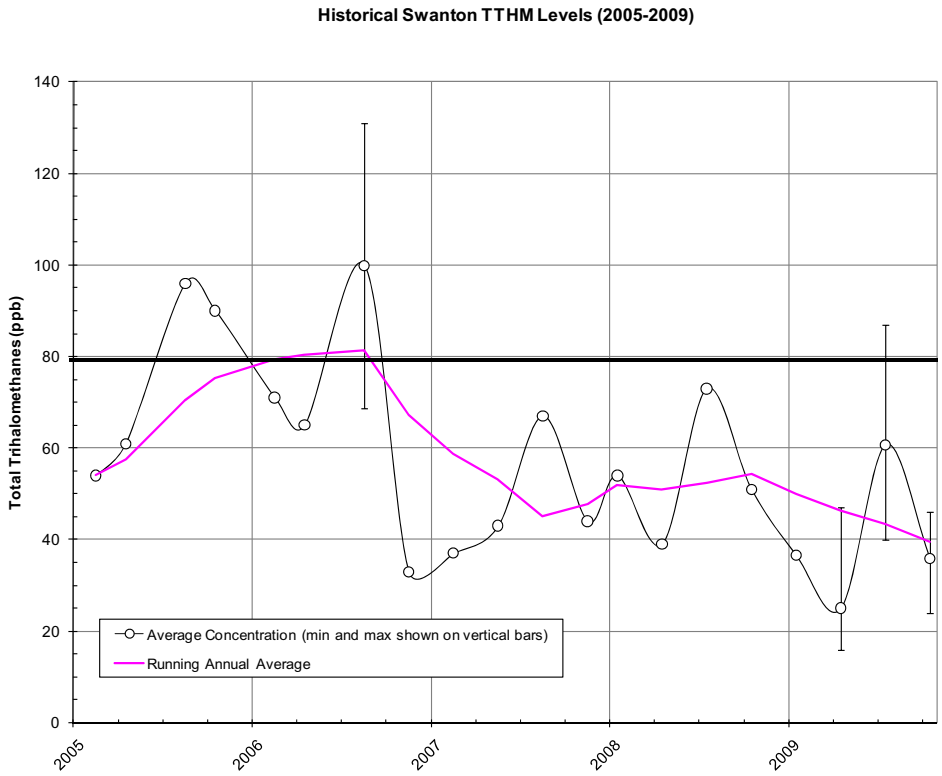
TABLE 5-47
PROPOSED ALTERNATIVES FOR SWANTON

Treatment- Existing System	Distribution
For Chlorine	
1.) Dissolved Air Flotation followed by deep bed GAC	None proposed
For Chloramine	
None proposed	Chloramine

5.2.3.39 Swanton - Other DBP Reduction Alternatives (Figure 5-46D)

A summary of other DBP reduction alternatives for Swanton is provided in Table 5-48. The following provides a brief discussion of each alternative.

1. In the interim period until the new filtration equipment is provided, the sodium hydroxide can be dosed after the clearwell to take advantage of a lower pH during disinfection, which allows chlorine to be more effective and require less contact time. In addition, a second chlorine injection point can be added after the clearwell. This allows the original chlorine dosed to be split so to reduce the overall contact time in which chlorine is in contact with TOC.
2. Install automatic flushing devices at select dead ends to pull fresh, chlorinated water into dead end and to reduce overall age of the water. The flushing would occur in the warmer seasons. The potential downside is that flushed water would not be consumed and could be considered wasted.
3. Line cast iron pipes to reduce the chlorine demand in the distribution.
4. Install booster chlorination to raise the chlorine at sections of the distribution with low chlorine.



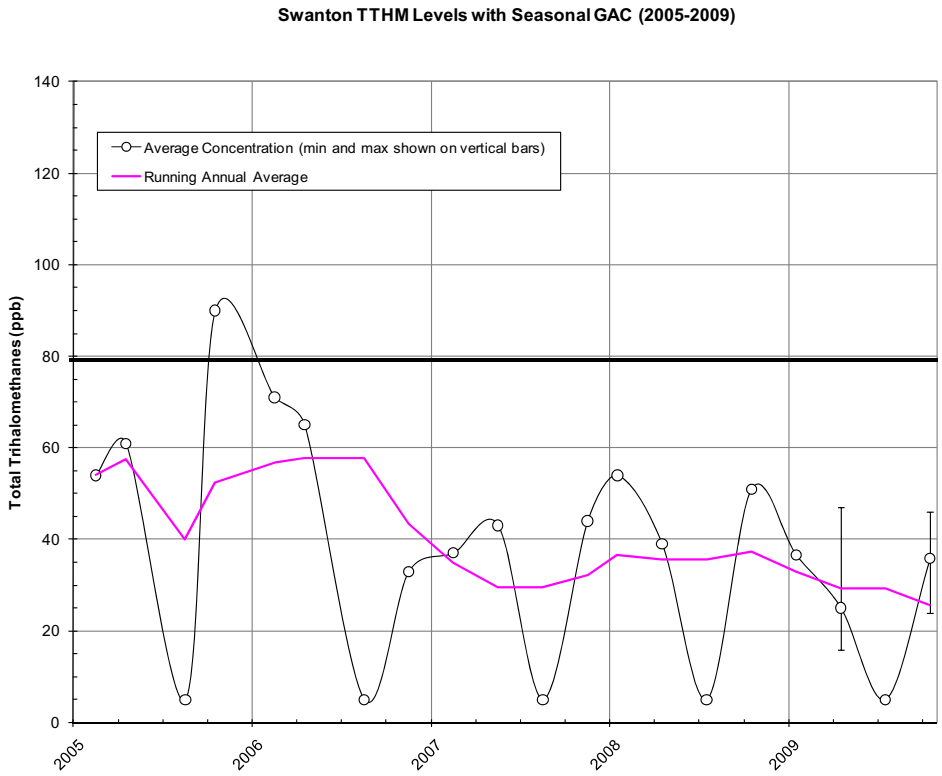
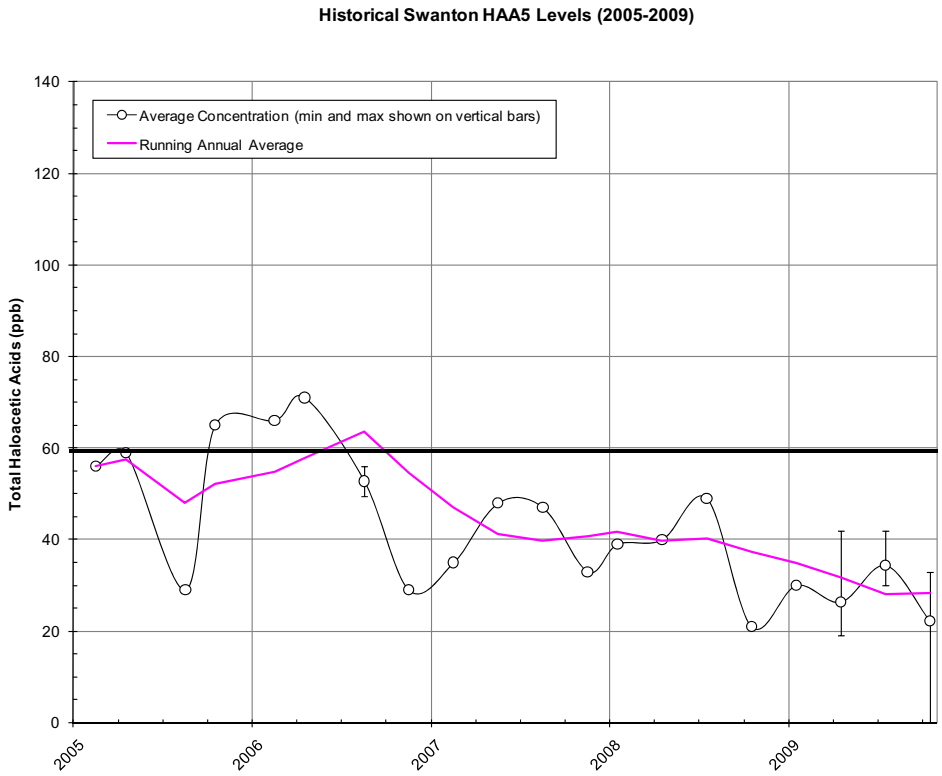


FIGURE 5-44
SWANTON TTHM LEVELS (2006-2009)



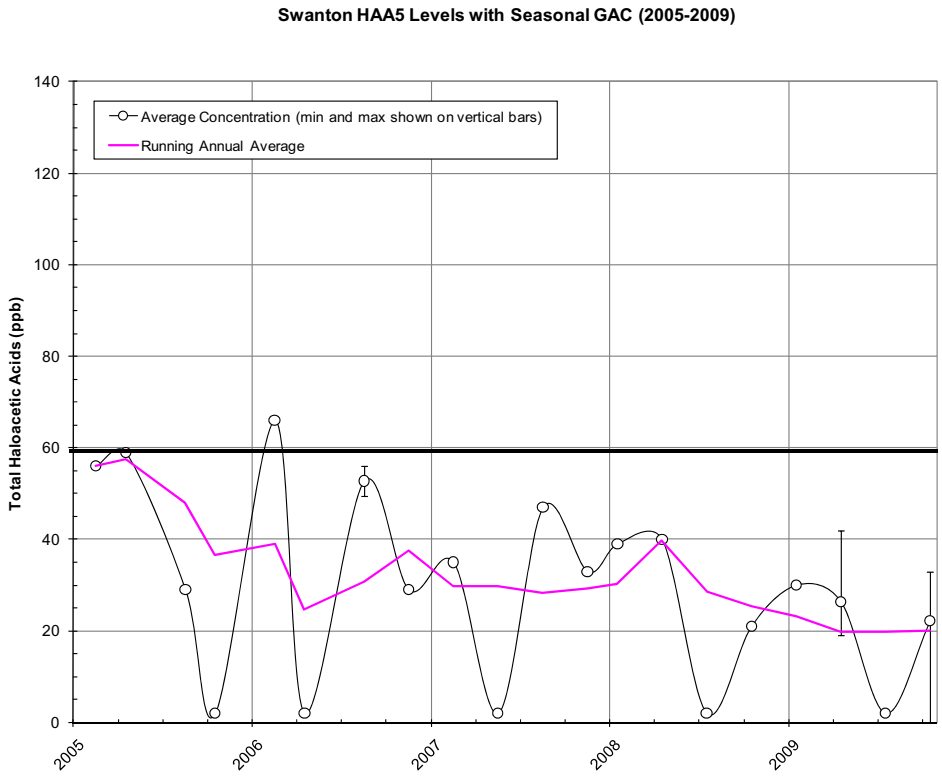


FIGURE 5-45
SWANTON HAA5 LEVELS (2006-2009)

TABLE 5-48
SWANTON - OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Effect*	Distribution	Effect*
1.) Move pH adjustment application point to after 0.5 MG onsite tank	Low	1.) Automatic flushing	Low
2.) Move distribution chlorination point to after 0.5 MG onsite tank	Low	2.) Line cast iron pipe	Low
		3.) Install booster chlorination	Low

*Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-46A
SWANTON EXISTING PROCESS FLOW DIAGRAM

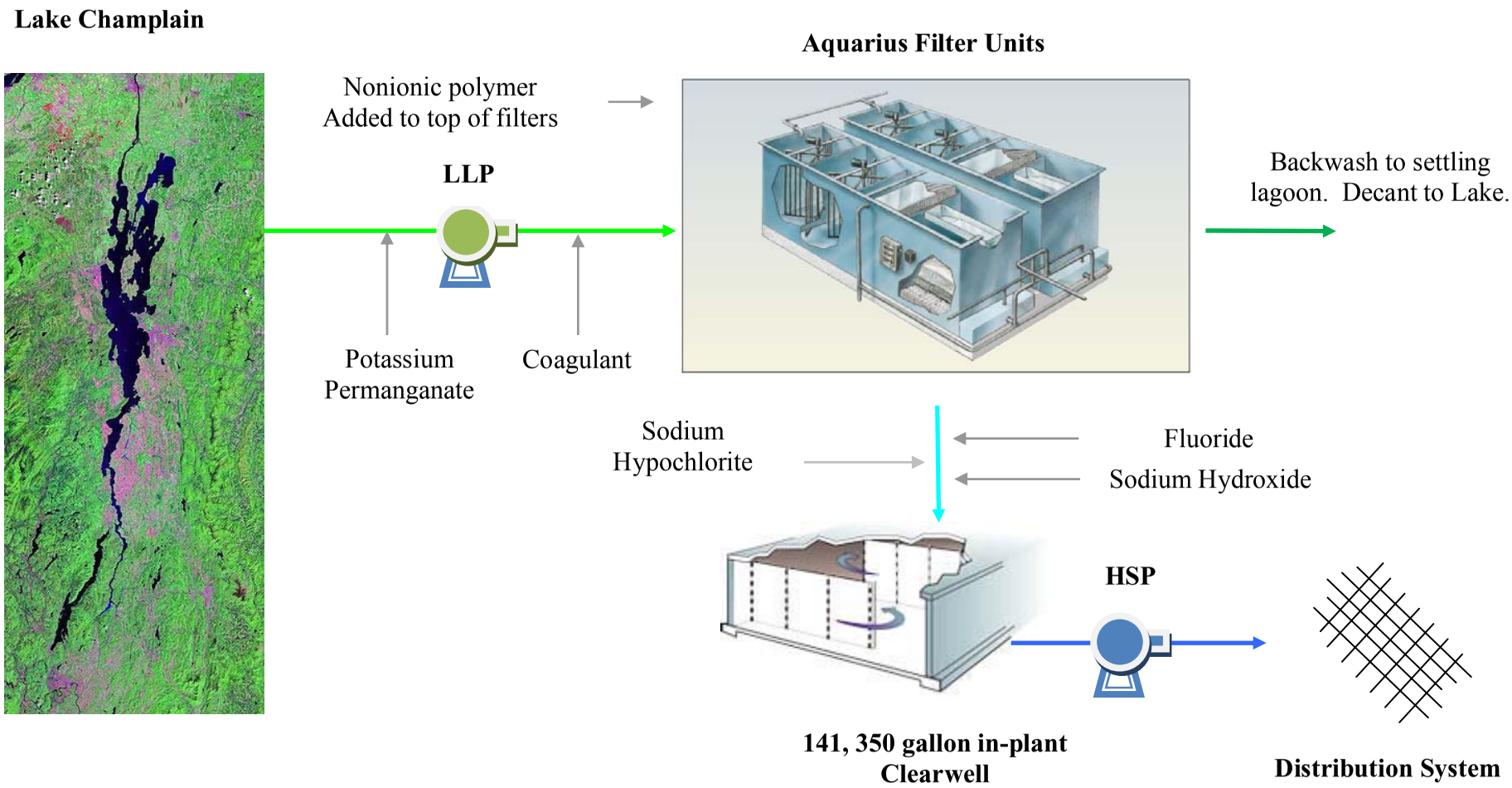


FIGURE 5-46B
SWANTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)

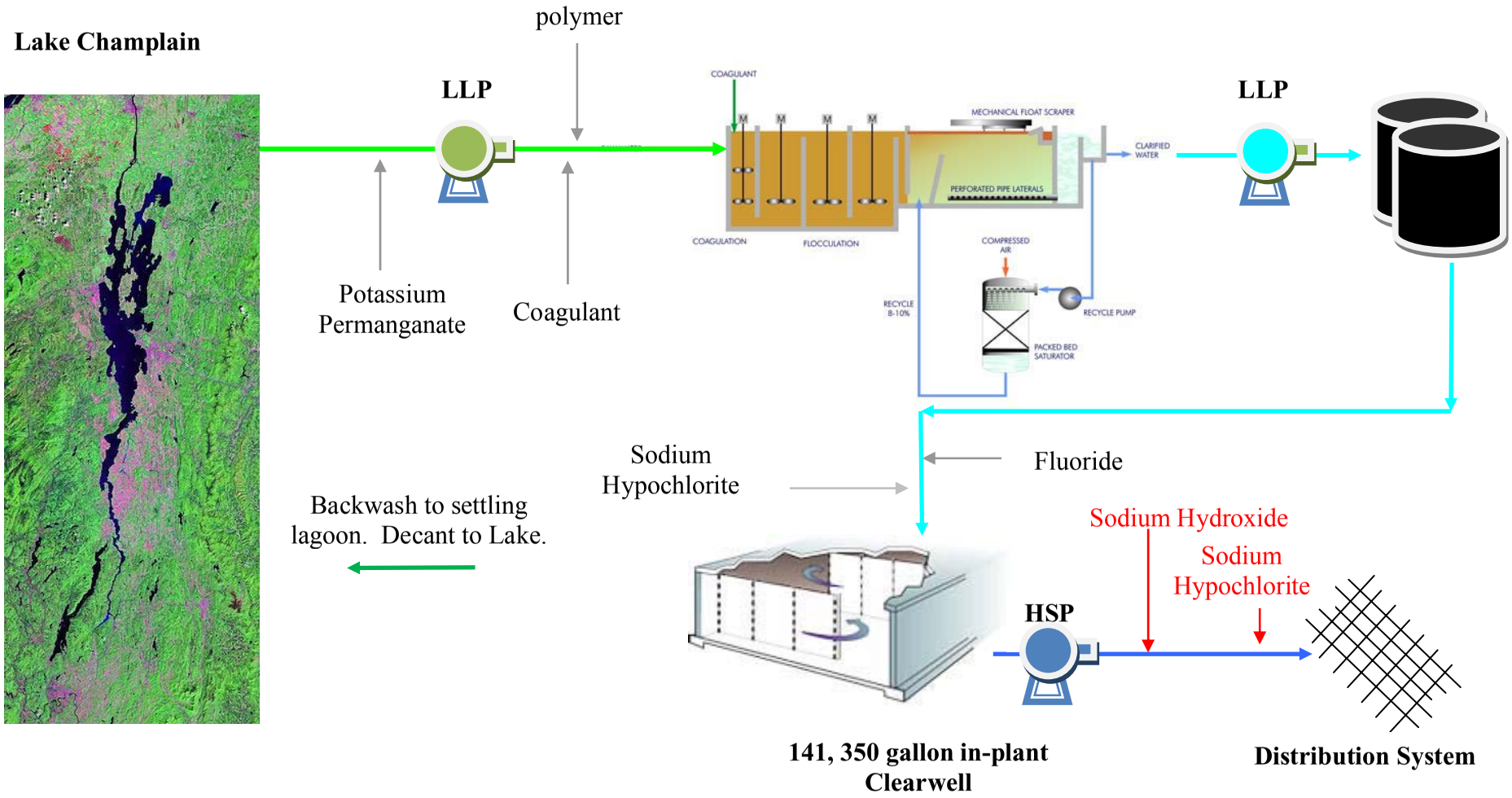


FIGURE 5-46C
SWANTON PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

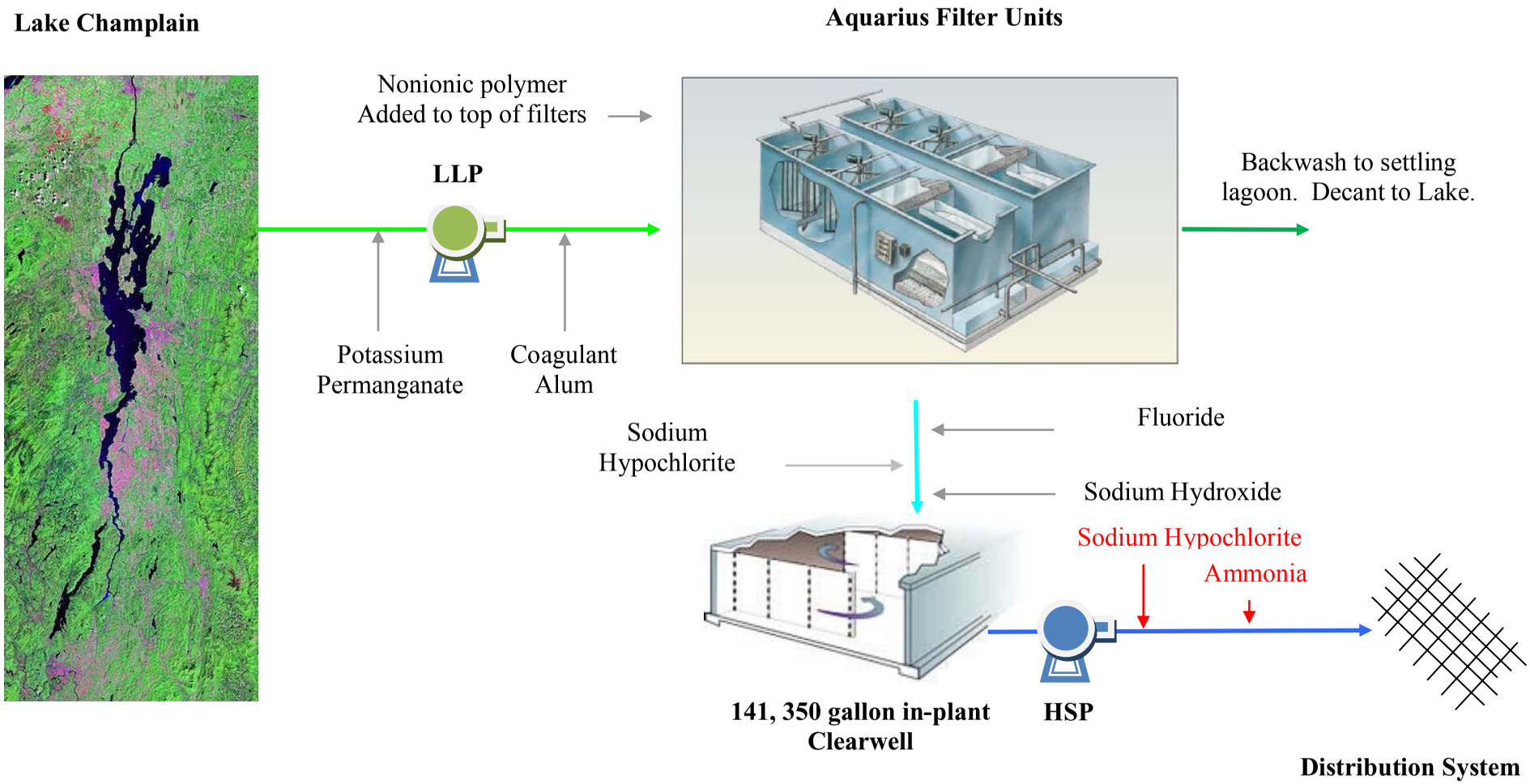
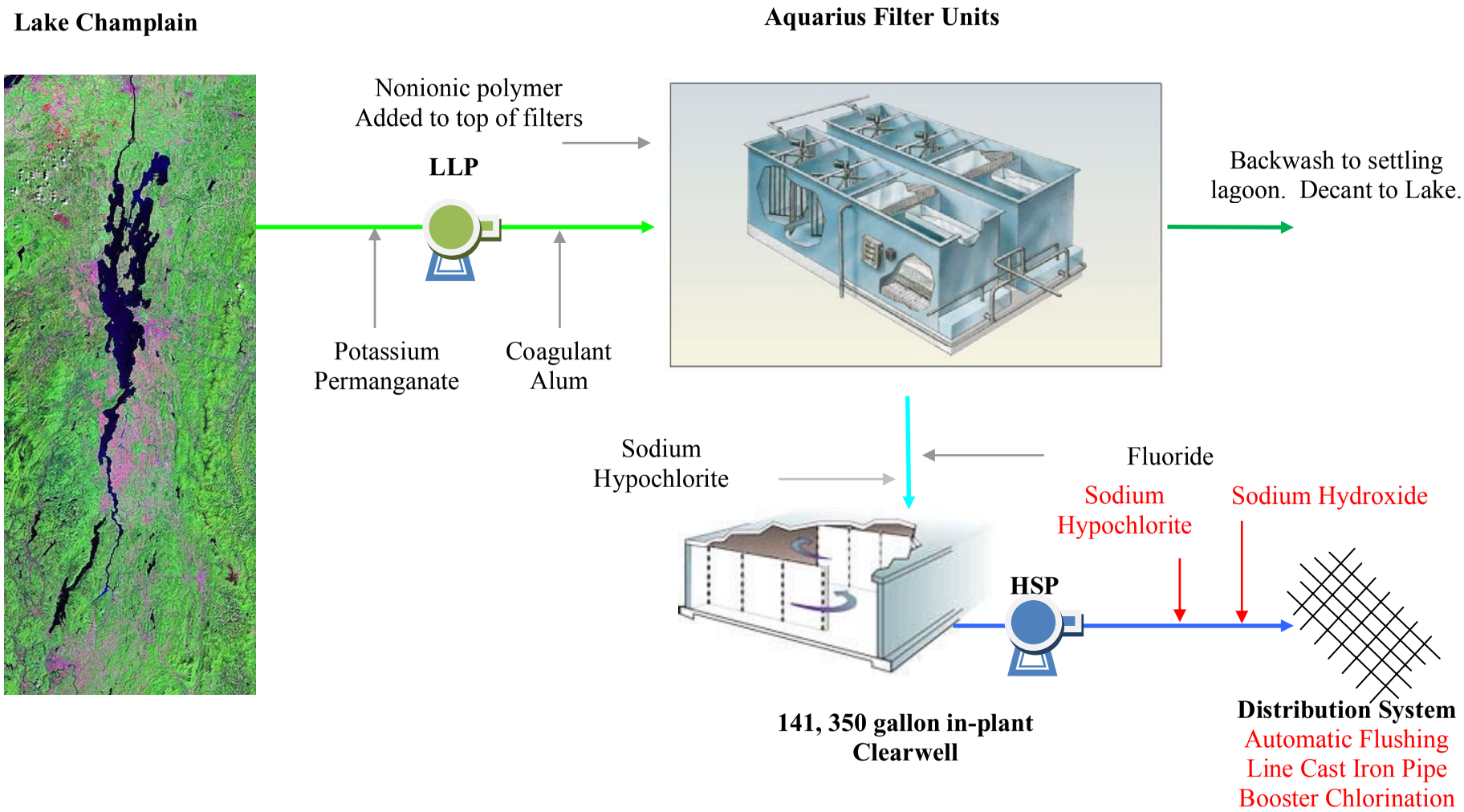


FIGURE 5-46D
SWANTON PROPOSED PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS
(UPDATES IN RED)



5.2.3.40 Tri-Town Water District - Existing System

The treatment summary for the existing system is provided below and in Table 5-49. Figure 5-49A provides a schematic of the existing process. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source is a shallow section of Lake Champlain. There is one intake at approximately 450 feet into lake with a riser off the bottom.

Treatment: Treatment consists of coagulation, custom Trident system with buoyant media followed by gravity mixed media filters. There is one in-plant clearwell followed by an on-site disinfection tank. Water is pumped from the plant to remote storage. The operator takes steps to reduce chlorine dosed during winter periods.

Distribution: The Tri-Town District services three Towns. As such, there are several hundred miles of pipe. The distribution is set up into two main zones: A low pressure close to Lake Champlain zone and a high pressure zone. Water from the WTF fills the low pressure zone tank. A booster pump takes water from the low pressure tank and sends it to the high pressure tank, creating two tanks in series. Tanks generally see a three to four foot fluctuation in the tank levels. The system is generally well looped. There are no consecutive systems.

Operator: The plant operator works full time, but is not at the plant 24 hours per day. There are three full time operators. The WTF is online 24 hours a day, seven days a week. However, the WTF runs based on tank level in the low pressure zone, so generally, the WTF will run continuously for a period of 20 hours during the day and then shut down.

VT DEC Observations: Turbidity can be elevated seasonally or during high winds.

AECOM Observations: The system is pre-chlorinating which contributes to the overall DBPs which formation. In addition, the process does not appear to be practices enhanced coagulation for targeted TOC removal. Lastly, the distribution system has two tanks in series which adds to the residences time in the distribution system.

TABLE 5-49
TRI-TOWN WATER DISTRICT SYSTEM SUMMARY

Population			
Value	3,500		
System Demand, MGD			
	Typical Values		
Minimum	0.613		
Average	0.712		
Maximum	0.820		
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	400 to 500		
-Estimated average diameter, inches	2 inches to 12 inches	Mostly Transite pipe (AC)	
Approximate System Detention Time	3 days plus		
Number of Storage Tanks	2		
-Provide Size of each, MG	0.75 and 0.625	Steel tank is 54 ft dia. X 40 ft High	
-Tank Type	Steel and concrete	Concrete is 52 ft x 103 ft x 16 ft Deep.	Two cells
Chemical Addition			
Process	Chemical	Average Concentration, ppm	Wet/Dry weight
Prechlorination	Yes		
Preoxidation	Potassium permanganate	Adjust to slight pink	
Pre-Chlorination	12.5% Sodium Hypochlorite	0.5	Dry
Coagulant	48.5% Alum	10	Dry
Polymer 1	Kemira C-572	3.5	Wet
Polymer 2	Kemira N-300	0.15 to 0.20 (not used in summer)	Wet
Coagulated pH adjust	25% Sodium Hydroxide	Typically not used	
Finished pH adjust	25% Sodium Hydroxide	Target pH 7.5-8.0	
Post Chlorine	12.5% Sodium Hypochlorite	2.5	Dry
Corrosion Control	25% Sodium Hydroxide	Adjust to pH target of 7.5	
Corrosion Control	Carus 300 Zn Ortho. (40-60% PO_4)	1.0	Dry

5.2.3.41 Tri Town Water District - Proposed Alternatives

AECOM recommends that Tri Town Water District follow the steps provided in Table 5-50 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-47 and 5-48 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Tri Town will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season) to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Tri Town. The summer DBP levels for calibration were 102 ppb for TTHM and 67 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 63 ppb, HAA5 = 48 ppb
2. UV and post clearwell chloramination: TTHM = 42 ppb, HAA5 = 24 ppb

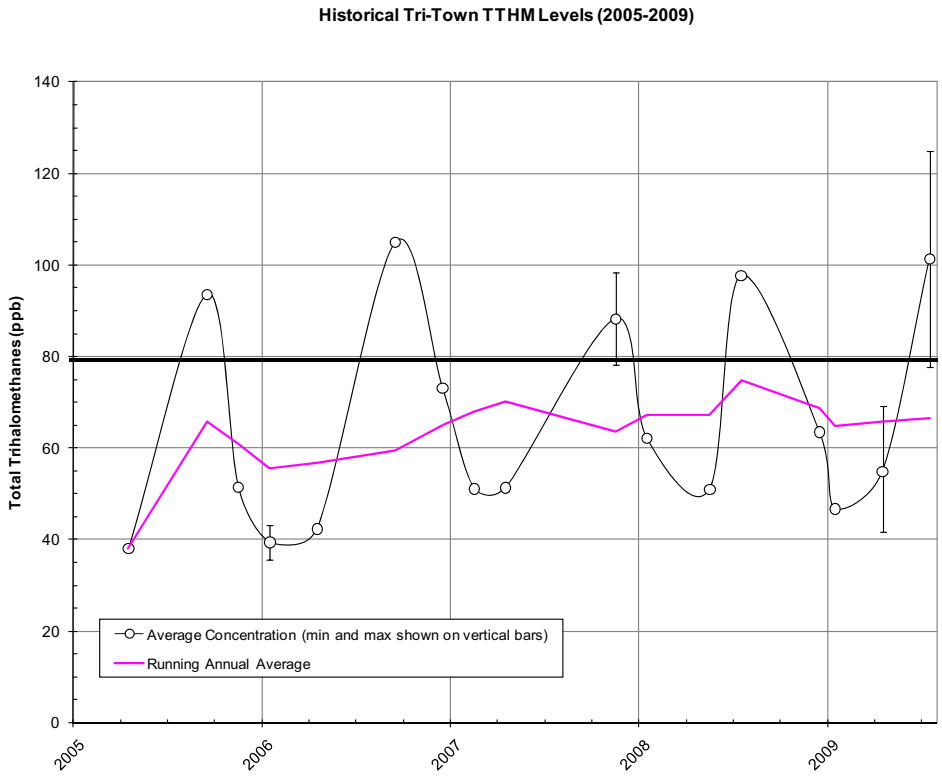
Chlorine as Distribution Residual (Figure 5-49B)

Tri Town Water District has the following options to address DBP reduction while using chlorine as a residual in the distribution system:

1. Stop prechlorination. Prechlorination allows for the interaction of chlorine with a large source of TOC which is caught in the filters and can elevate DBPs.
2. Optimize coagulation for precursor (e.g. TOC) removal. Lower filtered TOC levels will translate into lower DBP levels.
3. Once steps 1 and 2 are optimized, install GAC as a filter polishing step to remove more TOC prior to entering the distribution system.

Chloramine as the Distribution Residual (Figure 5-49C)

Tri Town Water District has the following options to address DBP reduction while using chloramine as a residual in the distribution system:



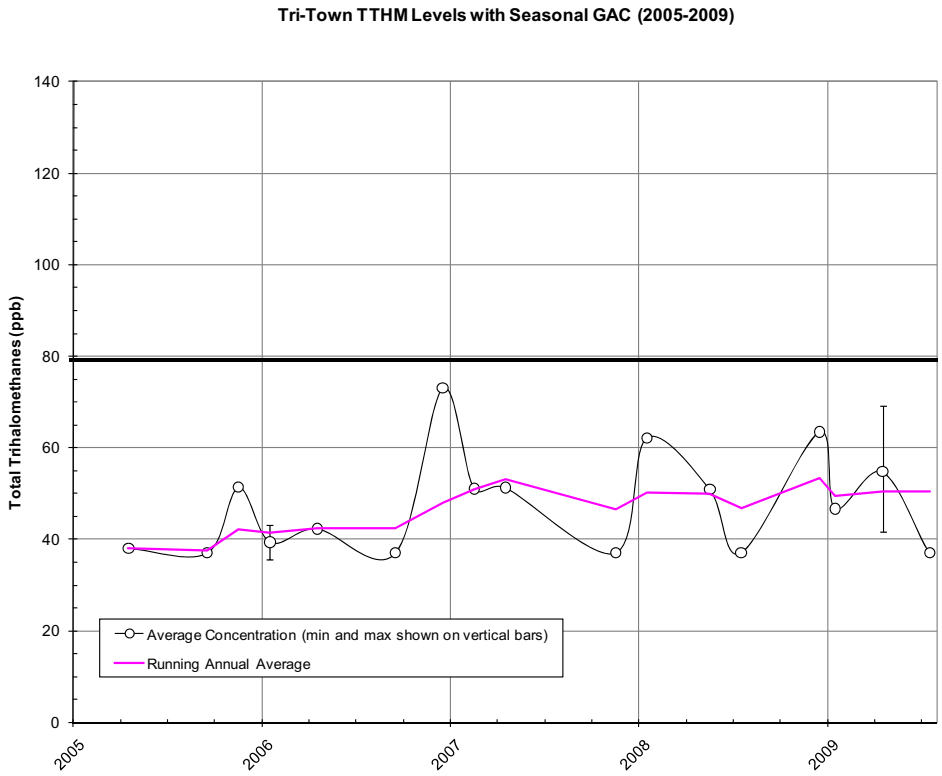
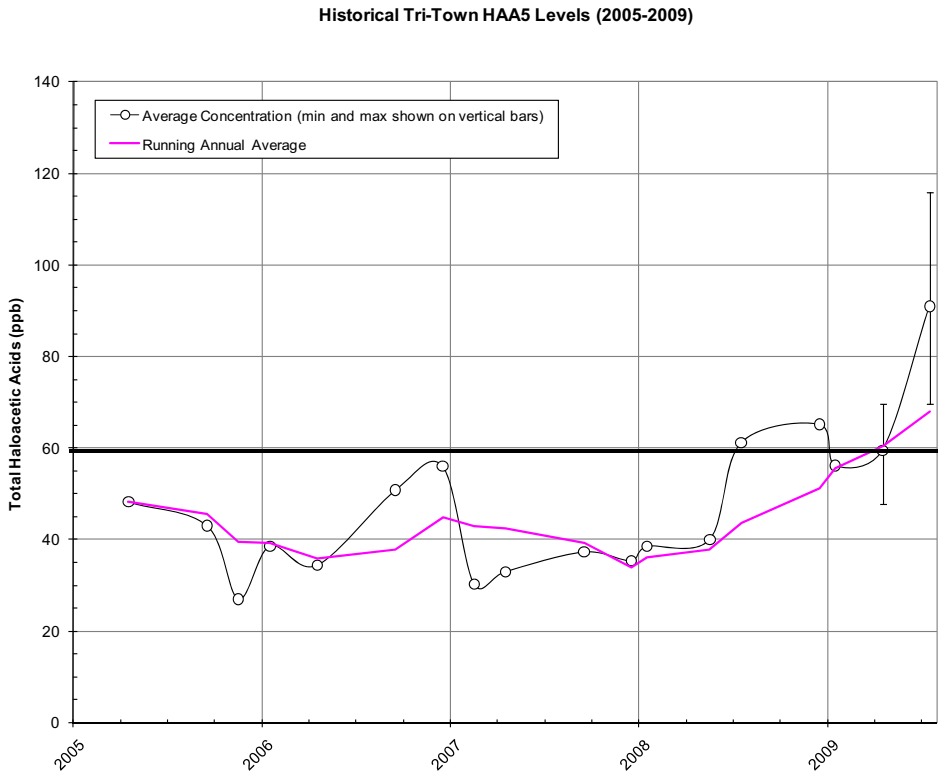


FIGURE 5-47
TRI-TOWN TTHM LEVELS (2006-2009)



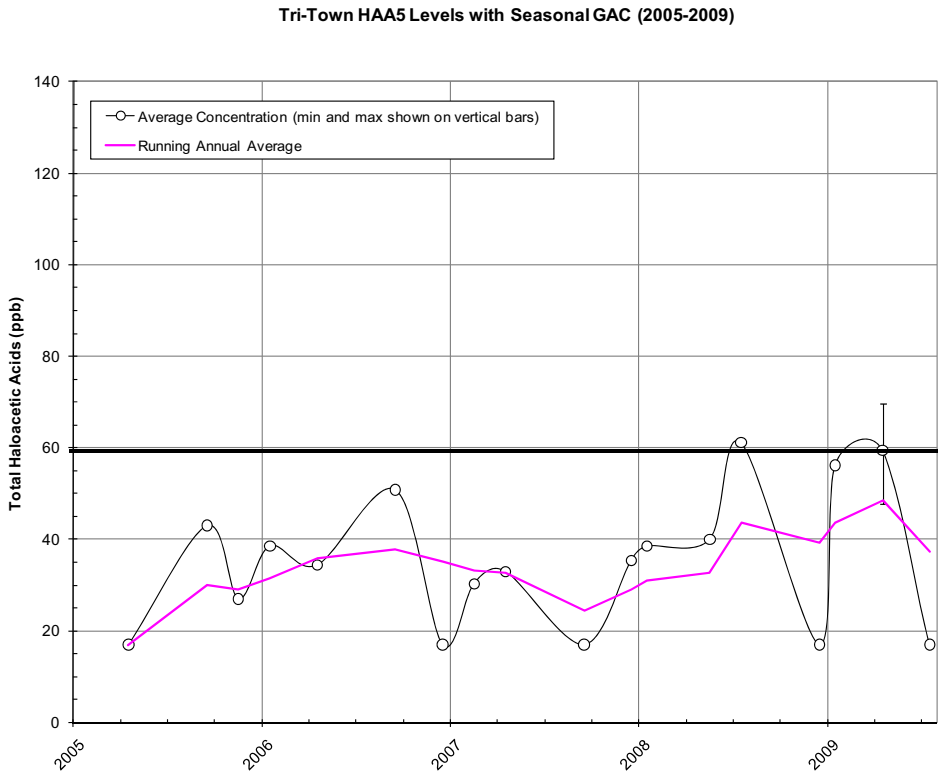


FIGURE 5-48
TRI-TOWN HAA5 LEVELS (2006-2009)

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to below MCLs.

TABLE 5-50
PROPOSED ALTERNATIVES FOR TRI TOWN WATER DISTRICT

Treatment- Existing System	Distribution
For Chlorine	
1.) Stop pre-chlorination	None proposed
2.) Optimize coagulation to remove TOC	
3.) Seasonal (summer) post GAC absorber with 20 minute EBCT	
For Chloramine	
None proposed	Chloramine

5.2.3.42 Tri Town Water District - Other DBP Reduction Alternatives (Figure 5-49D)

A summary of other DBP reduction alternatives for Tri Town Water District is provided in Table 5-51. The following provides a brief discussion of each alternative.

1. Incorporate a unidirectional flushing program to maintain pipe surfaces with lower chlorine demand and biological activity.
2. The WTF is at the end of the distribution system. Water is pumped from the WTF to the low zone distribution tank. From there water is booster pumped from the low zone to the high zone. Consider booster chlorination at the high zone tank to reduce the dose at the WTF as the distribution is a large system.

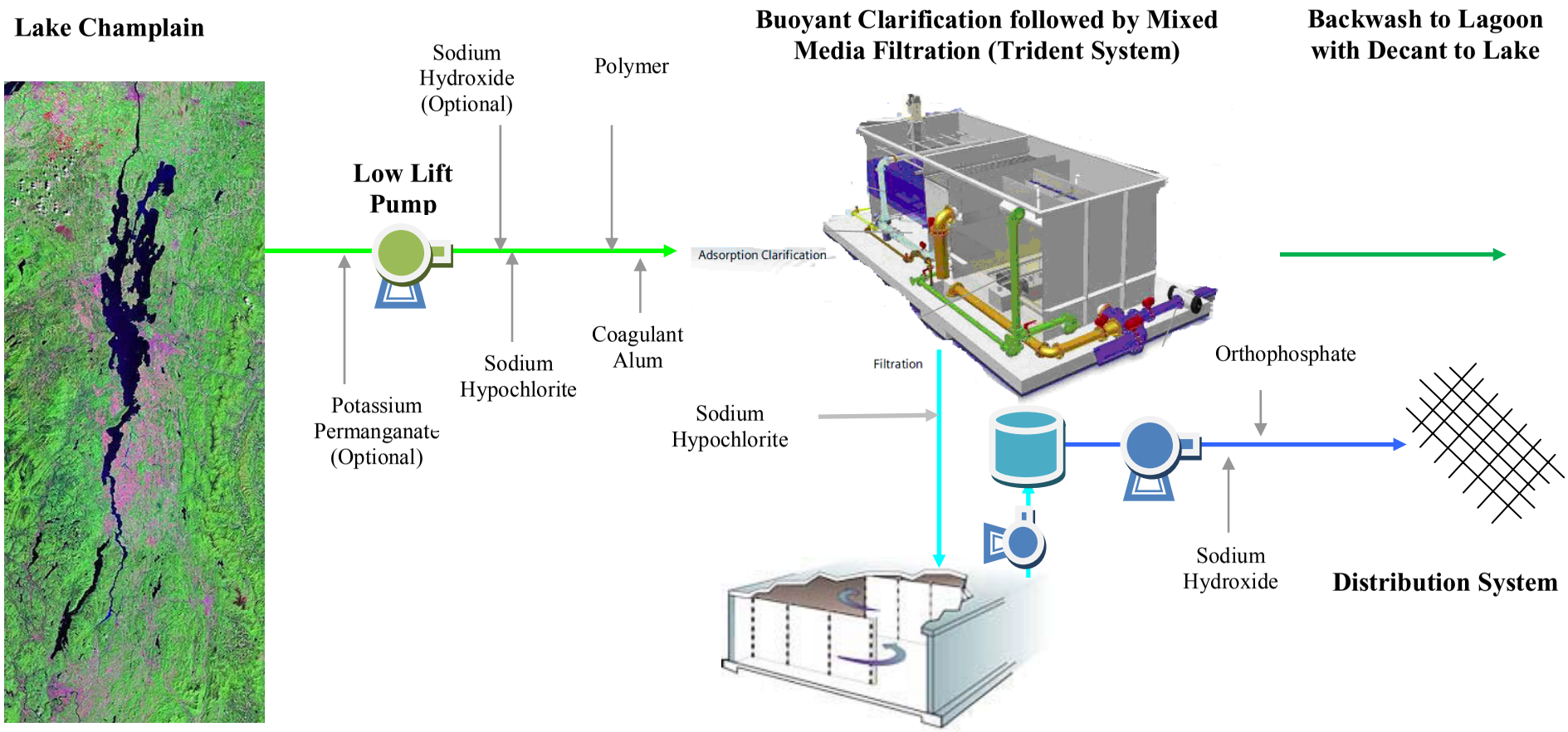
TABLE 5-51
TRI TOWN WATER DISTRICT - OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Distribution	Effect*
None proposed	1.) Consistent program for unidirectional flushing program	Low
	2.) Booster chlorination	Low to Medium

*Effect is the anticipated relative effect the action has on reducing overall DBPs.

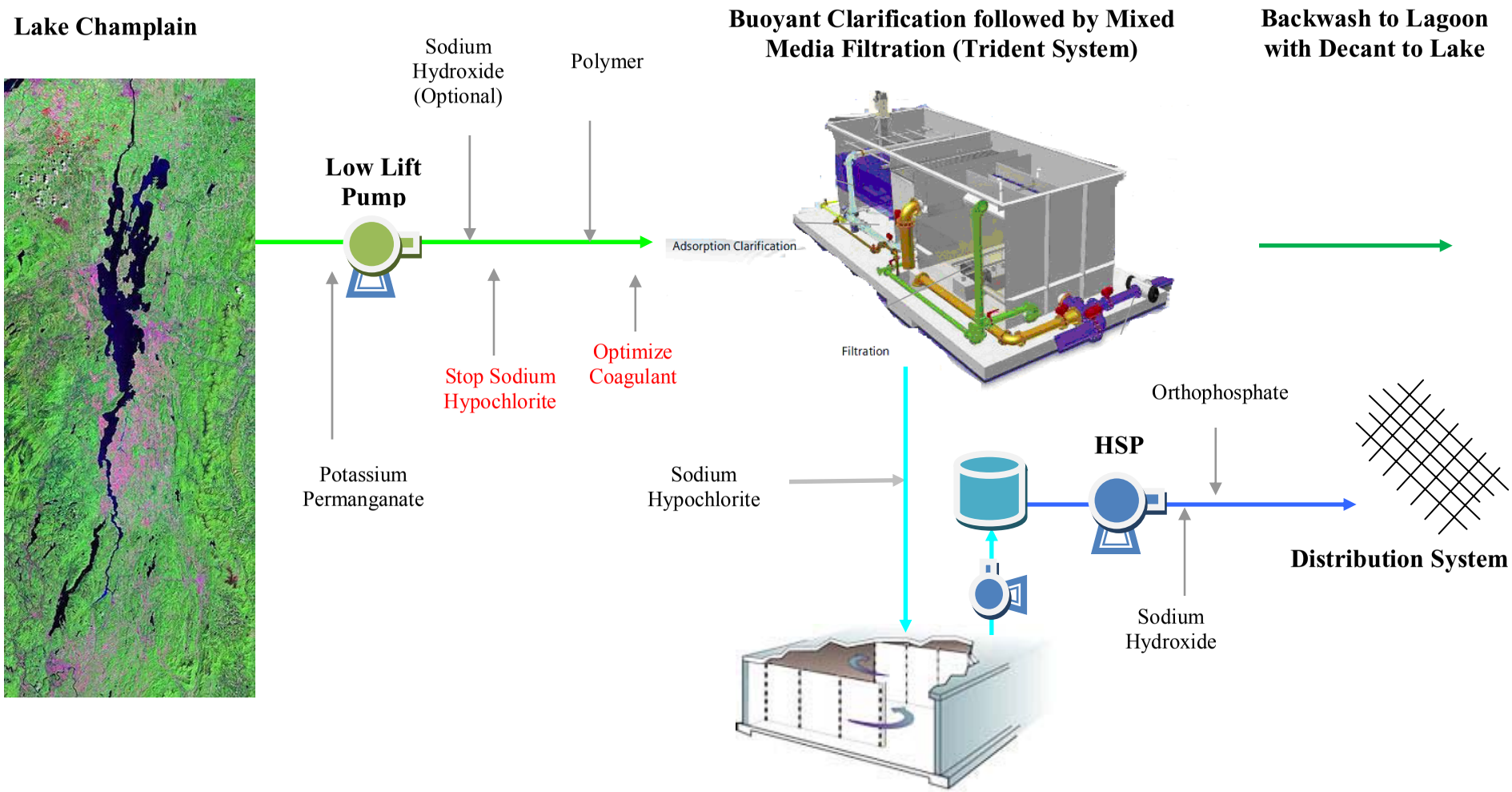
1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-49A
TRI TOWN WATER DISTRICT EXISTING PROCESS FLOW DIAGRAM



80,000 gallon in-plant slightly baffled clearwell (U-shape) with low lift pump to 100,000 gallon Steel baffled contact tank via Low Lift Pump

FIGURE 5-49B
TRI TOWN WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE (UPDATES IN RED)



In-plant clearwell with low lift pump to steel contact tank via Low Lift Pump.
Install GAC after low lift pump.

FIGURE 5-49C
TRI TOWN WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE
(UPDATES IN RED)

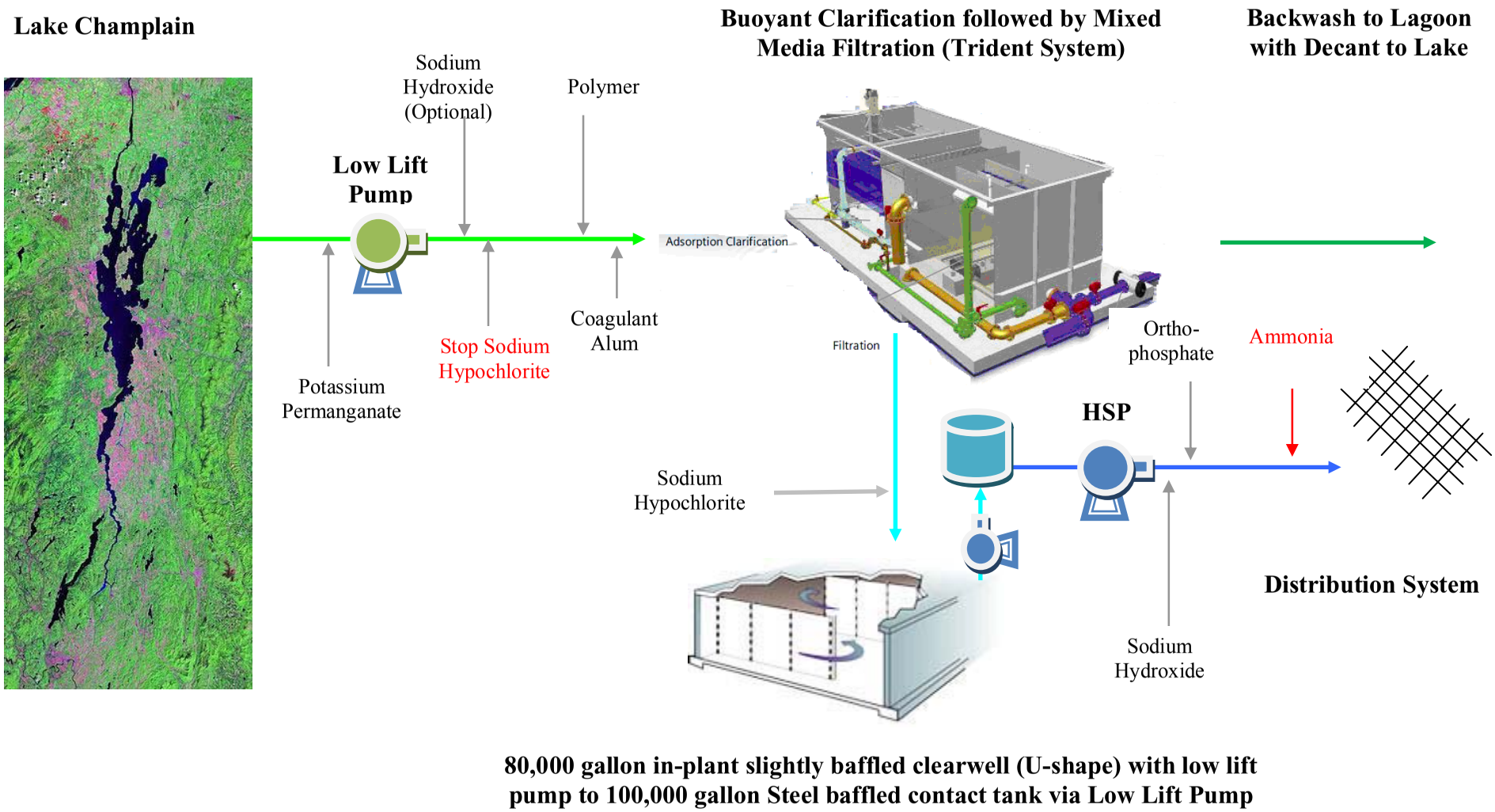
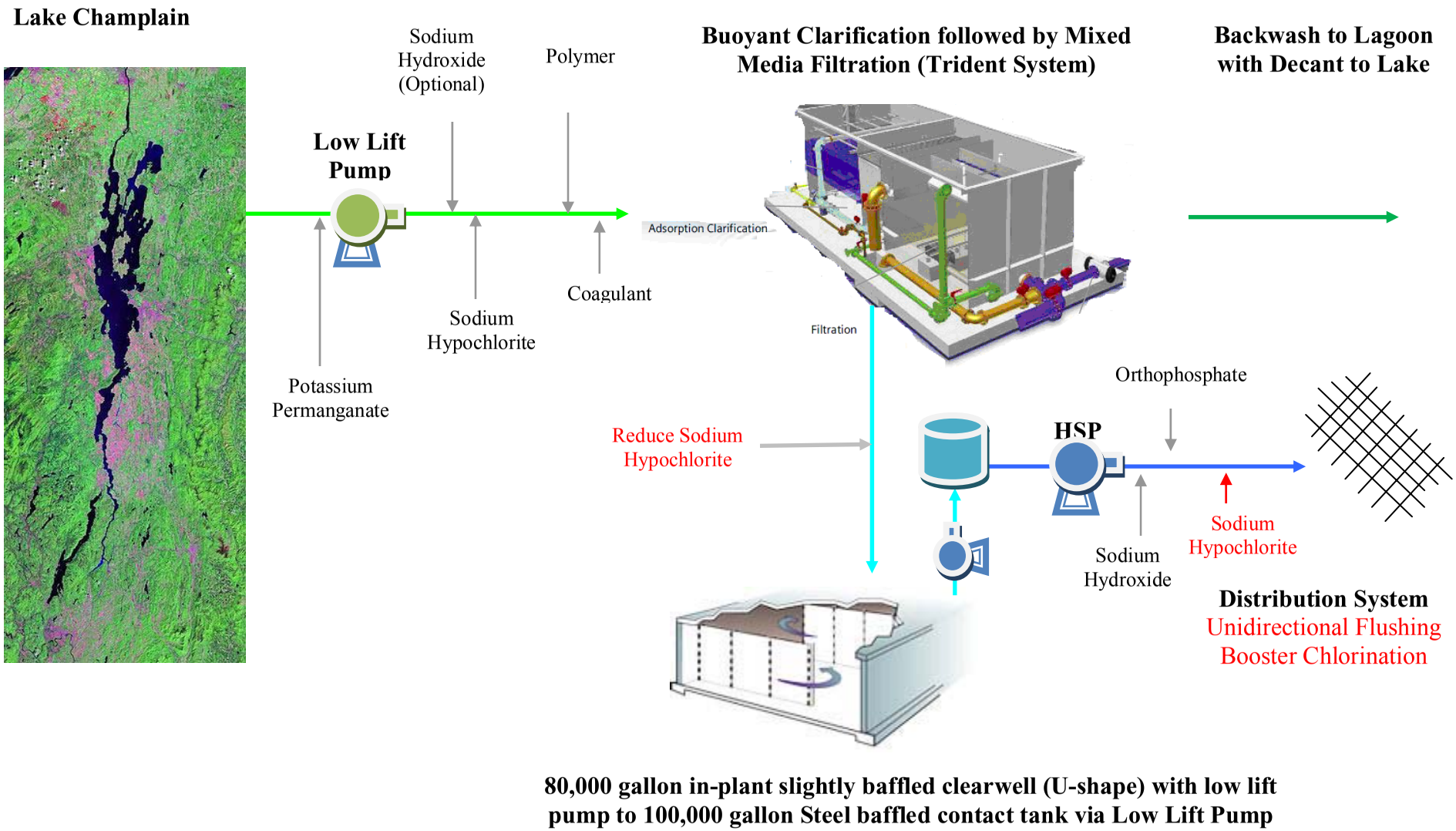


FIGURE 5-49D
TRI TOWN WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS
(UPDATES IN RED)



5.2.3.43 Vergennes-Panton Water District - Existing System

The treatment summary for the existing system is provided below and in Table 5-52. Figure 5-52A provides a schematic of the existing treatment system. Further information on water quality is provided in Appendix 3 - Operational Data and Survey Information and Appendix 4 - Distribution System DBP Data Over Time.

Source: The source is a shallow bay on Lake Champlain. There is one intake. The existing intake is 1,800 ft into the lake, roughly 40 feet from the surface and roughly 20 feet from bottom.

Treatment: Current treatment is sand pressure filtration in a low pressure filter vessel. The pressure filter is limited to a maximum of seven feet of headloss. The District's consulting engineer, Otter Creek Engineering is developing plans for Trident type replacement filtration system which will fit into the existing WTF and allow the existing facility to keep running. In general, the building is in good condition, but the filter equipment has reached the end of its useful life.

Distribution: The original system in Town was installed in the early twentieth century and is approaching 100 years old. The fringes of the distribution system were installed in the 1970's. The distribution has of two storage tanks. The system is flushed two times a year and the operator attempts to reduce chlorine in the winter but is restrained due to contact time. The District has noted that during flushing, they have experienced breaks, even when care is taken to close the hydrant. The soils in the Champlain Lake region are noted as being acidic and the older cast iron pipe, fittings and appurtenances have not held up well under these conditions. The system generally is not well looped. The District supplies two consecutive systems. The second system is Tri Town Water District which is emergency basis only. The active consecutive system has a population size of 180.

Operator: The water treatment facility runs 24 hours a day, seven days a week during the summer. During the winter, the facility runs 18 hours a day, seven days a week. There are three full time operators.

VT DEC Observations: The filters are 35 years old. The plant is currently undergoing engineering evaluation for a new set of filters and other improvements.

AECOM Observations: The current plant is not flexible in coagulant dosing. The new system should have flexibility in coagulant dosing to achieve TOC removal. In addition, the distribution is not well looped with several fingers outside the main system. There is limited land on site, and proposed replacement must fit in the existing building.

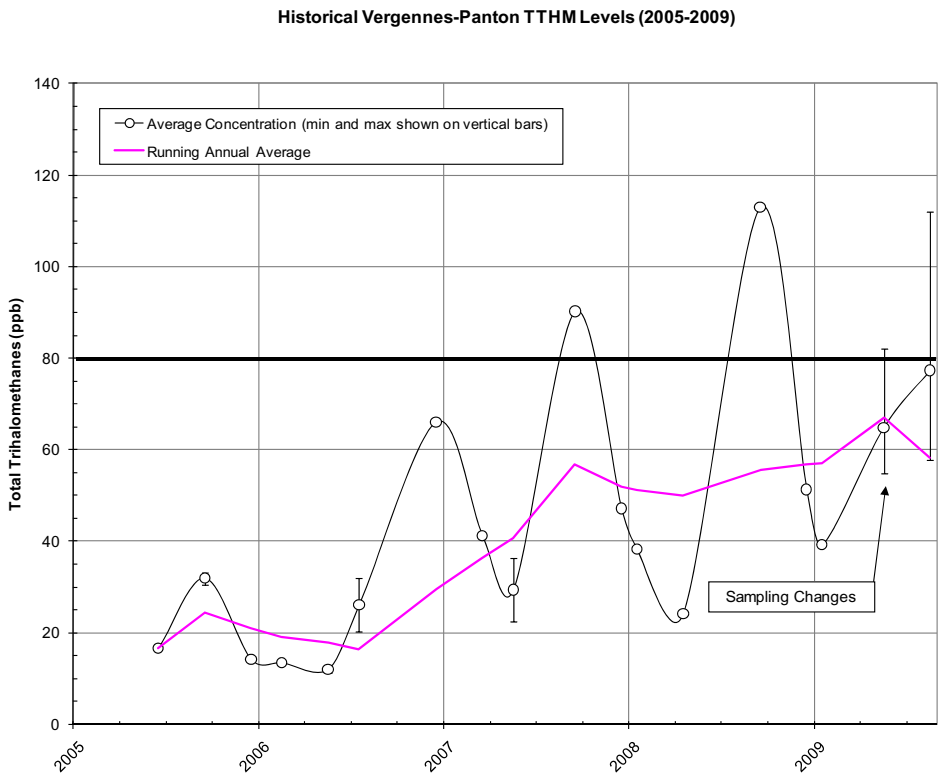
TABLE 5-52
VERGENNES-PANTON WATER DISTRICT SYSTEM SUMMARY

Population			
Value	5,000		
System Demand, MGD			
	February, 2008	May, 2008	August, 2008
Minimum	0.522	0.552	0.597
Average	0.549	0.639	0.657
Maximum	0.628	0.754	0.713
Distribution System Characteristics			
Parameter	**Value**	**Notes**	
Miles of Pipe	90		
-Estimated average diameter, inches	8 inch		
Approximate System Detention Time	Not known		
Number of Storage Tanks	3		
-Provide Size of each, MG	Distribution 0.50 0.75 Onsite at WTF 0.50		-1 steel tank in distribution system not in service
-Tank Type	Distribution –Both Concrete Onsite Steel		
Chemical Addition			
TBD based on new system			

5.2.3.44 Vergennes-Panton Water District - Proposed Alternatives

AECOM recommends that Vergennes-Panton Water District follow the steps provided in Table 5-53 to maintain compliance with the Stage 2 DBPR based on chlorine or chloramine.

Figures 5-50 and 5-51 provide the historical DBP RAA as well as the improved RAA for the recommended alternative(s) for chlorine as the secondary residual. It is forecasted that Vergennes-Panton will be required to operate the recommended treatment for 3 months (i.e. during the worst part of DBP season)



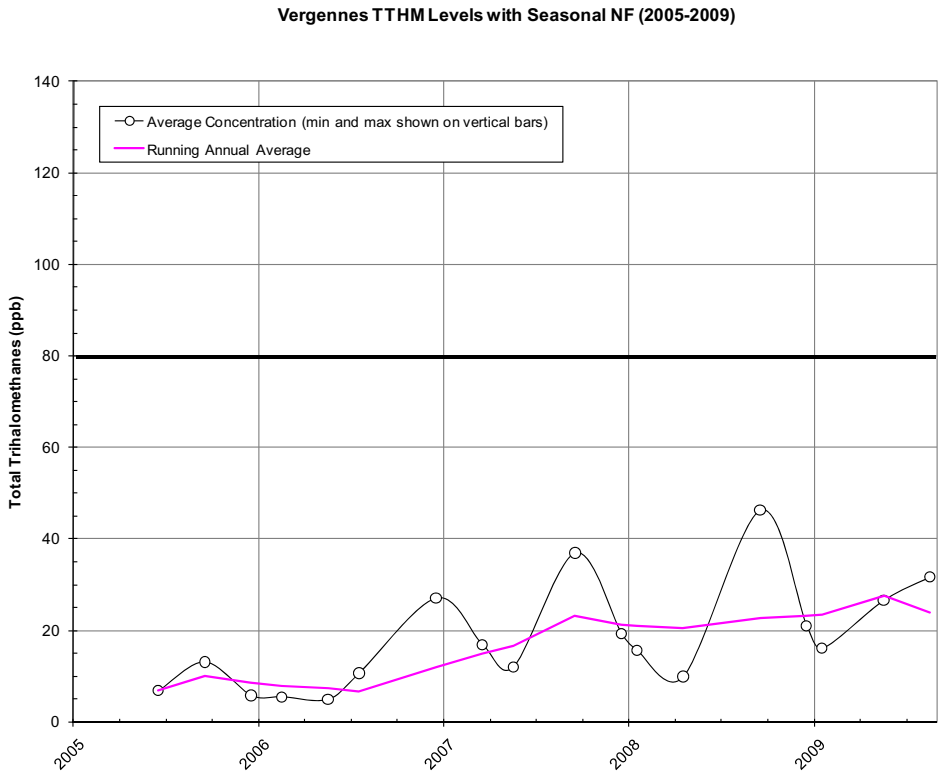
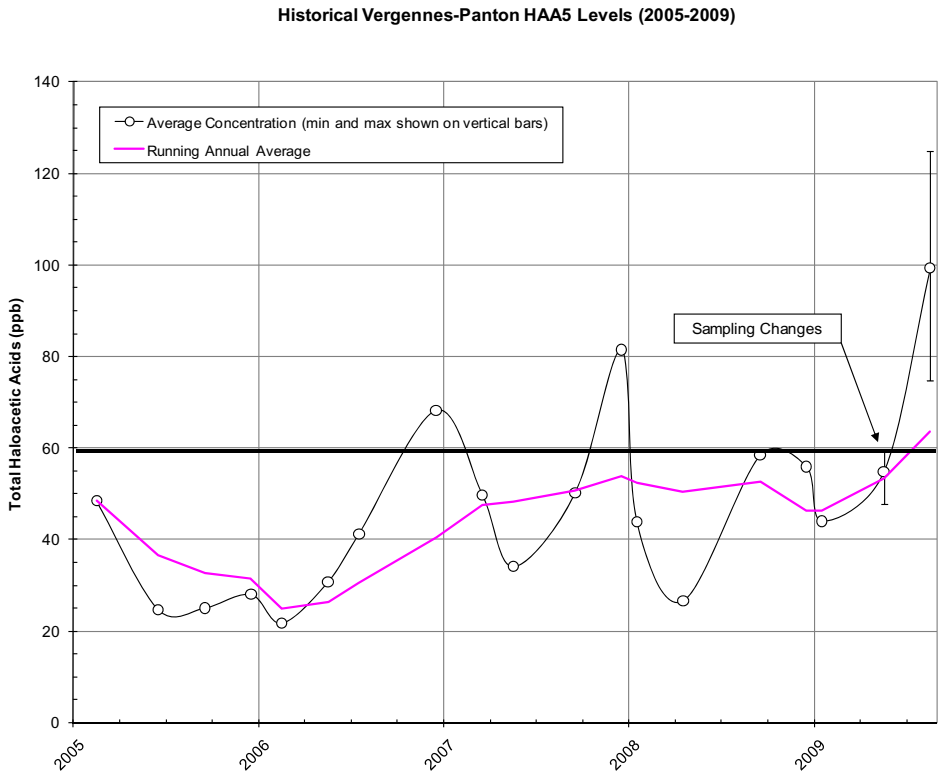


FIGURE 5-50
VERGENNES-PANTON TTHM LEVELS (2006-2009)



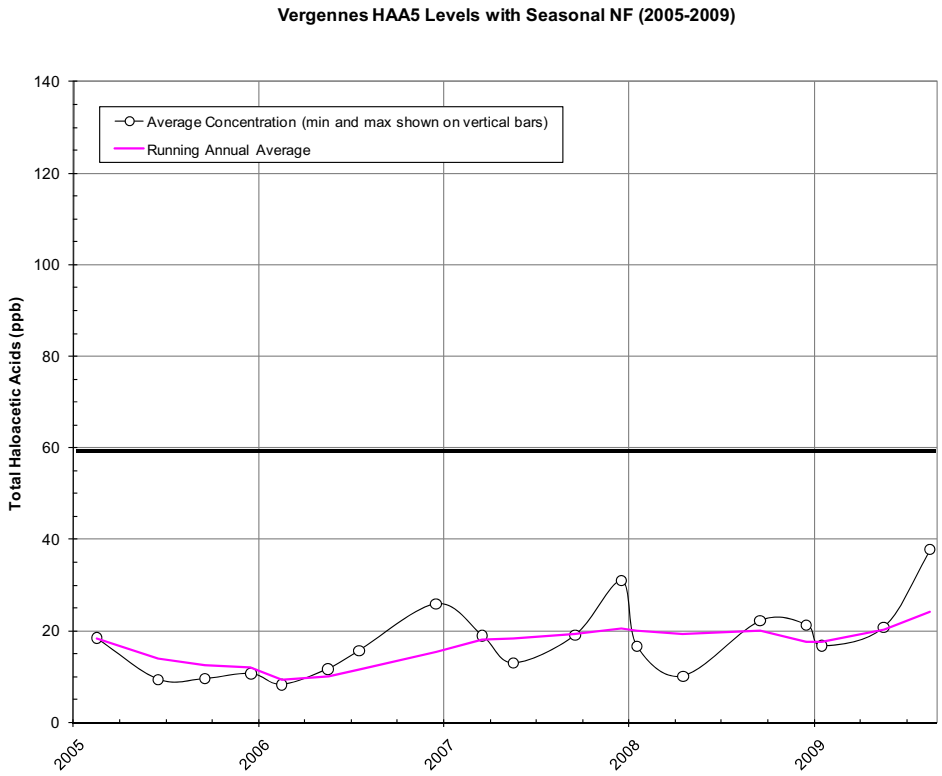


FIGURE 5-51
VERGENNES-PANTON HAA5 LEVELS (2006-2009)

to stay in compliance with the Stage 1 and 2 DBPR. The anticipated DBP level (summer) for chloramine can be found in Appendix 2 - Computer Model Output summary under Vergennes-Panton. The summer DBP levels for calibration were 103 ppb for TTHM and 60 ppb for HAA5. The modeled reduced DBP levels are as follows:

1. Chloramine as secondary residual: TTHM = 63 ppb, HAA5 = 44 ppb
2. UV and post clearwell chloramination: TTHM = 55 ppb, HAA5 = 28 ppb

Chlorine as Distribution Residual (Figures 5-52B)

1. Vergennes-Panton Water District is replacing the existing water treatment facility with new filtration equipment. The client's engineer (Otter Creek Engineering, Inc.) recommended a Trident HS packaged system. There has been an increasing trend of DBP levels in the latest DBP results. Typically, deep bed GAC filters would be used after the clarifiers for this application. However, since the existing building setup does not lend itself to deep bed GAC filtration for precursor removal. Thus room for nanofiltration membranes should be considered in the lower level where the existing filters are located.
2. Optimize coagulant for TOC removal.

Chloramine as the Distribution Residuals (Figure 5-52C)

Vergennes-Panton Water District has the following options to address DBP reduction while using chloramine as a residual in the distribution system:

1. The system should perform a simulated distribution system (SDS) bench top test to determine the potential DBP formation with chloramination. In AECOM's experience, addition of chloramine will likely reduce the distribution DBPs to a level below MCLs.

TABLE 5-53
PROPOSED ALTERNATIVES FOR VERGENNES-PANTON WATER DISTRICT

Treatment- Existing System	Distribution
For Chlorine	
1.) Buoyant contact clarifier followed by rapid sand filtration	None proposed
2.) Nanofiltration	
For Chloramine	
None proposed	Chloramines

5.2.3.45 Vergennes-Panton Water District - Other DBP Reduction Alternatives (Figure 5-52D)

A summary of other DBP reduction alternatives for Vergennes-Panton Water District is provided in Table 5-54. The following provides a brief discussion of each alternative.

1. Booster chlorination will allow the total time in which chlorine is in contact with finished water in the distribution system to be reduced.
2. Automatic flushing at the ends of the distribution allows the District to have some control over water age in the distribution. This has the drawback however of not consuming the finished water produced.

TABLE 5-54
VERGENNES-PANTON WATER DISTRICT: OTHER DBP REDUCTION ALTERNATIVES

Treatment- Existing System	Distribution	Effect*
None proposed	1.) Booster chlorination	Low to Medium
	2.) Automatic flushing	Low

*Effect is the anticipated relative effect the action has on reducing overall DBPs.

1. Low = Typically less than 20% DBP reduction
2. Medium = Typically less than 40% DBP reduction
3. High = Typically greater than 40% DBP reduction
4. Ranges = Depends on environmental conditions

FIGURE 5-52A
VERGENNES-PANTON WATER DISTRICT EXISTING PROCESS FLOW DIAGRAM

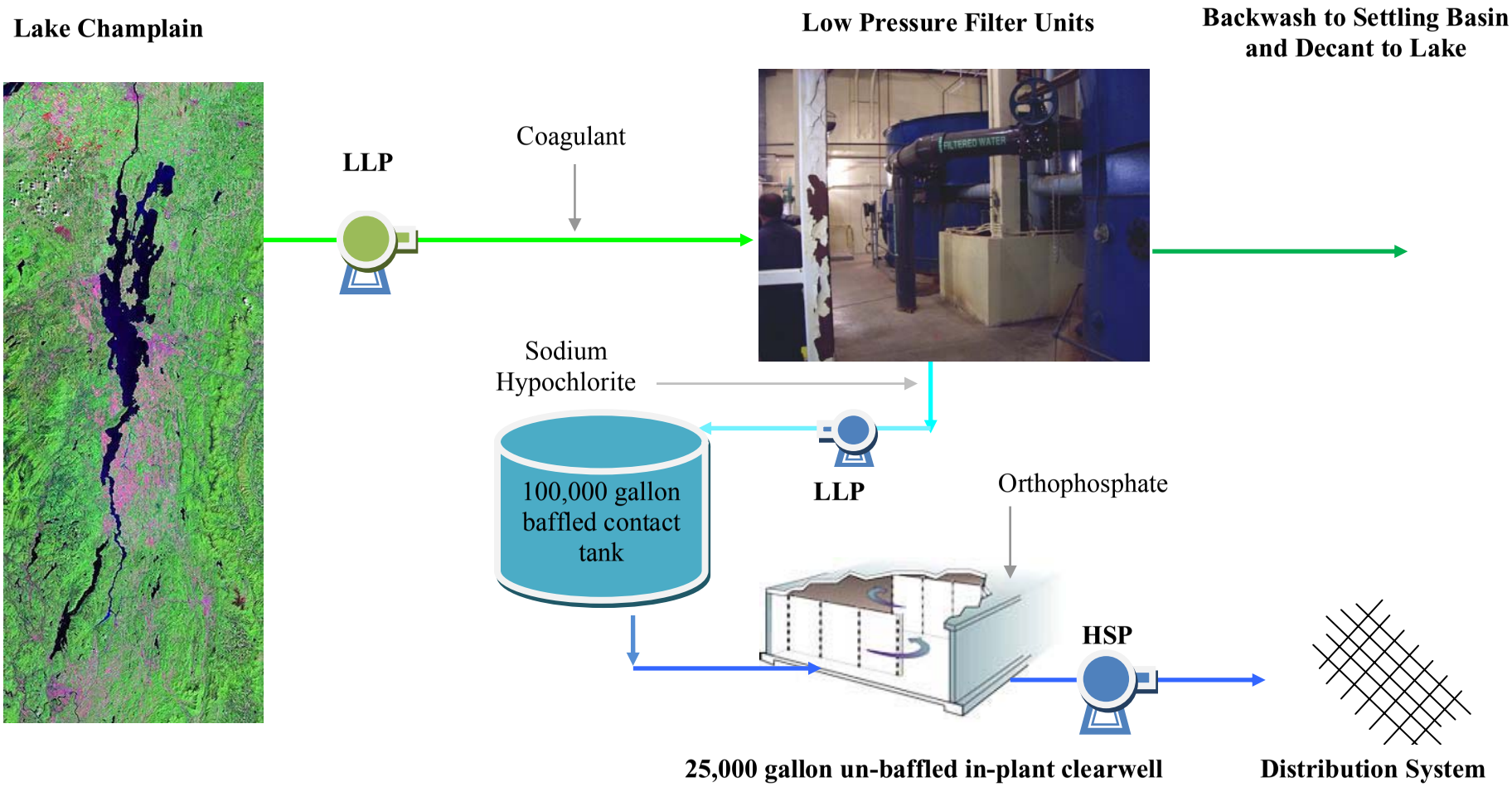


FIGURE 5-52B
VERGENNES-PANTON WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORINE
(UPDATES IN RED)

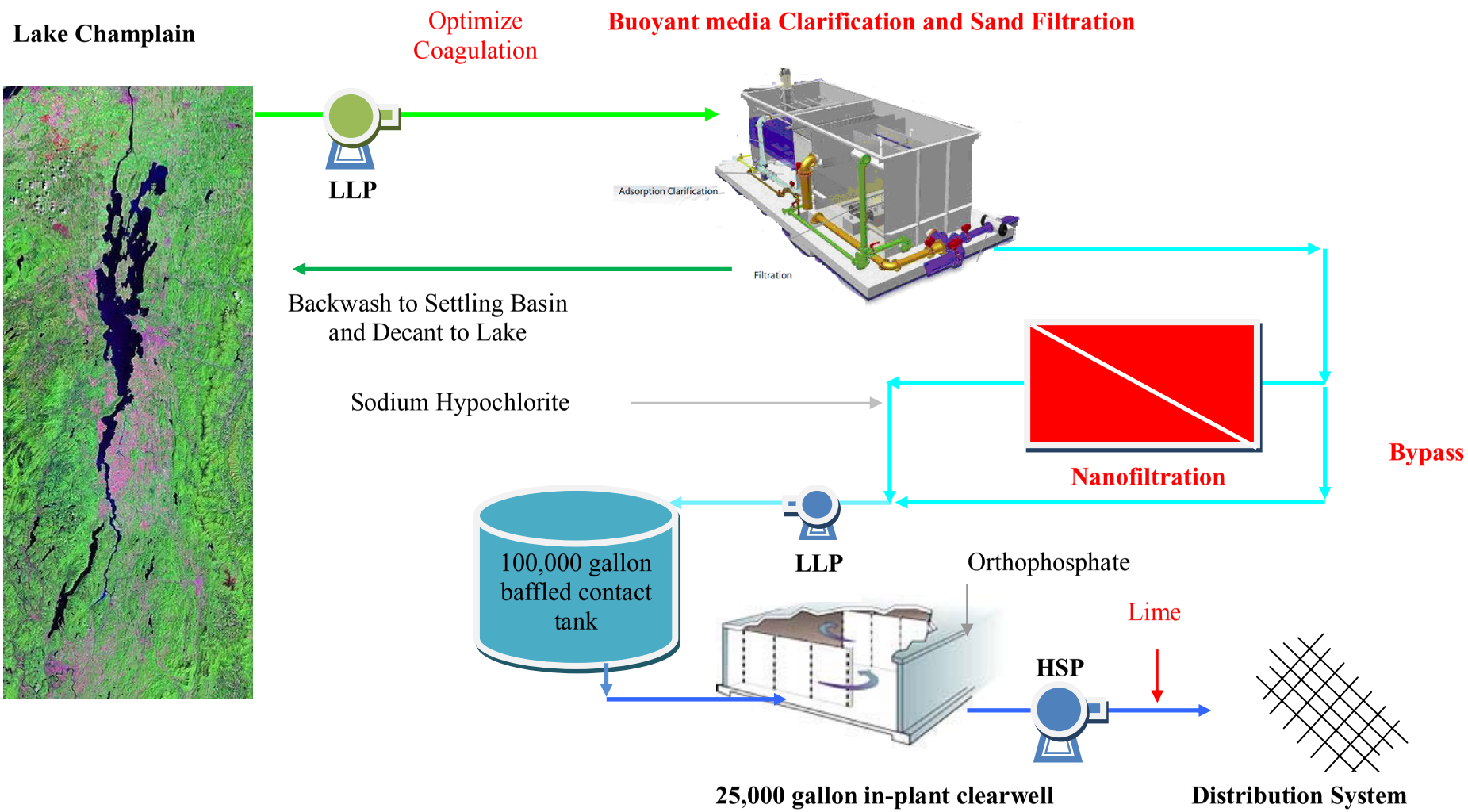


FIGURE 5-52C
VERGENNES-PANTON WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR CHLORAMINE (UPDATES IN RED)

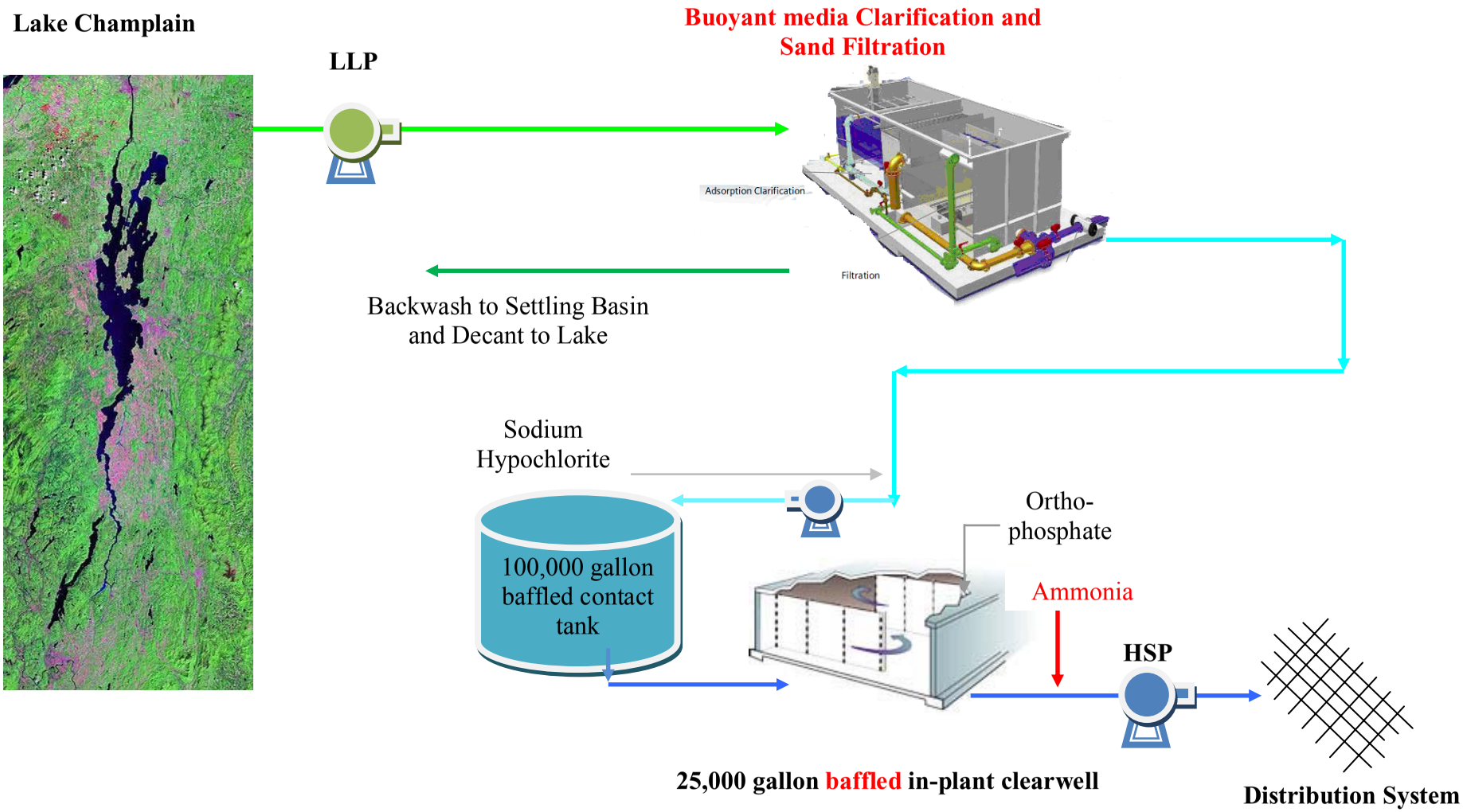
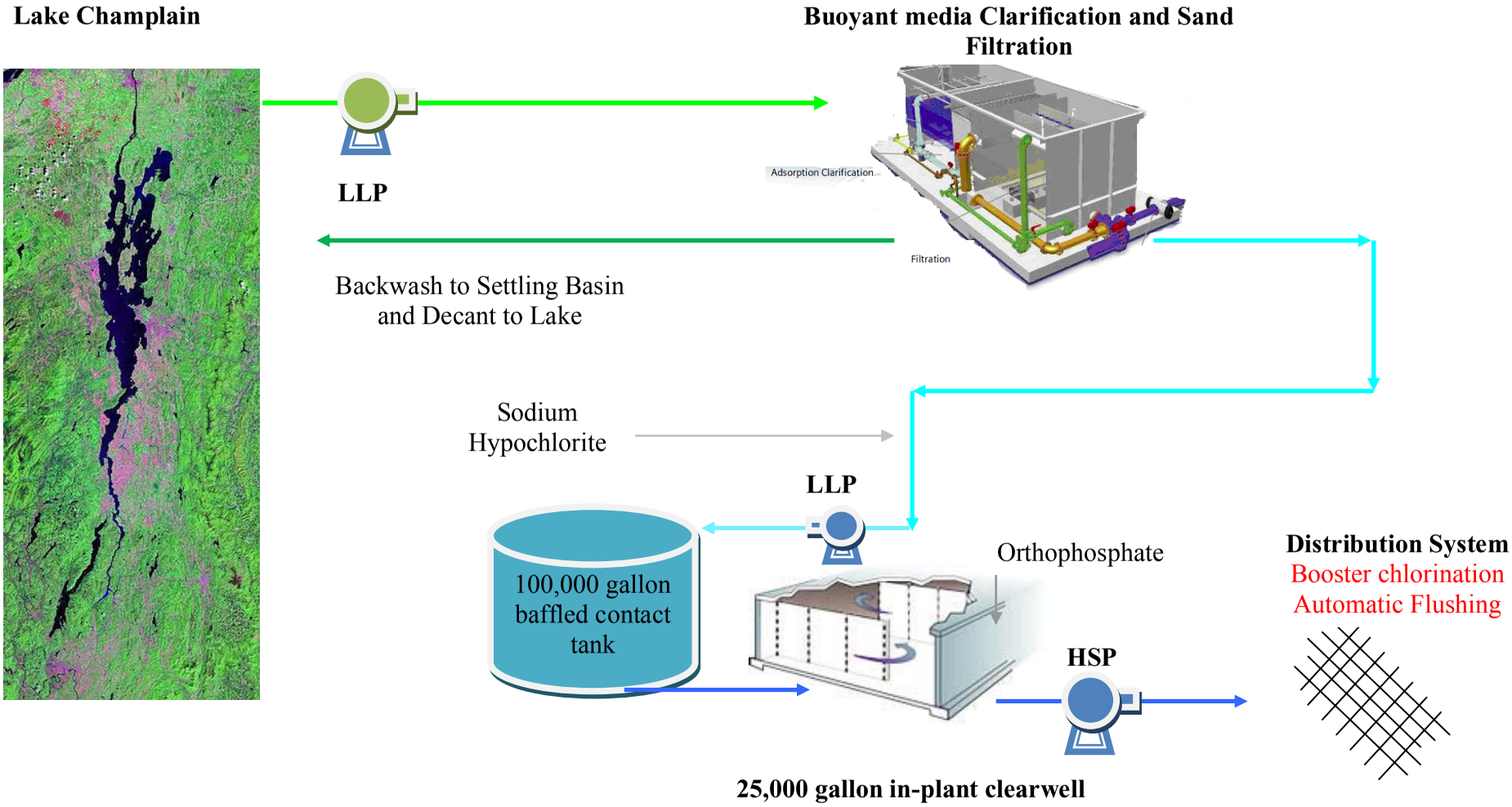


FIGURE 5-52D
VERGENNES-PANTON WATER DISTRICT PROPOSED PROCESS FLOW DIAGRAM FOR OTHER DBP REDUCTION OPTIONS (UPDATES IN RED)



5.3 CONCEPTUAL COST COMPARISONS

The purpose of the cost estimates included in this section is to assess the comparative costs between achieving USEPA Stage 1 and Stage 2 disinfection byproduct compliance one of two ways:

- Retain the use of free chlorine as a secondary residual but make other changes at the facility to enhance TOC removal (which in turn could be expected to lower DBPs). These other changes could be minor or significant, based on the severity of the current DBP compliance problem.
- Alternatively, switch from using free chlorine as the secondary disinfectant to chloramines. This would entail adding an ammonia feed system and potentially a UV disinfection system as well.

For both the free chlorine option and the chloramines option, order-of-magnitude capital (construction) and operation and maintenance costs were prepared for each site, according to the proposed approaches summarized in Section 5.2. For some systems, more than one treatment approach has been developed to provide a range of possible compliance alternatives. Sketches showing rough layouts of proposed processes are included in Appendix 7.

Please note that AECOM's scope for this project only covers evaluation of alternatives for compliance with the requirements of the DPBRs, using either chlorine or chloramine for secondary disinfection, and to provide cost estimates for the DEC and Legislature to consider in their decision making process. Issues of possible health concerns for chlorine or chloramine were not AECOM's purview. Further, AECOM makes no recommendation as to which disinfectant approach to use.

5.3.1 CAPITAL COST ESTIMATES

The accuracy of engineering construction costs estimates is proportional to the level of design detail. Accuracy is accounted for by use of contingency factors. AECOM uses the industry standard and widely accepted American Association of Cost Estimating (AACE) classification system to assign contingency. According to the AACE standard, preliminary estimates used for feasibility and conceptual design purposes are considered a Class 4 estimate, where Class 5 is for the most preliminary designs and Class 1 for the most complete. A Class 4 estimate carries a -30 to + 50 range of accuracy and is based on the use of factors, ratios, and other parametric techniques, as well as actual prices for

major items. A contingency of 30% has been assigned to the direct capital cost estimates. It must be emphasized that Class 4 estimates are not inclusive but are structured to capture the major cost elements of the various treatment alternatives. Further, these are not intended for appropriation of funding, but instead as a means of comparing alternatives prior to proceeding to the next phase of a project.

Capital construction cost estimates were developed for each site for both alternatives using the maximum flows outlined in Table 5-55. Costs were also developed for a well option at the Proctor and Readsboro sites, as well as for both a MIEX system and well option for the Rutland City site, and both a GAC and nanofiltration option for the CWD. Estimates were based on conceptual treatment designs developed from information collected during the December 2009 site visits, discussions with operators, and assessment of existing site conditions.

TABLE 5-55
SITE SUMMARY OF PROPOSED TREATMENT TO ADDRESS DBPS

Location	Capacity (mgd)		Free Chlorine			Chloramine Conversion	
	Max	Avg	Enhanced Coagulation	New Unit Process	Post-filter GAC	Chloramines	UV Disinfection
Alburgh	0.11	0.09	x	DAF		x	x
Bennington	3.0	1.6	x		x	x	x
Burlington	12.0	5.6	x	DAF		x	x
Catamount-Bolton	0.04	0.04	x	UV		x	x
Champlain	22.0	15.0		Booster Chlor.,& NF *or* GAC			
Grand Isle	0.20	0.13			x	x	x
North Hero	0.18	0.07	x		x	x	x
Proctor	0.41	0.41	x	DAF, UV, Wells		x	x
Readsboro	0.04	0.04		Wells	x	x	x
Richford	0.16	0.16			x	x	x
Rutland	5.6	2.3		MIEX, Wells	x	x	x
St Johnsbury	2.1	1.1			x	x	x
Swanton	0.70	0.60	x	DAF	x	x	x
Tri-Town	0.82	0.70	x		x	x	x
Vergennes	0.75	0.55	x	Trident/NF		x	x

5.3.2 Capital Cost Development

The capital (construction) costs were broken down into seven (7) categories:

- **Site Work**. Unit prices for excavation were applied per cubic yard of material or per linear foot of pipe trench. RS Means cost data were used to develop unit prices, as is the industry standard. The cost to install new wells at Proctor, Rutland, and Readsboro, as per the recommendations in Section 5-2, was included under this category.

- **Concrete.** Concrete for base slabs, walls, fill, and foundations was estimated on a cubic yard basis. Current unit prices for concrete (as installed) were then applied. A hardener and sealer cost for concrete floors was included on a per square foot basis.

- **Major equipment.** Budget prices were solicited from vendors for GAC filters, UV disinfection equipment, nanofiltration membranes, dissolved air flotation clarification systems, and certain pumps. Where applicable, prices form other recent projects were used as well as prices obtained from catalogs (McMaster-Carr, Burt Processing, Cole-Parmer, etc.). Chemical feed systems are included in this category.

- **Instrumentation.** This category includes turbidimeters, chlorine or ammonia residual analyzers, pH sensor, and other devices. Also, the cost to wire the various new input/output (I/O) points from the major equipment into the existing SCADA system was estimated on a per-I/O point basis.

- **Buildings.** The cost for either a pre-engineered metal building or a wood frame building, if needed, was handled on a per square foot basis, including lighting, HVAC, and appurtenances. Sizes of buildings were estimated based on ability to reasonable accommodate the new equipment (GAC filters, NF membranes, or UV systems, as applicable).

- **Piping.** Lengths of pipe were estimated by scaling directly off of the site drawings. Unit prices for pipes, valves, major fittings, were obtained from RS Means.

- **Electrical & Integration.** The electrical and integration costs were handled on a lump sum basis. The electrical costs were assumed to be 10% of the equipment costs only, that is, major equipment with electrical loads. Integration was estimated at 10% of the instrumentation costs. This is to account for reconfiguring existing SCADA system for new signals and devices.

Where appropriate, allowances for other works items such as work within confined spaces, coring or penetrations into existing concrete structures, and relocating existing utilities was added. In the Rutland and Champlain Water District (CWD) estimate, the cost for bringing 3-phase power to the water treatment plant site was included.

An average wage rate was calculated from 2008 RS Means Labor rates (ENR of 8092) for Construction Industry (ENR 8641). The reference city was Rutland, VT, with a construction wage of $26.45, escalated at 7% for today's ENR index of 8660 (January, 2010). Added to this was 31.2% for insurance, to bring the total average wage to $37.14 per hour. This wage was used throughout the capital costs estimates.

Because it would not be intuitive to the reader to see how costs are related to specific treatment recommendations, the cost categories above are presented and organized based on their contribution to the following:

For the free chlorine alternatives:

- Enhanced coagulation (chemical metering systems, mixing, bulk tanks and day tanks)

  [It should be noted that some facilities where enhanced coagulation was recommended are already equipped with the means to practice this. In these instances, only operations and maintenance (O&M) costs for the increased use of chemicals has been included.]

- UV disinfection (UV reactors, piping, instrumentation)

- GAC adsorption (filters, media, buildings for filters, piping)

- Replacement unit process (new package DAF system, MIEX system, Trident system, or booster chlorination)

- Nanofiltration (NF skid, feed pumps, membranes, and building where necessary)
- Well development

For conversion to chloramines:

- UV disinfection (UV reactors, piping, instrumentation)
- Ammonia feed system (dry ammonium sulphate feed systems, ammonia residual analyzers)

Table 5-56 and 5-57 summarize the cost breakdown for the free chlorine and chloramine alternatives, respectively. Separate costs are shown for the communities where well development (Proctor, Readsboro, and Rutland) and MIEX (Rutland) are proposed for the free chlorine alternatives. Separate costs are also shown for the GAC and nanofiltration options developed for the CWD. As shown in Table 5-56 and 5-57, the "Subtotal Conceptual Costs" are simply the sum of all the conceptual costs of each of the five potential process components, with no factors applied. The 30% contingency factor is applied next, resulting in a total direct cost. A 20% allowance for contractor's overhead and profit was added to the total direct cost to obtain the total construction cost.

A 10% factor was added to the total construction cost to account for engineering services and permits for all communities except CWD, where a 15% factor was used for engineering and permitting because of anticipated issues related to the NF reject or GAC spent filter washwater back to Lake Champlain. (The additional allowance was not applied at Vergennes-Panton, where NF membranes are also proposed, because the reject stream is small.) For all locations, an allowance of 10% for construction services was also added to the total construction cost for each community. The final cost, excluding piloting and bench top work, is derived by applying a series of factors yielding a combined multiplier of 1.89 on top of original subtotal conceptual cost (1.3 x 1.2 x 1.1 x 1.1 = 1.89).

Single or multiple potential approaches to DBP compliance have been developed for each system. However, it is possible that achieving compliance with the DBP rule could be attained by making relatively small changes, such as practicing enhanced coagulation, discontinuing pre-chlorination, and minimizing chlorine contact time, so that advancing to more costly efforts such as installing a new GAC filtration system would not be needed.

Pilot Study Considerations

Line items for bench top jar testing and pilot studies have been provided. For all options involving enhanced coagulation, it is recommended that a bench top jar testing program be conducted in order to establish the proper coagulant dose, coagulation pH, and expected additional removal of TOC. This will provide design criteria for eventual changes to the coagulation process. The cost for jar testing includes the development of a testing protocol (suitable to be submitted to VTDEC), a 3-day on site jar testing experimental program, analytical lab work to test TOC and simulated distribution system disinfection by-products, and a final report with recommendations. The estimated cost for this work is $6,800.

Dissolved air flotation (DAF) piloting will be warranted where DAF is a recommended replacement technology. Pilot studies are crucial in establishing levels of anticipated treatment, design criteria, and ultimately for confirmation that the system is a viable technology for the particular source water. It is assumed that two seasons of piloting would be conducted for three weeks during each season. One season would capture the cold water when coagulation is most difficult, and another season would be conducted when the source water organics are highest. The cost to conduct the piloting would include the preparation of a protocol, design of the pilot setup, leasing of a pilot unit and operation of the unit for 3-weeks, demobilization, outside laboratory analytical work, and development of a report suitable for submittal to DEC detailing the performance of the system and the recommend full-scale design criteria. The estimated cost for this work is $90,000.

Similarly, nanofiltration (NF) piloting would be required where NF systems are recommended. This would entail only a single 30-day study to coincide with the season in which organics are most prevalent, and in which the full-scale NF would actually be in use. Leasing costs, membranes, operations, analytical work, and shipping of the unit would be included at a total estimated cost of $27,000.

Although piloting of GAC filtration is a possibility as well, another method of assessing GAC adsorption capabilities exists. It is referred to as rapid small-scale column tests (RSSCTs). In this technique, a mathematical model is used to scale down from the proposed full-scale adsorber to a small diameter filter column which can be setup in a lab as opposed to an on-site pilot plant. Carbon media suppliers can conduct these experiments and require only a sample of the water to be fed to the GAC filter for testing with. The cost to conduct RSSCT test is $5000.

Well Development

Capital costs for well development were assessed for Rutland, Readsboro, and Proctor . Test well investigations, well site approvals, and land acquisition were included in the costs, along with the well and pump station construction. These costs are listed in the same rows in Table 5-56 along with pilot and bench top testing, with notes indicating what they are. Test well investigation costs are based on the test wells themselves, geophysical surveys, and hydrogeological consulting services. Pump testing is included in the approval cost. Also, an allowance of $50,000 per acre was budgeted for land acquisition. Further basis for cost are provided below.

<u>Basis for cost estimates -Rutland:</u>

1. Well exploration will focus along East Creek north of the confluence of Mendon Brook. This area is mapped as an extensive kame terrace deposit, which are normally composed chiefly of sand and gravel. This deposit could be more than one hundred feet thick.
2. Develop a supply of about 1 to 1.5 MGD of groundwater, which represents about half of Rutland's average day demand. This will supplement Rutland's existing surface water supply, with the expectation of reducing overall DBP formation.

3. Test well investigation costs are based on 1,000 linear feet of 2.5-inch diameter test wells, geophysical surveys, plus hydrogeological consulting services. For budgeting purposes, we have assumed test well depths of 100 feet.
4. Well site approval costs are based on two pumping tests, each involving a 100-foot deep x 12-inch diameter test well, five days of continuous pumping and three observation wells, plus water quality, survey and hydrogeological consulting and permitting services.
5. We have budgeted a total of 6,000 feet of water main and 6,000 feet of new electrical power to connect the new wells to the City Reservoir.
6. The cost estimates are based on current pricing. Rutland may elect to undertake this project in phases over several years. Pricing should be revisited once the test-well program is complete to more accurately reflect actual quantities and designs.

Basis for cost estimates - Proctor:

1. The Town of Proctor will have the capability of pumping approximately 500,000 gallons per day (350 gallons per minute on a 24-hour-per-day basis) from its Field Street well, once it is in service. The peak day demand in Proctor is approximately 400,000 gpd. A second well is needed for redundancy in case of failure of the Field Street well. The second well will also create flexibility in operating the water system, such as may be required for maintenance, water quality or other purposes. A well west of Otter Creek creates balance from the standpoint of water-supply protection, and may enhance water-system pressures.
2. We have assumed the test-well investigation will focus on a large area in the northern end of town between Gorham Bridge Road and Otter Creek. We have budgeted for up to 8 test wells, each 150 feet deep (approximately $70 per foot) at this location, with an allowance for
3. Geophysical services, and hydrogeological consulting services. We selected a depth of 150 feet for budgeting purposes; by comparison, the Field Street well is 125 feet deep.
4. We have budgeted for a five-day pumping test, a single 12-inch diameter test-pumping well, three observation wells and hydrogeological consulting services to permit the well under VT DEC regulations.

5. The goal is to develop a new well with a capacity of at least 350 gallons per minute. We have budgeted for a 12 x 24-inch gravel-packed well, 150 feet deep, at an approximate cost of $1,000 per foot.
6. 6-inch diameter water main is available on Elm Street. We have assumed that an additional 2,000 feet of water main ($120 per foot) will be needed to connect the new well with the main on Elm Street. In addition, 3-phase power is available near Elm Street/Willow Street. We have assumed that an additional 2,000 feet of 3-phase power ($100 per foot) will be needed to connect the new well to existing power.
7. The cost estimates are based on current pricing. Proctor may elect to undertake this project in phases over several years. Pricing should be revisited once the test-well program is complete to more accurately reflect actual depths and distances.

Basis for cost estimates - Readsboro:

1. One well at 75 gallons per minute to meet peak demands.
2. Test well program costs are based on one week of exploration in the glacial overburden using a mini-sonic drilling rig, and three 6.5-inch diameter test wells in fractured bedrock, plus hydro-fracturing of two bedrock wells, plus hydrogeological services and water-quality testing.
3. Test drilling will take place along the banks of the Deerfield River southeast of town between the wastewater plant and Harriman Station.
4. The costs for well approval and construction include the installation of a 70-foot deep 12-inch diameter well for test pumping; and a three-day pumping test with three observation wells. If successful, the 12-inch well will become the permanent production well.
5. One mile of new water main is budgeted to connect the well with the existing water system. A river crossing may or may not be required, and was not included in the estimate.
6. 3-Phase electric power appears to be available on both sides of the Deerfield River from the wastewater plant to the town line.
7. The cost estimates are based on current pricing. Readsboro may elect to undertake this project in phases over several years. Pricing should be revisited once the test-well program is complete to more accurately reflect actual depths and distances.

Community Specific Considerations

Readsboro, Proctor, and Rutland were treated differently than the other sites for the chlorine option, as the costs for wells were developed independently of the other proposed treatment alternatives. A third independent option for Rutland was estimated that involved the application of a MIEX system. Independent evaluation was necessary as if any of these options were pursued, the other proposed treatment alternatives would likely be unnecessary.

Assessment of options for CWD was also unique. At CWD, chloramination is already being practiced. Therefore, only the cost of mitigating DBP concerns using free chlorine was evaluated. Both GAC filtration and NF treatment were evaluated. The existing CWD site is essentially fully built-out, and the only feasible space available for construction is reserved for a second finished water storage tank. Therefore, a site approximately 0.3 miles west of the treatment plant, at 149 Industrial Parkway, was identified as a potential location (see Appendix 7) based on discussions with local realtor , Mike Simoneau (Hickok & Boardman Realty) for either an NF or GAC facility. A 3-acre parcel was assumed for both, including setbacks, parking, paved surfaces, and a building space. Based on conversations with a local realtor, a $300,000 cost per acre was carried.

Filtered water from the existing treatment plant would be pumped to this site, and treated water would then flow back to the existing WTP for final treatment. Waste streams consisting of either spent filter wash water in the case of a GAC system or membrane reject in the case of NF treatment, would be pumped to the lake through an outfall. A $10/ft^2 Excavation Fee was also carried in the CWD capital costs. This fee may be levied by the City of Burlington for the excavation in a paved right of way that would necessary as part of the GAC and NF alternatives. Also different in the Champlain Chlorine Option capital cost estimate was an additional allowance of 5% of the total conceptual cost for permitting to address membrane reject disposal.

Capital costs provided by the CWD for their conversion to chloramination are provided in the Present Worth summaries in Executive Summary provide a capital cost comparison. It should be noted that the capital costs do not include UV treatment, but represent a significantly larger system than any of the other sites, and reflect site specific challenges.

TABLE 5-56
CHLORINE OPTION CAPITAL COST SUMMARY

Process Components	Location									
	Alburgh	Bennington	Burlington	Catamount	Champlain (NF)	Champlain (GAC)	Grand Isle	North Hero	Proctor	Proctor (Well)
Enhanced Coag.	$25,881	$99,210	$150,570	$37,442	$0	$0	$0	$23,365	$25,624	
UV Disinfection	$0	$0	$0	$47,974	$0	$0	$0	$0	$123,707	
GAC Adsorption	$0	$1,657,327	$0	$0	$0	$10,695,539	$424,988	$280,034	$0	
(1)Replacement unit process	$879,884	$0	$1,941,336	$0	$491,673	$491,664	$0	$0	$700,525	$756,250
Nanofiltration	$0	$0	$0	$0	$14,000,145	$0	$0	$0	$0	
Subtotal Conceptual Cost ($)	$906,000	$1,757,000	$2,092,000	$85,000	$14,492,000	$11,187,000	$425,000	$303,000	$850,000	$756,000
30% Contingency ($)	$272,000	$527,000	$628,000	$26,000	$4,348,000	$3,356,000	$128,000	$91,000	$255,000	$227,000
Total Direct Cost ($)	$1,178,000	$2,284,000	$2,720,000	$111,000	$18,840,000	$14,543,000	$553,000	$394,000	$1,105,000	$983,000
20% Contractors OH&P ($)	$236,000	$457,000	$544,000	$22,000	$3,768,000	$2,909,000	$111,000	$79,000	$221,000	$197,000
Total Const. Cost ($)	$1,414,000	$2,741,000	$3,264,000	$133,000	$22,608,000	$17,452,000	$664,000	$473,000	$1,326,000	$1,180,000
Pilot & Bench Studies:										
Enhanced Coag. Jar testing ($)	$6,800	$6,800	$6,800	$6,800				$6,800	$6,800	(3)$310,000
DAF Piloting ($)	$90,000		$90,000						$90,000	(4)$200,000
NF piloting ($)					$27,000					
GAC study (rapid scale) ($)		$5,000				$5,000	$5,000	$5,000		
(2)Engineering/permits ($)	$141,000	$274,000	$326,000	$13,000	$2,261,000	$1,745,000	$66,000	$47,000	$133,000	$118,000
Construction Services ($)	$141,000	$274,000	$326,000	$13,000	$2,441,000	$1,925,000	$66,000	$47,000	$133,000	$118,000
Total Project Cost ($)	**$1,793,000**	**$3,301,000**	**$4,013,000**	**$166,000**	**$27,337,000**	**$21,127,000**	**$801,000**	**$579,000**	**$1,689,000**	**$1,926,000**

(1) At Alburgh, replacement unit process is dissolved air flotation as a package system
At Burlington, replacement unit process is dissolved air flotation in existing basins
At Champlain, replacement unit process is free chlorine instead of chloramines, requiring booster chlorination stations at 9 distribution system locations
At Readsboro, replacement unit process is new wells
At Swanton, replacement unit process is dissolved air flotation as a package system
(2) Engineering & permits is assumed at an additional 10% to the total construction cost.
(3) Represents well site testing and approvals.
(4) Represents land acquisition costs ($50,000/acre).

TABLE 5-56 (Continued)
CHLORINE OPTION CAPITAL COST SUMMARY

Process Components	Location									
	Readsboro	Readsboro (Well)	Richford	Rutland	Rutland (MIEX)	Rutland (Wells)	St Johnsbury	Swanton	Tri-Town	Vergennes
Enhanced Coag.	$0		$0	$0	$0		$0	$0	$0	$0
UV Disinfection	$0		$0	$0	$0		$0	$0	$0	$0
GAC Adsorption	$98,066		$349,443	$3,988,139	$0		$1,752,447	$914,946	$827,721	$0
[1]Replacement unit process	$0	$779,165	$0	$0	$3,244,984	$2,612,500	$0	$1,172,860	$0	$1,140,101
Nanofiltration	$0		$0	$0	$0		$0	$0	$0	$1,677,375
Subtotal Conceptual Cost ($)	$98,000	$779,000	$349,000	$3,988,000	$3,245,000	$2,613,000	$1,752,000	$2,088,000	$828,000	$2,817,000
30% Contingency ($)	$29,000	$234,000	$105,000	$1,196,000	$974,000	$784,000	$526,000	$626,000	$248,000	$845,000
Total Direct Cost ($)	$127,000	$1,013,000	$454,000	$5,184,000	$4,219,000	$3,397,000	$2,278,000	$2,714,000	$1,076,000	$3,662,000
20% Contractors OH&P ($)	$25,000	$203,000	$91,000	$1,037,000	$844,000	$679,000	$456,000	$543,000	$215,000	$732,000
Total Const. Cost ($)	$152,000	$1,216,000	$545,000	$6,221,000	$5,063,000	$4,076,000	$2,734,000	$3,257,000	$1,291,000	$4,394,000
Pilot & Bench Studies:										
Enhanced Coag. Jar testing ($)		[3]$265,000				[3]$450,000		$6,800	$6,800	$6,800
DAF Piloting ($)		[4]$200,000				[4]$400,000		$90,000		
NF piloting ($)										$27,000
GAC study (rapid scale) ($)	$5,000		$5,000	$5,000			$5,000	$5,000	$5,000	
[2]Engineering/permits ($)	$15,000	$122,000	$55,000	$622,000	$506,000	$408,000	$273,000	$326,000	$129,000	$439,000
Construction Services ($)	$15,000	$122,000	$55,000	$622,000	$506,000	$408,000	$273,000	$326,000	$129,000	$439,000
Total Project Cost ($)	**$187,000**	**$1,925,000**	**$660,000**	**$7,470,000**	**$6,075,000**	**$5,742,000**	**$3,285,000**	**$4,011,000**	**$1,561,000**	**$5,306,000**

[1] At Alburgh, replacement unit process is dissolved air flotation as a package system
At Burlington, replacement unit process is dissolved air flotation in existing basins
At Champlain, replacement unit process is free chlorine instead of chloramines, requiring booster chlorination stations at 9 distribution system locations
At Readsboro, replacement unit process is new wells
At Swanton, replacement unit process is dissolved air flotation as a package system
[2] Engineering & permits is assumed at an additional 10% to the total construction cost.
[3] Represents well site testing and approvals.
[4] Represents land acquisition costs ($50,000/acre).

TABLE 5-57
CHLORAMINE OPTION CAPITAL COST SUMMARY

Process Components	Location						
	Alburgh	Bennington	Burlington	Catamount	Grand Isle	North Hero	Proctor
Ammonia system (chloramines)	$30,973	$39,855	$43,683	$29,423	$31,676	$31,676	$43,111
UV Disinfection	$54,020	$234,027	$317,406	$48,470	$72,722	$72,722	$94,992
Subtotal Conceptual Cost ($)	$85,000	$274,000	$361,000	$78,000	$104,000	$104,000	$138,000
30% Contingency ($)	$26,000	$82,000	$108,000	$23,000	$31,000	$31,000	$41,000
Total Direct Cost ($)	$111,000	$356,000	$469,000	$101,000	$135,000	$135,000	$179,000
20% Contractors OH&P ($)	$22,000	$71,000	$94,000	$20,000	$27,000	$27,000	$36,000
Total Const. Cost ($)	$133,000	$427,000	$563,000	$121,000	$162,000	$162,000	$215,000
Engineering/permits ($)	$13,000	$43,000	$56,000	$12,000	$16,000	$16,000	$22,000
Construction Services ($)	$13,000	$43,000	$56,000	$12,000	$16,000	$16,000	$22,000
Total Project Costs ($)	**$159,000**	**$513,000**	**$675,000**	**$145,000**	**$194,000**	**$194,000**	**$259,000**

Process Components	Location						
	Readsboro	Richford	Rutland	St Johnsbury	Swanton	Tri-Town	Vergennes
Ammonia system (chloramines)	$29,251	$30,119	$42,829	$39,001	$34,316	$35,816	$34,316
UV Disinfection	$48,297	$53,166	$316,552	$233,173	$129,652	$161,152	$129,652
Subtotal Conceptual Cost ($)	$78,000	$83,000	$359,000	$272,000	$164,000	$197,000	$164,000
30% Contingency ($)	$23,000	$25,000	$108,000	$82,000	$49,000	$59,000	$49,000
Total Direct Cost ($)	$101,000	$108,000	$467,000	$354,000	$213,000	$256,000	$213,000
20% Contractors OH&P ($)	$20,000	$22,000	$93,000	$71,000	$43,000	$51,000	$43,000
Total Const. Cost ($)	$121,000	$130,000	$560,000	$425,000	$256,000	$307,000	$256,000
Engineering/permits ($)	$12,000	$13,000	$56,000	$43,000	$26,000	$31,000	$26,000
Construction Services ($)	$12,000	$13,000	$56,000	$43,000	$26,000	$31,000	$26,000
Total Project Costs ($)	**$145,000**	**$156,000**	**$672,000**	**$511,000**	**$308,000**	**$369,000**	**$308,000**

Operation and Maintenance Cost Estimates

Comparative operations and maintenance (O&M) cost estimates were developed for each site based on the free chlorine and chloramine conversion (with UV) alternatives outlined in the previous section and the flows listed in Table 5-55. The O&M costs were determined by estimating chemical dosages, power costs, and manpower requirements. Replacement costs for membranes, media, UV lamps, and cartridge filters were provided by equipment vendors, where applicable. Annual O&M cost estimates for the likely DBP compliance approaches for each system are summarized in Table 5-58. As discussed in Section 5.0, more than one approach was developed for some systems. These are italicized and indented in the row(s) below the most likely approaches that could achieve compliance. Note that the annual O&M costs are either for chlorine as the residual disinfectant, or for chloramines (with or without UV). Thus, the costs for a given community are not additive.

TABLE 5-58
OPERATIONS AND MAINTENANCE SUMMARY

Community[1]	**Annual O&M Costs**		
	Chlorine	**Chloramine with UV**	**Chloramine Only**
Alburgh (DAF)	$ 147,000	$ 54,000	$ 13,000
Burlington (DAF)	177,000	56,000	21,000
Bennington (GAC)	356,000	48,000	15,000
Catamount-Bolton (UV)	28,000	27,000	12,000
Champlain (NF)	1,673,000	n/a	[3]68,000
Champlain (GAC)[2]	2,139,000		
Proctor (DAF,UV)	234,000	30,000	13,000
Proctor (Well)[2]	85,000		
Grand Isle (GAC)	45,000	29,000	13,000
North Hero (GAC)	92,000	29,000	12,000
Readsboro (GAC)	45,000	27,000	12,000
Readsboro (Well)[2]	30,000		
Richford (GAC)	39,000	28,000	13,000
Rutland City (MIEX)	160,000	51,000	16,000
Rutland City (GAC)[2]	1,153,000		
Rutland City (Well)[2]	128,000		
St Johnsbury (GAC)	182,000	47,000	14,000
Swanton (DAF,GAC)	299,000	30,000	13,000
Tri-Town (GAC)	109,000	34,000	13,000
Vergennes-Panton (NF,Trident)	192,000	30,000	13,000

[1] Treatment in parenthesis represents most likely treatment option for chlorine alternative only
[2] Indented and italicized sites indicate alternative treatment options to the most likely chlorine alternative treatment option
[3] Chloramine Option O&M costs based on existing information provided by the Champlain Water District

Similar to the capital costs, the O&M costs for the proposed treatment alternatives presented for both the chlorine and chloramine option were broken out to show the contribution of each recommended treatment process. As the CWD already practices chloramination, existing inclusive annual O&M expenses are provided. All other chloramination costs only include operational expenses over and above what would already be incurred (e.g. chlorine costs are existing and are therefore excluded, whereas ammonium sulphate costs are carried).

Operations and Maintenance Cost Development

Operation and Maintenance costs considered the electrical, chemical, and recurring maintenance and process equipment replacement expenses. Additional labor that would be necessary with the proposed process equipment was also considered. Development of the costs for each of these categories is described in the subsequent paragraphs. As with the capital costs estimates, it must be emphasized that these O&M costs are equally conceptual in nature, and are based on preliminary information and simplifying assumptions.

Electrical

Power usage inherent with each process alternative was estimated based on the loads of the introduced process equipment and pumping systems. Electrical costs also considered the power necessary for heating, ventilation, and cooling of any new building. An electrical power rate of $0.13 per kW-hr was used in the estimate based on an average calculated from various Vermont utility companies current rates (January, 2010). Rates used in determining the average electricity cost considered both the energy (kW-hr) and demand (kW) charges. Only charges over and above what currently would be on each water treatment facilities power bill were considered.

Chemical

Chemical costs include the total chemical use for each process alternative, developed based on the average flows at each site. Coagulants, lime and caustic for pH adjustment, membrane and UV lamp cleaning solutions, and GAC media replacement were considered. Coagulation chemical volumes were based on the additional coagulant necessary to achieve TOC removal above what is currently being dosed. A target total dose of coagulant of 33 mg/l was used as a baseline for all sites in which enhanced

coagulant was recommended. It was assumed that polymer doses would remain unchanged. Chemical costs were either obtained from vendors or from previous vendor estimates (2007) that were brought to 2010 dollars using a 10% inflation rate as a one-time adjustment on the 2007 costs.

The NF membranes will require cleaning with citric acid and caustic soda, to restore permeability. As it is not advisable to recycle these wastes to the headworks of the water treatment plant due to excess metals concentration and the potential for sequestering the coagulation process, costs for hauling spent NF membrane cleaning waste (unit disposal costs and tipping fees) were included in the O&M costs. A holding tank has been included in the capital costs for both the Champlain and Vergennes free-chlorine options, to which the spent cleaning wastes will be directed. On-site storage and disposal costs for the brine byproduct of the MIEX process were similarly assessed at the Rutland site.

Maintenance, Replacement, and Recurring Costs

Maintenance and replacements costs are generally defined as anticipated, regularly scheduled maintenance expenditures that result from replacement of process components, i.e., consumables. Costs for the replacement of membranes and UV lamps were considered, as was the annual re-calibration of the UV sensors. An additional 1% of the capital value of all major process equipment was carried for regular maintenance and parts replacement costs.

Municipal taxes and fees were not considered for any of the sites with the exception of the CWD. As the majority of the facilities are either small, or public, taxes are either not applicable or not significant. Tax for the CWD GAC and NF alternatives was assessed based on a rate of $2.25/$100 of valuation of the new treatment works. A City of Burlington Franchise Fee representing 3.5% of total revenue was also applied, on an annual basis.

Not considered in the O&M costs are potential costs associated with the discharge of phosphorous to Lake Champlain. Discharge credits may need to be purchased by the CWD (as well as Vergennes-Panton Water District) to allow disposal of NF concentrate back to Lake Champlain. As there is no precedent to date in the State of Vermont for purchasing phosphorous credits, a potential cost estimate cannot be generated at this time.

Labor

Implementation of the proposed processes would lead to increased labor costs in most cases, due to the introduction of new and potentially unknown process and associated additional tasks. It is likley, for example, that the practice of enhanced or optimized coagulation could require more attention on behalf of the operator in order to check coagulation pH, observation of raw water conditions that effect coagulation (i.e., UV-254, turbidity, color, alkalinity, etc.), making sure that flow pacing of chemicals is operational, and generally ensuring that the chemical feed systems are functioning properly. Also, another skill that operators would need to acquire would be the ability to experiment with coagulant doses & pH through jar testing (however, this would be a fairly infrequent task).

For other more significant changes in the treatment scheme, such as the addition of a new unit treatment process, operators would require training on the new process. However, this is normally in the scope of supply of the equipment supplier, and operation of the new equipment would be under direct supervision of the equipment supplier until such time that the operator has the proper understanding of the system. This is usually followed by a period of a few weeks to months where the operator and supplier (and consulting engineer) are in frequent contact while the finer points of the new technology as well as trouble shooting techniques are witnessed by the operator. According to the State of Vermont Water Supply Division, Environmental Protection Rules Chapter 21 (April 2005) Subchapter 21-12, all of the surface water treatment plants fall under a Class 4 designation, which is the highest grade. Sub-classification 4C, specifically, requires the most operating experience, at 3 years. However, Classes 4 A, B, and C are differentiated based on population served, not by treatment technology. Therefore, the potential addition of new processes as described herein will not change the operator's level of certification, and re-certification of operators to a higher grade will not be required.

Staff cost increases were accounted for through the addition of labor in eight hour shifts. Each process was assessed, and an appropriate number of shifts per week were allocated based on the estimated operator time. A loaded labor rate of approximately $29 was used based on a 40 hour work week.

Enhanced or optimized coagulation and UV treatment were considered minor additions to an operator's workload and therefore one shift per week was added. As the GAC contactors involve manual operation and will have a more significant labor implication, one shift per operational GAC unit was accordingly accounted for. Based on vendor recommendations, one shift per week was accounted for with the MIEX system.

The Champlain Water District and Burlington each have unique labor considerations. The cost to Champlain Water District for operating nine (9) new booster chlorination sites was assumed to require the addition of a full-time (7 shifts per week) staff member. An additional two daily shifts of two operators was considered for the GAC and NF options, equating to four new operators.

As the proposed treatment under the chlorine option for Burlington involves replacing the Superpulsator clarifiers with a dissolved air flotation system, the labor was assumed available and no additional labor costs were carried. Unlike Burlington, additional labor was considered for the replacement of the pressure filters with the Trident system at Vergennes-Panton. This is due to the additional complexity associated with the Trident systems.

Labor hours considered for chlorine alternative for each of the communities are summarized in Table 5-58. Each shift represents eight hours, organized in the table to match the associated treatment process. Labor hours for the chloramine alternative were the same for each community and were assumed to be one shift per week for UV treatment and another shift for chloramines application.

TABLE 5-59
LABOR HOURS SUMMARY

Community	New Labor (Shifts/Wk)[1]		Treatment		Comments
	A	B	A	B	
Alburgh	7	1	DAF	EC	
Burlington	0		DAF		Assumed same labor requirements as existing process
Bennington	4	1	GAC	EC	
Catamount-Bolton	1	1	EC	UV	
Champlain	28	7	NF or GAC	Booster Chlor.	
Proctor	7	1	DAF	UV	
	2	0	New well[2]		
Grand Isle	2	1	GAC		
North Hero	2	1	GAC	EC	
Readsboro	1	3	GAC	Operational	Full time operation
	2	0	New well[2]		
Richford	1	0	GAC		
Rutland City	1	0	MIEX		Estimated by vendor
	8	0	GAC[2]		Estimated by vendor
	2	0	New wells[2]		
St Johnsbury	2	0	GAC		
Swanton	7	2	DAF	GAC	
Tri-Town	2	1	GAC	EC	
Vergennes-Panton	7	7	NF	Trident	

[1] Shift represents 8 hours
[2] Alternative treatment options to the most likely option that is shown in the first line for each community
EC = Enhanced coagulation, UV = Ultraviolet Disinfection, Booster Chlor = distribution booster chlorination, GAC = granular activated carbon, Trident = US Filter buoyant clarifier followed by filtration, NF = Nanofiltration, DAF = Dissolved Air Flotation,

Seasonal Operation Considerations

As the intent of the proposed alternatives are to deal with DBP issues, many of which are seasonal, some of the proposed treatment processes would only operate at times when there is a threat of high DBP formation (3-6 months per year). Processes considered seasonal are the GAC units, the MIEX system at Rutland, and the NF membranes at the CWD and the Vergennes-Panton facility. Other processes proposed to address DBP formation (e.g. enhanced or optimized coagulation) would be used year round as a fully integrated part of the existing plant operation.

For the seasonal processes, chemical usage, power requirements, and labor costs were only considered for three months of the year at most of the sites. Due to more severe DBP issues, six months of operation was carried for Rutland and North Hero. Where a building is necessary for the proposed processes, the heating, ventilation, and cooling costs were weighted based on partial use of the facility. All costs were determined on a square footage basis, with heating and cooling considered for ½ of the year, lighting for ¾ of the year, and ventilation for the entire year.

Replacement of NF membranes and cartridge filters was recommended by the vendor on a 5-year interval, but this would be based on continuous use. Since it was assumed the membranes would be in use only 3 months per year and would be preserved after each use, a longer replacement interval was used in the O&M cost estimates. In addition to the effect of compaction and irreversible fouling resulting from normal use, the age of the membrane and repeated preservations will also reduce the service life. To account for this, it was assumed that the NF membranes would be replaced on a 15-year cycle.

Net Present Worth Analysis

A present worth analysis was used to evaluate and compare the economic impacts of the chlorine and chloramine options at each of the sites over a 20 year period. The present worth of an expenditure related with a given option is today's dollar value (i.e., at the date of implementation of a given option) of all routine annual expenditures (the O&M costs) ascribable to that option over a defined period of time. Assigning present value to O&M costs provides a method of comparing the life cycle costs at each site that incorporate both the capital and O&M expenditures.

The present worth factor, PF, is a function of the assumed interest rate and the period of investment. A real interest rate of 2.7% was used in the analysis, based on the U.S. Government Office of Management and Budget Circular No. A-94 (Updated December, 2009). The present worth factor is calculated in the following manner:

$$PF = \frac{(1+r)^n - 1}{r \times (1+r)^n}$$

where r = real interest rate, and n = number of years.

To determine the present worth of the O&M costs, the present worth factor is used as a multiplier against the O&M costs. The total present worth is then found by adding the present worth of the O&M cost to the capital cost. The present worth of the proposed chlorine and chloramine treatment options are summarized and totaled in Table 5-60. The chloramines option in Table 5-60 includes both chloramines and UV treatment. A present worth cost for implementing only chloramination is compared with the chloramines and UV option in Table 5-61.

Using another approach, capital and O&M costs are presented on an annual basis and shown in Table 5-62 and Table 5-63. This approach assumes that the cost of the project construction and the annual cost of operations would be spread out over 20 years and paid for, in annual increments, through the user base. Annualized capital costs were derived by applying an interest rate of 4.4% on the capital costs based on the nominal rate provided in the U.S. Government Office of Management and Budget Circular No. A-94 (Updated December, 2009). The annualized costs were added to the annual O&M values and normalized to population by showing the costs per person and per household (assuming 3 persons per household) for each community.

In order to better communicate the costs to the various communities, the information in Table 5-60 through Table 5-63 are shown graphically in Figure 5-53 through Figure 5-56. Figure 5-53 demonstrates the present worth of each community relative to its size, Figure 5-54 the total annual cost, Figure 5-55 the annual cost per person, and Figure 5-56 the annual cost per household.

For the free chlorine options, only the treatment alternatives considered most likely to enable compliance with DBP limits (as described in Section 5-3) are included in Figures 5-53 through 5-56. In other words, if an overall compliance strategy at a particular facility included some relatively minor changes (optimizing coagulation for example) plus some major changes such as a new unit process, it was assumed for the purposes of developing the total costs that both minor and major changes would be needed, thus providing the most conservative cost estimates and most conservative treatment approaches. Details of the methodologies used for both the present worth and annual costs are included in Section 5.

For the chloramines options, unit costs generally decrease with increasing populations as economies of scale are experienced. Although this is true to some extent for the chlorine options, there is substantially more variability from community to community. This can be attributed to the type of treatment implemented at each site based on, in part, the severity of the DBP concerns. DAF units were considered for each of Alburgh, Proctor, and Swanton. When combined with the small size of these three communities, the DAF addition equates to a greater per household cost. The CWD supplies the greatest demand, but its costs are elevated by the constrictions on the existing site for expansion, and the need for a large new building and treatment works.

TABLE 5-60
PRESENT WORTH COST SUMMARY (CHLORINE AND CHLORAMINE WITH UV OPTIONS)

Treatment Site	Pop.	Total Present Worth (Chlorine)			Total Present Worth (Chloramine with UV)		
		Capital	Present Worth of O&M	Total Present Worth	Capital	Present Worth of O&M	Total Present Worth
Alburgh	578	$ 1,793,000	$ 2,243,000	$ 4,036,000	$ 159,000	$ 421,000	$ 580,000
Burlington	38,889	4,013,000	2,711,000	6,724,000	675,000	857,000	1,532,000
Bennington	12,000	3,301,000	5,450,000	8,751,000	513,000	729,000	1,242,000
Catamount-Bolton	850	166,000	429,000	595,000	145,000	412,000	557,000
Champlain (NF)	68,923	27,337,000	25,591,000	52,928,000	*N/A - Already practicing chloramine disinfection*		
Champlain (GAC)		21,127,000	32,720,000	53,847,000			
Proctor	2,200	1,689,000	3,583,000	5,272,000	259,000	459,000	718,000
Proctor (Well)		1,926,000	1,295,000	3,221,000			
Grand Isle	1,113	801,000	692,000	1,493,000	194,000	437,000	631,000
North Hero	600	579,000	1,410,000	1,989,000	194,000	437,000	631,000
Readsboro	440	187,000	695,000	882,000	145,000	412,000	557,000
Readsboro (Well)		1,925,000	463,000	2,388,000			
Richford	1,700	660,000	593,000	1,253,000	156,000	423,000	579,000
Rutland City (GAC)	18,500	7,470,000	17,634,000	25,104,000	672,000	779,000	1,451,000
Rutland City (MIEX)		6,075,000	2,441,000	8,516,000			
Rutland City (Well)		5,742,000	1,959,000	7,701,000			
St Johnsbury	3,800	3,285,000	2,791,000	6,076,000	511,000	716,000	1,227,000
Swanton	3,500	4,011,000	4,575,000	8,586,000	308,000	463,000	771,000
Tri-Town	3,500	1,561,000	1,673,000	3,234,000	369,000	525,000	894,000
Vergennes-Panton	5,000	5,306,000	2,932,000	8,238,000	308,000	461,000	769,000
TOTAL[(1)]	**161,593**	**60,764,000**	**57,809,000**	**118,573,000**	**4,608,000**	**7,531,000**	**12,139,000**

[(1)] Chlorine Option totals only include most likely treatment option, NF for the Champlain Water District, MIEX for Rutland City, and GAC (non-well) options for Proctor and Readsboro

TABLE 5-61
PRESENT WORTH COST SUMMARY (CHLORAMINE WITH UV AND CHLORAMINE ONLY OPTIONS)

Treatment Site	Pop.	Total Present Worth (Chloramine with UV)			Total Present Worth (Chloramine Only)		
		Capital	Present Worth of O&M	Total	Capital	Present Worth of O&M	Total
Alburgh	578	$ 159,000	$ 421,000	$ 580,000	$ 58,000	$ 192,000	$ 250,000
Burlington	38,889	675,000	857,000	1,532,000	82,000	325,000	407,000
Bennington	12,000	513,000	729,000	1,242,000	74,000	228,000	302,000
Catamount-Bolton	850	145,000	412,000	557,000	56,000	191,000	247,000
Champlain[1]	68,923	*N/A - Already practicing chloramine disinfection*			641,000	1,041,000	1,682,000
Proctor	2,200	259,000	459,000	718,000	81,000	199,000	280,000
Grand Isle	1,113	194,000	437,000	631,000	60,000	193,000	253,000
North Hero	600	194,000	437,000	631,000	60,000	191,000	251,000
Readsboro	440	145,000	412,000	557,000	56,000	190,000	246,000
Richford	1,700	156,000	423,000	579,000	57,000	193,000	250,000
Rutland City	18,500	672,000	779,000	1,451,000	81,000	245,000	326,000
St Johnsbury	3,800	511,000	716,000	1,227,000	73,000	216,000	289,000
Swanton	3,500	308,000	463,000	771,000	63,000	204,000	267,000
Tri-Town	3,500	369,000	525,000	894,000	68,000	206,000	274,000
Vergennes-Panton	5,000	308,000	461,000	769,000	63,000	203,000	266,000
TOTAL	**161,593**	**4,608,000**	**7,531,000**	**12,139,000**	**1,573,000**	**4,017,000**	**5,590,000**

[1] Chloramine Option Capital and O&M costs based on existing information provided by the Champlain Water District

ANNUALIZED COST SUMMARY (CHLORINE AND CHLORAMINE WITH UV OPTIONS)

Treatment Site	Pop.	Annualized Cost (Chlorine)					Annualized Cost (Chloramine with UV)				
		Capital	O&M	Total	Per Person	Per Household[1]	Capital	O&M	Total	Per Person	Per Household[1]
Alburgh	578	$ 135,000	$ 147,000	$ 282,000	$ 488	$ 1,467	$ 12,000	$ 28,000	$ 40,000	$ 69	$ 206
Burlington	38,889	302,000	177,000	479,000	12	37	51,000	56,000	107,000	3	8
Bennington	12,000	248,000	356,000	604,000	50	151	39,000	48,000	87,000	7	22
Catamount-Bolton	850	12,000	28,000	40,000	47	143	11,000	27,000	38,000	45	134
Champlain (NF)	68,923	2,058,000	1,673,000	3,731,000	54	162	*N/A - Already practicing chloramine disinfection*				
Champlain (GAC)		1,590,000	2,139,000	3,729,000	54	162					
Proctor	2,200	127,000	234,000	361,000	164	493	19,000	30,000	49,000	22	68
Proctor (Well)		145,000	85,000	230,000	105	313					
Grand Isle	1,113	60,000	45,000	105,000	94	284	15,000	29,000	44,000	40	116
North Hero	600	44,000	92,000	136,000	227	679	15,000	29,000	44,000	73	216
Readsboro	440	14,000	45,000	59,000	134	408	11,000	27,000	38,000	86	259
Readsboro (Well)		145,000	30,000	175,000	398	1,200					
Richford	1,700	50,000	39,000	89,000	52	156	12,000	28,000	40,000	24	70
Rutland City (GAC)	18,500	562,000	1,153,000	1,715,000	93	278	51,000	51,000	102,000	6	16
Rutland City (MIEX)		457,000	160,000	617,000	33	100					
Rutland City (Well)		432,000	128,000	560,000	30	91					
St Johnsbury	3,800	247,000	182,000	429,000	113	339	38,000	47,000	85,000	22	67
Swanton	3,500	302,000	299,000	601,000	172	515	23,000	30,000	53,000	15	46
Tri-Town	3,500	117,000	109,000	226,000	65	195	28,000	34,000	62,000	18	53
Vergennes-Panton	5,000	399,000	192,000	591,000	118	355	23,000	30,000	53,000	11	32
TOTAL[2]	**161,593**	**4,572,000**	**3,778,000**	**8,350,000**	**52**	**155**	**348,000**	**494,000**	**842,000**	**5**	**16**

[1] Household represents three persons

[2] Chlorine Option totals only include most likely treatment option, NF for the Champlain Water District, MIEX for Rutland City, and GAC (non-well) options for Proctor and Readsboro

ANNUALIZED COST SUMMARY (CHLORAMINE WITH UV AND CHLORAMINE ONLY OPTIONS)

Treatment Site	Pop.	Annualized Cost (Chloramine with UV)					Annualized Cost (Chloramine Only)				
		Capital	O&M	Total	Per Person	Per Household[1]	Capital	O&M	Total	Per Person	Per Household[1]
Alburgh	578	$ 12,000	$ 28,000	$ 40,000	$ 69	$ 206	$ 4,000	$ 13,000	$ 17,000	$ 29	$ 88
Burlington	38,889	51,000	56,000	107,000	3	8	6,000	21,000	27,000	1	2
Bennington	12,000	39,000	48,000	87,000	7	22	6,000	15,000	21,000	2	5
Catamount-Bolton	850	11,000	27,000	38,000	45	134	4,000	12,000	16,000	19	59
Champlain[2]	68,923	*N/A - Already practicing chloramine disinfection*					48,249	68,045	116,294	2	5
Proctor	2,200	19,000	30,000	49,000	22	68	6,000	13,000	19,000	9	26
Grand Isle	1,113	15,000	29,000	44,000	40	116	5,000	13,000	18,000	16	46
North Hero	600	15,000	29,000	44,000	73	216	5,000	12,000	17,000	28	85
Readsboro	440	11,000	27,000	38,000	86	259	4,000	12,000	16,000	36	114
Richford	1,700	12,000	28,000	40,000	24	70	4,000	13,000	17,000	10	30
Rutland City	18,500	51,000	51,000	102,000	6	16	6,000	16,000	22,000	1	4
St Johnsbury	3,800	38,000	47,000	85,000	22	67	5,000	14,000	19,000	5	15
Swanton	3,500	23,000	30,000	53,000	15	46	5,000	13,000	18,000	5	16
Tri-Town	3,500	28,000	34,000	62,000	18	53	5,000	13,000	18,000	5	16
Vergennes-Panton	5,000	23,000	30,000	53,000	11	32	5,000	13,000	18,000	4	11
TOTAL	**161,593**	**348,000**	**494,000**	**842,000**	**5**	**16**	**118,000**	**261,000**	**379,000**	**2**	**7**

[1] Household represents three persons

[2] Chloramine Option Capital and O&M costs based on existing information provided by the Champlain Water District

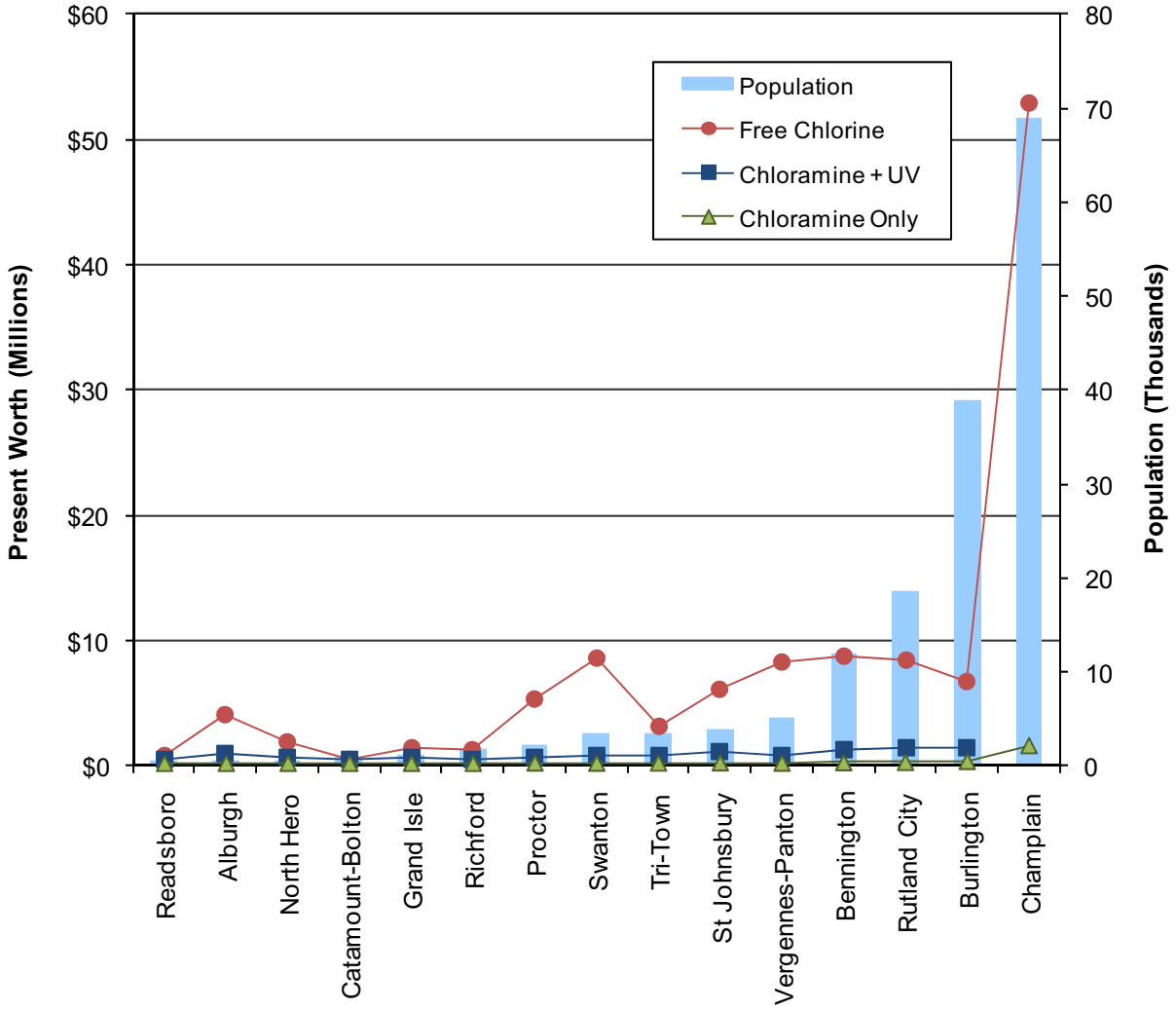


FIGURE 5-53
PRESENT WORTH OF MOST LIKELY TREATMENT OPTIONS

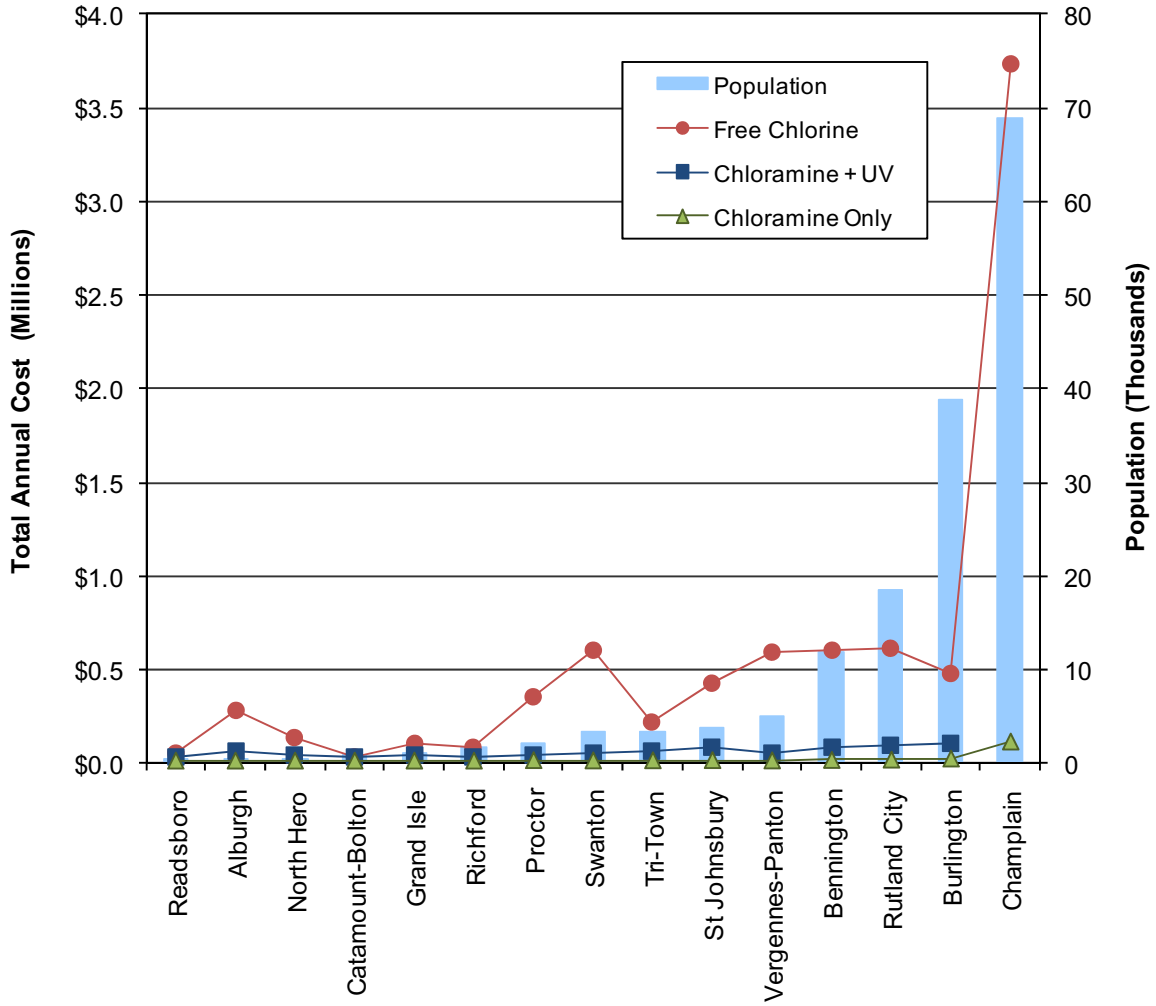


FIGURE 5-54
TOTAL ANNUAL COST OF MOST LIKELY TREATMENT OPTIONS

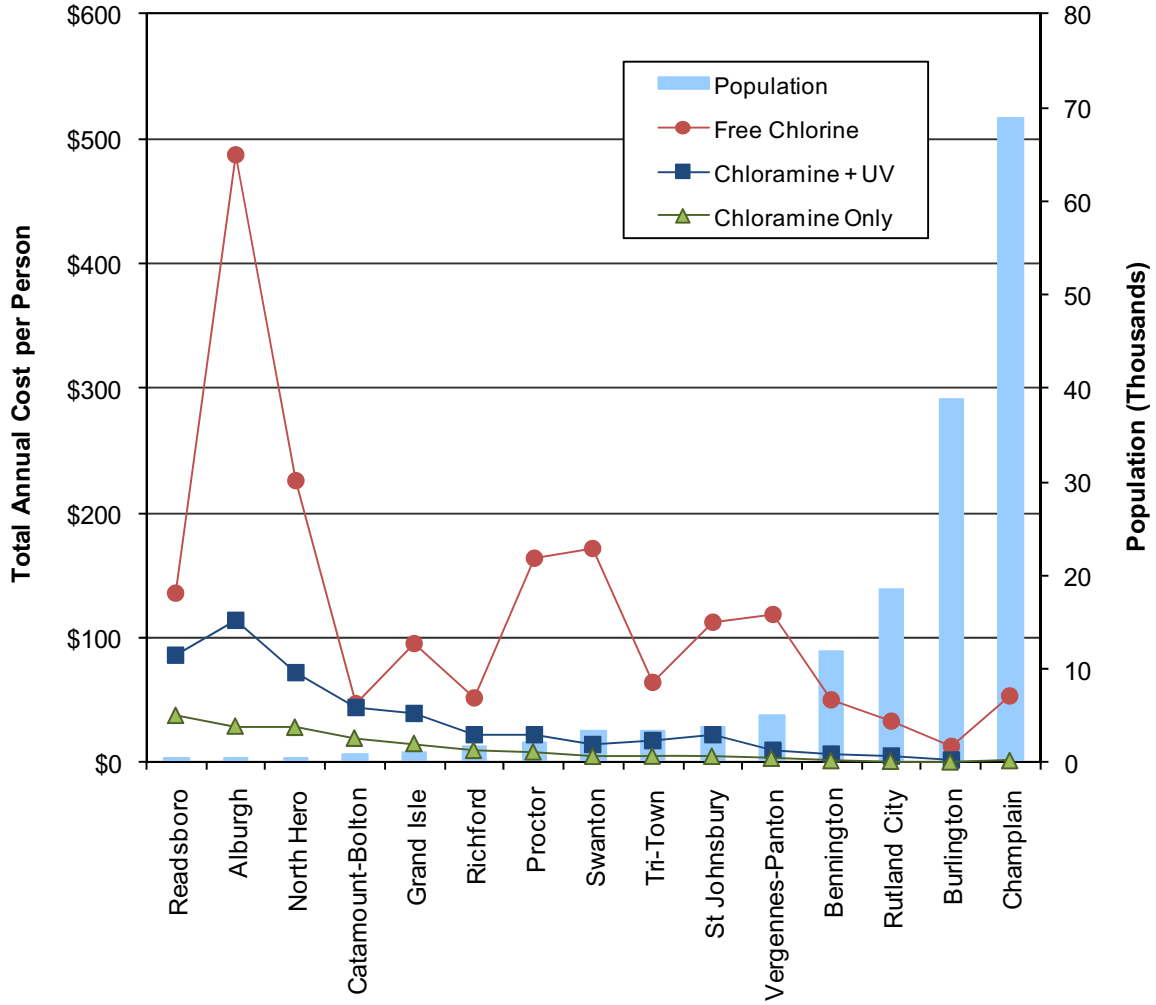


FIGURE 5-55
TOTAL ANNUAL COST PER PERSON OF MOST LIKELY TREATMENT OPTIONS

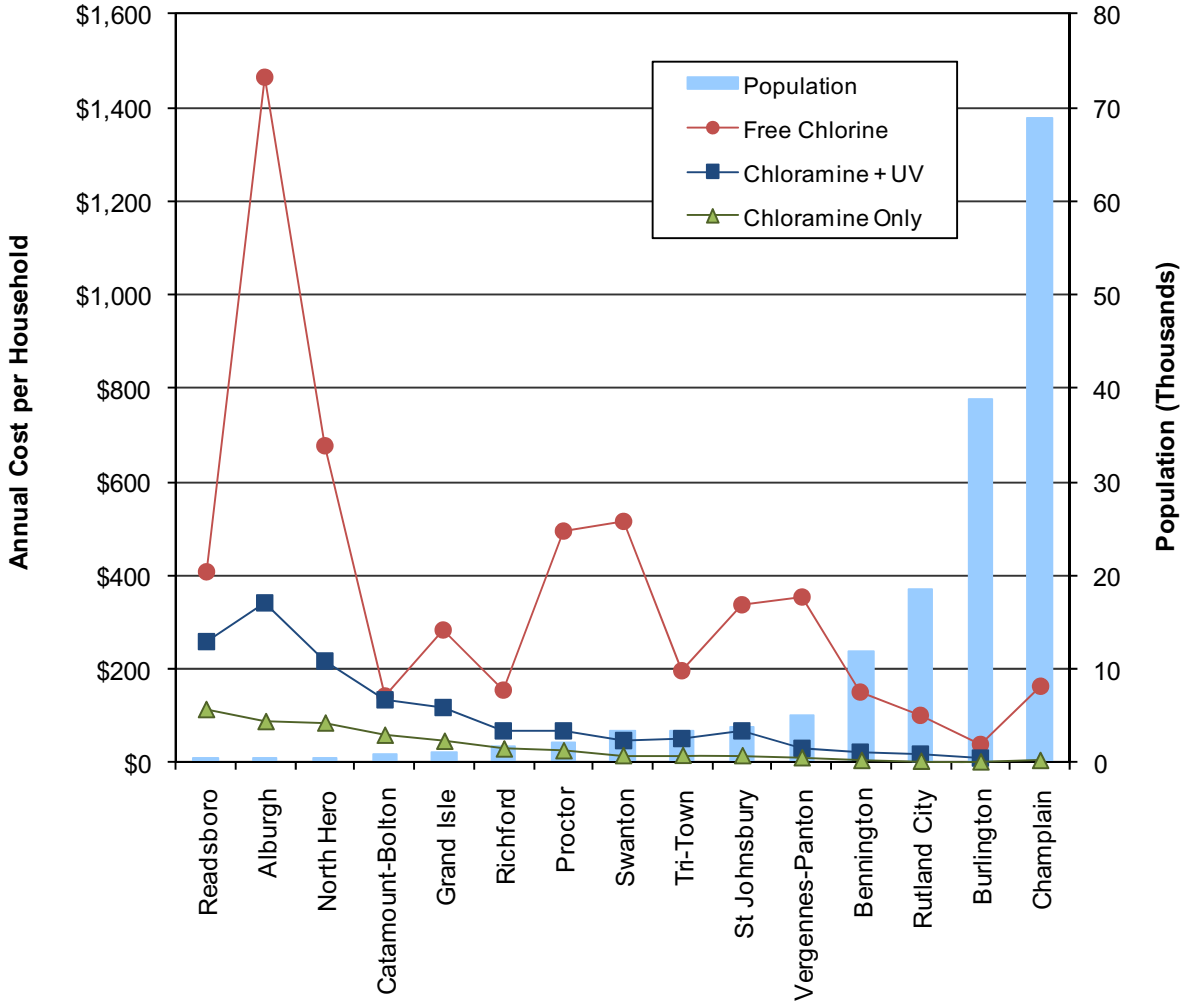


FIGURE 5-56
TOTAL ANNUAL COST PER HOUSEHOLD OF MOST LIKELY TREATMENT OPTIONS

In order to evaluate the impact of each proposed process component, the present value was separated for each unit process that made up the most likely treatment option at each site. Table 5-64 summarizes the total present worth and contribution of individual processes for both options. Costs for the alternate treatment options, including well development and MIEX, are not included in Table 5-64 as their cost information is already broken out in Table 5-60 through Table 5-63. As its intent is to show the cost make-up of the various treatment options, contingencies and engineering costs are not included in Table 5-64. Only the direct cost sub totals are listed.

Using Alburgh as an example in Table 5-64, costs for both enhanced coagulation and DAF are listed under the chlorine option, as well as chloramines and UV for the chloramines with UV option, as they together make up the most likely treatment for each option developed in Section 5-1. It is evident that the majority of the estimated cost is attributed to the enhanced coagulation for the chlorine option, where they are more evenly split between UV and chloramines costs in the chloramines with UV option. Table 5-64 also identifies the impact of cost should one of the components of the treatment option be removed. More detailed O&M information is included in Appendix 11.

Summary

A few important conclusions can be drawn from this study.

1. Secondary disinfection with chloramine to achieve DBP compliance is the least expensive approach on a total present worth basis considering both capital and O&M costs.
2. UV, if required to achieve DBP compliance while using chloramine for secondary disinfection, increases the both the capital and O&M cost of a chloramine approach.
3. Secondary disinfection using chlorine to achieve DBP compliance is a significantly more expensive approach.
4. The total capital cost, annualized over a 20 year basis, of implementing a chlorine secondary disinfection approach for all of the systems evaluated would be approximately $4.6 MM compared to only $348,000 for a chloramine approach with UV, or compared to $118,000 for a chloramine approach without UV.

5. Considering capital cost and annual operating costs, the total per household cost of implementing a chlorine secondary disinfection approach averages $155/yr but ranges widely according to system size with small systems having the larger cost impact; while a chloramine approach with UV would have an average per household cost of approximately $16/yr; and an even lower average per household cost of about $7/yr, with a chloramine approach without UV. For chloramine approaches, as for chlorine approaches, there is a greater cost impact on smaller systems.

Please note that AECOM's scope for this project only covers evaluation of alternatives for compliance with the requirements of the Stage 2 DPBR, using either chlorine or chloramine for secondary disinfection, and to provide cost estimates for the DEC and Legislature to consider in their decision making process. Issues of possible health concerns for chlorine or chloramine were not AECOM's purview. Further, AECOM, makes no recommendation as to which disinfectant approach to use.

TABLE 5-64
PRESENT WORTH BREAKDOWN SUMMARY

Site	Chlorine						Chloramines with UV		
Process Unit	Enhanced Coagulation	UV	GAC	Other Process(1)	NF	Sub Total(2)	UV	Chloramines	Sub Total(2)
Alburgh				*DAF*					
Capital	$ 32,700			$ 969,900		$ 1,002,600	$ 54,100	$ 31,000	$ 85,100
O&M	$ 12,720			$ 133,900		$ 146,620	$ 14,970	$ 12,500	$ 27,470
O&M Present Worth	$ 195,000			$ 2,049,000		$ 2,244,000	$ 229,000	$ 192,000	$ 421,000
Net Present Worth	**$ 227,700**			**$ 3,018,900**		**$ 3,246,600**	**$ 283,100**	**$ 223,000**	**$ 506,100**
Burlington				*DAF*					
Capital	$ 157,400			$ 2,031,400		$ 2,188,800	$ 317,500	$ 43,700	$ 361,200
O&M	$ 114,430			$ 62,770		$ 177,200	$ 34,870	$ 21,180	$ 56,050
O&M Present Worth	$ 1,751,000			$ 961,000		$ 2,712,000	$ 534,000	$ 325,000	$ 859,000
Net Present Worth	**$ 1,908,400**			**$ 2,992,400**		**$ 4,900,800**	**$ 851,500**	**$ 368,700**	**$ 1,220,200**
Bennington									
Capital	$ 106,100		$ 1,663,000			$ 1,769,100	$ 234,100	$ 39,900	$ 274,000
O&M	$ 66,010		$ 290,200			$ 356,210	$ 32,740	$ 14,880	$ 47,620
O&M Present Worth	$ 1,010,000		$ 4,440,000			$ 5,450,000	$ 501,000	$ 228,000	$ 729,000
Net Present Worth	**$ 1,116,100**		**$ 6,103,000**			**$ 7,219,100**	**$ 735,100**	**$ 267,900**	**$ 1,003,000**
Catamount									
Capital	$ 44,300	$ 48,000				$ 92,300	$ 48,500	$ 29,500	$ 78,000
O&M	$ 13,380	$ 14,580				$ 27,960	$ 14,460	$ 12,420	$ 26,880
O&M Present Worth	$ 205,000	$ 223,000				$ 428,000	$ 222,000	$ 191,000	$ 413,000
Net Present Worth	**$ 249,300**	**$ 271,000**				**$ 520,300**	**$ 270,500**	**$ 220,500**	**$ 491,000**
Champlain (NF)				*Bstr Chlor.*					
Capital				$ 491,700	$ 14,028,000	$ 14,519,700			
O&M				$ 118,850	$ 1,553,880	$ 1,672,730			
O&M Present Worth				$ 1,819,000	$ 23,773,000	$ 25,592,000			
Net Present Worth				**$ 2,310,700**	**$ 37,801,000**	**$ 40,111,700**			
Champlain (GAC)				*Bstr Chlor.*					
Capital			$ 10,701,000	$ 491,700		$ 11,192,700			
O&M			$ 2,048,610	$ 118,850		$ 2,167,460			
O&M Present Worth			$ 31,341,000	$ 1,819,000		$ 33,160,000			
Net Present Worth			**$ 42,042,000**	**$ 2,310,700**		**$ 44,352,700**			

TABLE 5-64
PRESENT WORTH BREAKDOWN SUMMARY

Site	Chlorine						Chloramines with UV		
Process Unit	Enhanced Coagulation	UV	GAC	Other Process(1)	NF	Sub Total(2)	UV	Chloramines	Sub Total(2)
Proctor				*DAF*					
Capital	$ 32,500	$ 124,000		$ 790,600		$ 947,100	$ 95,000	$ 43,200	$ 138,200
O&M	$ 8,490	$ 16,970		$ 208,750		$ 234,210	$ 17,000	$ 13,010	$ 30,010
O&M Present Worth	$ 130,000	$ 260,000		$ 3,194,000		$ 3,584,000	$ 261,000	$ 199,000	$ 460,000
Net Present Worth	**$ 162,500**	**$ 384,000**		**$ 3,984,600**		**$ 4,531,100**	**$ 356,000**	**$ 242,200**	**$ 598,200**
Grand Isle									
Capital			$ 430,000			$ 430,000	$ 72,800	$ 31,700	$ 104,500
O&M			$ 45,170			$ 45,170	$ 15,980	$ 12,560	$ 28,540
O&M Present Worth			$ 692,000			$ 692,000	$ 245,000	$ 193,000	$ 438,000
Net Present Worth			**$ 1,122,000**			**$ 1,122,000**	**$ 317,800**	**$ 224,700**	**$ 542,500**
North Hero									
Capital	$ 30,200		$ 286,000			$ 316,200	$ 72,800	$ 31,700	$ 104,500
O&M	$ 13,740		$ 78,330			$ 92,070	$ 15,980	$ 12,470	$ 28,450
O&M Present Worth	$ 211,000		$ 1,199,000			$ 1,410,000	$ 245,000	$ 191,000	$ 436,000
Net Present Worth	**$ 241,200**		**$ 1,485,000**			**$ 1,726,200**	**$ 317,800**	**$ 222,700**	**$ 540,500**
Readsboro				*7 Day Oper.*					
Capital			$ 104,000			$ 104,000	$ 48,300	$ 29,300	$ 77,600
O&M	$ 4,000		$ 5,370	$ 36,010		$ 45,380	$ 14,460	$ 12,420	$ 26,880
O&M Present Worth	$ 62,000		$ 83,000	$ 551,000		$ 696,000	$ 222,000	$ 190,000	$ 412,000
Net Present Worth	**$ 62,000**		**$ 187,000**	**$ 551,000**		**$ 800,000**	**$ 270,300**	**$ 219,300**	**$ 489,600**
Richford									
Capital			$ 355,000			$ 355,000	$ 53,200	$ 30,200	$ 83,400
O&M			$ 38,680			$ 38,680	$ 14,970	$ 12,610	$ 27,580
O&M Present Worth			$ 592,000			$ 592,000	$ 229,000	$ 193,000	$ 422,000
Net Present Worth			**$ 947,000**			**$ 947,000**	**$ 282,200**	**$ 223,200**	**$ 505,400**
Rutland City (GAC)									
Capital			$ 3,994,000			$ 3,994,000	$ 316,600	$ 42,900	$ 359,500
O&M			$ 1,152,570			$ 1,152,570	$ 34,890	$ 15,980	$ 50,870
O&M Present Worth			$ 17,633,000			$ 17,633,000	$ 534,000	$ 245,000	$ 779,000
Net Present Worth			**$ 21,627,000**			**$ 21,627,000**	**$ 850,600**	**$ 287,900**	**$ 1,138,500**

TABLE 5-64
PRESENT WORTH BREAKDOWN SUMMARY

Site	Chlorine						Chloramines with UV		
Process Unit	Enhanced Coagulation	UV	GAC	Other Process[1]	NF	Sub Total[2]	UV	Chloramines	Sub Total[2]
St Johnsbury									
Capital			$ 1,758,000			$ 1,758,000	$ 233,200	$ 39,100	$ 272,300
O&M			$ 182,450			$ 182,450	$ 32,740	$ 14,090	$ 46,830
O&M Present Worth			$ 2,792,000			$ 2,792,000	$ 501,000	$ 216,000	$ 717,000
Net Present Worth			**$ 4,550,000**			**$ 4,550,000**	**$ 734,200**	**$ 255,100**	**$ 989,300**
Swanton				*DAF*					
Capital	$ 6,800		$ 920,000	$ 1,262,900		$ 2,189,700	$ 129,700	$ 34,400	$ 164,100
O&M	$ 3,250		$ 82,690	$ 213,020		$ 298,960	$ 16,890	$ 13,310	$ 30,200
O&M Present Worth	$ 50,000		$ 1,266,000	$ 3,259,000		$ 4,575,000	$ 259,000	$ 204,000	$ 463,000
Net Present Worth	**$ 56,800**		**$ 2,186,000**	**$ 4,521,900**		**$ 6,764,700**	**$ 388,700**	**$ 238,400**	**$ 627,100**
Tri-Town									
Capital	$ 6,800		$ 833,000			$ 839,800	$ 161,200	$ 35,900	$ 197,100
O&M	$ 23,350		$ 85,940			$ 109,290	$ 20,850	$ 13,460	$ 34,310
O&M Present Worth	$ 358,000		$ 1,315,000			$ 1,673,000	$ 320,000	$ 206,000	$ 526,000
Net Present Worth	**$ 364,800**		**$ 2,148,000**			**$ 2,512,800**	**$ 481,200**	**$ 241,900**	**$ 723,100**
Vergennes				*Trident*					
Capital	$ 6,800			$ 1,140,200	$ 1,705,000	$ 2,852,000	$ 129,700	$ 34,400	$ 164,100
O&M	$ 10,510			$ 102,080	$ 78,970	$ 191,560	$ 16,890	$ 13,230	$ 30,120
O&M Present Worth	$ 161,000			$ 1,562,000	$ 1,209,000	$ 2,932,000	$ 259,000	$ 203,000	$ 462,000
Net Present Worth	**$ 167,800**			**$ 2,702,200**	**$ 2,914,000**	**$ 5,784,000**	**$ 388,700**	**$ 237,400**	**$ 626,100**

[1] *Other Processes* include new treatment processes and operational changes
[2] Sub Total does not include contingencies and engineering costs

SECTION 6: SUMMARY

6.1. SUMMARY

A kickoff workshop was held in Waterbury, VT which introduced the project to all concerned stakeholders. After the kickoff workshop, each of the 15 utilities provided operational data, water quality data, and record drawings for their WTF. The DEC also provided similar information from their records. Some utilities mailed samples of metal service line pipe for scale analysis.

AECOM reviewed the current regulations as well as treatment technologies to meet those regulations. This information is provided in Section 3 and Section 4. The review demonstrated a multitude of options available in each utility's toolbox for addressing DBP reduction. Based on the data provided, AECOM developed preliminary strategies for each utility in order to meet the current Stage 1 DBPR as well as the upcoming Stage 2 DBPR. The strategies were broken down into two categories: those implementing chlorine as the distribution residual, and those which use chloramine as the distribution residual. From there, the results were further broken down into treatment and distribution practices.

AECOM met with each utility, as part of the utility workshop, and verified the data as well discussed the preliminary strategies. The strategies were revised for the draft report and presented in Section 5. As part of process, AECOM narrowed down the options to those which would most likely be successful in consistently reducing the DBPs to below the regulated MCLs. Other alternatives were placed under the section listed as "Other DBP Reduction Alternatives." These alternatives were either close to the cutoff (e.g. had a high likelihood for success in DBP reduction), the WTF was easily setup for the recommendation, and/or were relatively inexpensive to employ and allowed for system optimization.

A 20 year life cycle cost was assembled for each strategy and presented in Section 5 as well as annualized cost. The life cycle analysis broke down the costs into capital costs as well as operational costs. The 20 year life cycle cost and annual cost were also normalized to the population served for each utility. AECOM presented the costs up to the first major treatment capital costs incurred such as GAC absorption. In reality, it may be able for the utility to meet the regulations with more careful operation of the treatment process (e.g. targeting TOC removal with enhanced coagulation), as well as distribution operation (e.g. looping the distribution system and flushing).