Wastewater Utility Plan

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Contents

SECTION 1 ..... **1-1**

EXECUTIVE SUMMARY **1-1**

OBJECTIVES FOR WASTEWATER UTILITY PLAN 1-1
BASIS OF PLANNING 1-3
IMPLEMENTATION SCHEDULE AND TIMING ISSUES 1-3
APPLICABLE COMPLIANCE SCHEDULES OR CONSENT DECREES 1-3
WASTEWATER FLOW AND LOAD PROJECTIONS 1-3
NWRF PROCESS SYSTEM 1-4
SWRF PROCESS SYSTEM 1-5
COLLECTION SYSTEM 1-5
ALTERNATIVES ANALYSIS 1-6
MANAGEMENT AND FINANCIAL PLANS 1-8

SECTION 2 ..... **2-1**

INTRODUCTION **2-1**

BACKGROUND AND HISTORY 2-1
OBJECTIVES FOR WASTEWATER UTILITY PLAN 2-3
FACILITIES PLAN SUMMARY 2-3
IMPLEMENTATION SCHEDULE AND TIMING ISSUES 2-3
SUMMARY OF DIFFERENCES BETWEEN WUP AND NFRQWPA / DRCOG OUTLINES 2-4

SECTION 3 ..... **3-1**

GENERAL PLANNING **3-1**

CONSOLIDATION OF FACILITIES 3-1
WASTEWATER REUSE 3-2
Water Rights and Water Supply Sources *3-3*
Non-potable and Raw Water Supplies *3-4*
Reuse of Windy Gap Return Flows *3-5*
Transmission and Treatment Systems *3-5*
Non-potable Transmission Systems *3-6*
APPLICABLE COMPLIANCE SCHEDULES OR CONSENT DECREES 3-6
ENVIRONMENTAL COMPONENTS EVALUATION 3-6
Ecoregions *3-6*
Climate *3-11*
Topography *3-12*
Underlying Geology *3-12*
Coal Mining *3-12*
Local Soils *3-13*
South Water Reclamation Facility 3-13
North Water Reclamation Facility 3-14
Site Specific Geotechnical Data *3-14*
South Water Reclamation Facility 3-14
North Water Reclamation Facility 3-15
Groundwater *3-15*
Wetlands *3-16*
Floodplain *3-16*
Wildlife *3-18*
SUMMARY OF ENVIRONMENTAL INFORMATION OR SPECIAL STUDIES 3-18

SECTION 4 ..... **4-1**

WASTEWATER CHARACTERISTICS **4-1**

SERVICE AREA DESIGNATIONS 4-1
WUSA and CWP Planning Areas *4-2*
Water and Wastewater Service 4-3
Water Rights and Wastewater Reuse 4-4
Summary 4-5
Historic Population Growth *4-5*
Future Population Growth *4-6*
WASTEWATER FLOWS 4-8
Selection of Peak Hour Factor *4-9*
CHARACTER OF INFLUENT 4-11
Temperature *4-11*
Biological Oxygen Demand *4-12*
Total Suspended Solids *4-13*
Ammonia Nitrogen *4-15*
Total Kjeldahl Nitrogen *4-16*
Fats, Oils, and Greases *4-16*
Wastewater Parameter Ratio Analysis *4-18*
Per Capita Loading *4-19*
PEAKING FACTORS 4-20
WASTEWATER FLOW AND LOAD PROJECTIONS 4-21
Flow and Load Projections from Historic Generation Rates 4-21
Flow and Load Projections using Recommended State Design Criteria 4-22
Flow and Load Projections Selected for Planning Purposes 4-23
Flow Apportionment Between SWRF and NWRF 4-24
INDUSTRIAL PRETREATMENT PROGRAM 4-25
INFLOW AND INFILTRATION ANALYSIS 4-25

SECTION 5 ..... **5-1**

EVALUATION OF WASTEWATER TREATMENT FACILITIES **5-1**

NWRF PROCESS SYSTEM 5-1
NWRF INFRASTRUCTURE SIZING AND STAGING 5-1
NWRF Headworks *5-2*
NWRF Secondary Treatment *5-3*
NWRF Disinfection *5-6*
NWRF Solids Handling *5-6*
NWRF LOCATION AND SITING 5-7
NWRF BIOSOLIDS HANDLING 5-7
NWRF SCHEMATIC OF TREATMENT WORKS 5-8
NWRF ODOR CONTROL CONSIDERATIONS 5-8
NWRF AIR QUALITY PERMIT 5-8
NWRF STORMWATER MANAGEMENT PLAN 5-8
NWRF SITE CHARACTERIZATION REPORT 5-8
SWRF PROCESS SYSTEM 5-9
SWRF INFRASTRUCTURE SIZING AND STAGING 5-9
SWRF Headworks *5-9*
SWRF Secondary Treatment Process *5-13*
Aeration Basins 5-16
Aeration System 5-16
Dissolved Oxygen Probe 5-17
Diffusers. 5-17
Secondary Clarifiers 5-17
Splitter Box 5-18
Collection Mechanisms 5-18
Return and Waste Activated Sludge Pumps 5-18

SWRF Disinfection *5-19*
SWRF Solids Handling *5-20*
SWRF Biosolids Quality 5-22
SWRF Location and Siting *5-23*
SWRF BIOSOLIDS HANDLING 5-24
SWRF SCHEMATIC OF TREATMENT WORKS 5-24
SWRF ODOR CONTROL CONSIDERATIONS 5-24
SWRF AIR QUALITY PERMIT 5-24
SWRF STORMWATER MANAGEMENT PLAN 5-24
SWRF SITE CHARACTERIZATION REPORT SUMMARY 5-24

SECTION 6 6-1

COLLECTION SYSTEM 6-1

DESCRIPTION OF EXISTING COLLECTION SYSTEM 6-1
Drainage Basins *6-1*
Manhole Naming Convention *6-2*
Major Interceptors *6-2*
Diversion Structures *6-3*
MODEL CONSTRUCTION 6-4
Geometric Network *6-4*
Wastewater Flow Allocation *6-5*
Peak Hour Flows *6--7*
Diversion Structure Modeling *6-8*
Diurnal Curve for EPS Modeling and Peak Hour Factors *6-8*
MODEL EVALUATION RESULTS 6-9
Evaluation Criteria *6-9*
Future Interceptors, Extensions, and Lift Stations *6-10*
Future System Evaluation Results *6-12*
FUTURE WASTEWATER COLLECTION SYSTEM RECOMMENDATIONS 6-13
Major Interceptors *6-15*
Highway 52 Interceptor Alternative 1 6-15
Interstate Interceptor Alternative 1 6-15
Highway 52 and Interstate Interceptors Alternative 2 6-15
Lift Stations and Force Mains *6-15*
Highway 52 Lift Station and 24-inch Force Main 6-15
Interstate Lift Station and 4-inch Force Main 6-16
Highway 52 and Interstate Lift Stations Alternative 2 6-16
Capacity Improvements *6-17*
Arapahoe Ridge 6-17
West Side 6-17
Old Town 6-17
Austin Industrial 6-18
Construction Timeline for Improvements *6-18*
Interstate Interceptor 6-18
Highway 52 Interceptor 6-18
FLOW MONITORING 6-18
PREVENTATIVE MAINTENANCE 6-19
SUMMARY 6-20

SECTION 7 7-1

WATER QUALITY CHARACTERIZATION 7-1

WATER QUALITY MANAGEMENT AGENCIES 7-1
North Front Range Water Quality Planning Association *7-1*
Denver Regional Council of Governments *7-2*
WATER QUALITY OF RECEIVING WATERS 7-2
Water Quality Standards *7-3*

Ambient Water Quality *7-8*
TOTAL MAXIMUM DAILY LOADS AND/OR WASTE LOAD ALLOCATIONS 7-9
WATERSHED ISSUES 7-10
REQUIRED LEVEL OF TREATMENT 7-11
Current Permit Limits *7-11*
Future Permit Limits *7-14*
WATERSHED CHARACTERISTICS AND ISSUES 7-14
Aquatic Life and Habitat *7-15*
Existing Facilities *7-16*
Potable Water Wells *7-16*
Water Diversions *7-17*
Impaired Waters *7-18*

SECTION 8 8-1

ALTERNATIVES ANALYSIS 8-1

INFLUENT FLOWS AND LOADS 8-1
COST ESTIMATE METHODOLOGY 8-2
NORTH WATER RECLAMATION FACILITY 8-3
SOUTH WATER RECLAMATION FACILITY 8-5
SWRF ALTERNATIVES ANALYSIS 8-6
No Action Alternative *8-8*
Extended Aeration Activated Sludge *8-9*
Membrane Bioreactor *8-12*
Design Criteria for SWRF MBR System 8-14
Estimated Construction Costs for SWRF MBR System 8-16
Integrated Fixed Film Activated Sludge *8-18*
Design Criteria for SWRF IFAS System 8-19
Estimated Construction Costs for SWRF IFAS System 8-20
IFAS Future Direction 8-22
Process Technology Selection *8-22*
TERTIARY FILTRATION 8-23
Hydrotech Discfilter – Kruger 8-24
Ultrafiltration – Miller-Leaman 8-26
Volcano Continuous Downflow Filter – Lighthouse Filters 8-28
Recommendation 8-29
BIOLOGICAL PHOSPHORUS REMOVAL 8-29
CHEMICAL PHOSPHORUS REMOVAL 8-30
REUSE WATER STORAGE 8-31
SOLIDS THICKENING 8-32
Rotary Drum Thickener 8-33
LEVELS OF TREATMENT 8-34
PUBLIC PARTICIPATION 8-35

SECTION 9 9-1

MANAGEMENT AND FINANCIAL PLANS 9-1

MANAGEMENT STRUCTURE AND AGREEMENTS 9-1
PRELIMINARY OPERATING PLAN 9-1
WASTEWATER MANAGEMENT PLAN 9-2
FINANCIAL MANAGEMENT PLAN 9-2
TWENTY-YEAR PLANNING PERIOD FINANCIALS 9-3
INTEREST IN STATE FINANCIAL ASSISTANCE 9-5
USER CHARGE STUDY SUMMARY 9-6
RATE COMPARISON 9-7

WORKS CITED REF-1

TABLES

SECTION 1 ..... **1-1**

EXECUTIVE SUMMARY **1-1**

TABLE 1-1 FUTURE POPULATION GROWTH 1-3
TABLE 1-2 SELECTED FUTURE FLOWS FOR PLANNING PURPOSES 1-4
TABLE 1-3 FUTURE LOADS FOR TOWN OF ERIE FROM HISTORIC GENERATION RATES 1-4

SECTION 2 ..... **2-1**

INTRODUCTION **2-1**

SECTION 3 ..... **2-1**

GENERAL PLANNING **3-1**

TABLE 3-1 EXISTING WATER RIGHTS, APRIL 27, 2010 3-4
TABLE 3-2 SUGGESTED WATER QUALITY CRITERIA GOALS FOR ECOREGION 25 3-8
TABLE 3-3 COLORADO'S NUTRIENT WATER QUALITY STANDARDS 3-9
TABLE 3-4 TECHNOLOGICAL LIMITS FOR BIOLOGICAL NUTRIENT REMOVAL 3-9
TABLE 3-5 REGULATION 85 NUTRIENTS MANAGEMENT CONTROL DISCHARGE LIMITS 3-10
TABLE 3-6 CLIMATE DATA FOR THE TOWN OF ERIE, YEARS 1893 TO 2004 3-11
TABLE 3-7 LOCAL SOIL TYPES AT SWRF SITE BY AASHTO GROUP CLASSIFICATION 3-14
TABLE 3-8 LOCAL SOIL TYPES AT NWRF SITE BY AASHTO GROUP CLASSIFICATION 3-14

SECTION 4 ..... **4-1**

WASTEWATER CHARACTERISTICS **4-1**

TABLE 4-1 TOWN OF ERIE SOUTH WATER RECLAMATION FACILITY LEGAL DESCRIPTION AND CAPACITY 4-1
TABLE 4-2 TOWN OF ERIE NORTH WATER RECLAMATION FACILITY LEGAL DESCRIPTION AND CAPACITY 4-2
TABLE 4-3 HISTORIC POPULATION GROWTH 4-6
TABLE 4-4 NEW TAPS ISSUED FOR TOWN OF ERIE, 2002 THROUGH 2009 4-6
TABLE 4-5 FUTURE POPULATION GROWTH 4-7
TABLE 4-6 MAXIMUM DWELLING UNITS PER ACRE 4-8
TABLE 4-7 MONTHLY AVERAGE INFLUENT FLOW, MGD 4-9
TABLE 4-8 MINIMUM AND MAXIMUM DAILY INFLUENT FLOW, MGD 4-9
TABLE 4-9 RANGES OF HOURLY PEAKING FACTORS 4-10
TABLE 4-10 ESTIMATED FUTURE WASTEWATER GENERATION RATES FOR OVERALL SYSTEM 4-11
TABLE 4-11 ESTIMATED PEAK HOUR PEAKING FACTORS AND PEAK HOUR FLOWS BY FACILITY 4-11
TABLE 4-12 MONTHLY AVERAGE EFFLUENT TEMPERATURE, DEGREES CELSIUS 4-12
TABLE 4-13 MONTHLY AVERAGE INFLUENT BOD, MG/L 4-13
TABLE 4-14 MONTHLY AVERAGE INFLUENT BOD, PPD 4-13
TABLE 4-15 MONTHLY AVERAGE INFLUENT TSS, MG/L 4-14
TABLE 4-16 MONTHLY AVERAGE INFLUENT TSS, PPD 4-14
TABLE 4-17 MONTHLY AVERAGE INFLUENT AMMONIA, MG/L AS N 4-15
TABLE 4-18 MONTHLY AVERAGE INFLUENT AMMONIA, PPD 4-16
TABLE 4-19 WASTEWATER PARAMETER RATIOS FOR TOWN OF ERIE 4-19
TABLE 4-20 PER CAPITA GENERATION RATES FOR TOWN OF ERIE, 2001-2008 4-19
TABLE 4-21 HISTORICAL INFLUENT PEAKING FACTORS FOR THE TOWN OF ERIE SWRF, 2004 TO 2009 4-20
TABLE 4-22 ESTIMATED FUTURE WASTEWATER FLOWS FROM HISTORIC GENERATION RATES 4-21
TABLE 4-23 ESTIMATED PEAK HOUR PEAKING FACTORS AND PEAK HOUR FLOWS BY FACILITY 4-22
TABLE 4-24 FUTURE LOADS FOR TOWN OF ERIE FROM HISTORIC GENERATION RATES 4-22
TABLE 4-25 COMPARISON OF RECOMMENDED YEAR 2025 FLOW AND LOAD PROJECTIONS 4-23
TABLE 4-26 SELECTED FUTURE FLOWS FOR PLANNING PURPOSES 4-24
TABLE 4-27 PROJECTED FUTURE AVERAGE AND MAXIMUM MONTH FLOWS 4-24
TABLE 4-28 COMPARE WINTER WATER USE BILLED TO WASTEWATER FLOWS RECEIVED AT SWRF 4-26

SECTION 5 ..... **5-1**

EVALUATION OF WASTEWATER TREATMENT FACILITIES **5-1**

TABLE 5-1 DESIGN CRITERIA FOR NWRF 5-1
TABLE 5-2 NWRF HEADWORKS DESIGN CRITERIA 5-2
TABLE 5-3 DESIGN CRITERIA FOR NWRF INFLUENT AND GRIT PUMPS 5-3
TABLE 5-4 DESIGN CRITERIA FOR NWRF ACTIVATED SLUDGE BASINS 5-4
TABLE 5-5 DESIGN CRITERIA FOR NWRF SECONDARY CLARIFIERS 5-5
TABLE 5-6 DESIGN CRITERIA FOR NWRF RAS AND IR PUMPS AND MIXERS 5-5
TABLE 5-7 DESIGN CRITERIA FOR NWRF SOLIDS HANDLING TANKS 5-6
TABLE 5-8 DESIGN CRITERIA FOR NWRF SOLIDS HANDLING PUMPS AND MIXERS 5-7
TABLE 5-9 TOWN OF ERIE NORTH WATER RECLAMATION FACILITY LEGAL DESCRIPTION AND CAPACITY 5-7
TABLE 5-10 DESIGN CRITERIA FOR SWRF 5-9
TABLE 5-11 DESIGN CRITERIA FOR SWRF INFLUENT WET WELL 5-10
TABLE 5-12 DESIGN CRITERIA FOR SWRF INFLUENT PUMPS 5-10
TABLE 5-13 DESIGN CRITERIA FOR SWRF VULCAN STEP SCREEN AND SCREENING WASH PRESS 5-11
TABLE 5-14 DESIGN CRITERIA FOR THE SWRF GRIT BASINS 5-12
TABLE 5-15 DESIGN CRITERIA FOR THE SWRF GRIT PUMPS 5-13
TABLE 5-16 MONTHLY AVERAGE EFFLUENT BOD, MG/L 5-14
TABLE 5-17 MONTHLY AVERAGE EFFLUENT TSS, MG/L 5-14
TABLE 5-18 MONTHLY AVERAGE EFFLUENT AMMONIA, MG/L AS N 5-14
TABLE 5-19 MONTHLY AVERAGE EFFLUENT NITRATE, MG/L AS N 5-15
TABLE 5-20 MONTHLY AVERAGE EFFLUENT PH, MG/L 5-15
TABLE 5-21 MONTHLY AVERAGE EFFLUENT TEMPERATURE, MG/L 5-15
TABLE 5-22 DESIGN CRITERIA FOR SWRF AERATION BASINS 5-16
TABLE 5-23 DESIGN CRITERIA FOR SWRF ACTIVATED SLUDGE BLOWERS 5-16
TABLE 5-24 DESIGN CRITERIA FOR SWRF SECONDARY CLARIFIERS 5-18
TABLE 5-25 DESIGN CRITERIA FOR SWRF SECONDARY CLARIFIER SPLITTER BOX 5-18
TABLE 5-26 DESIGN CRITERIA FOR SWRF RAS AND WAS PUMPS 5-19
TABLE 5-27 DESIGN CRITERIA FOR SWRF UV DISINFECTION SYSTEM 5-20
TABLE 5-28 DESIGN CRITERIA FOR SWRF AEROBIC DIGESTERS AND BIOSOLIDS HOLDING TANKS 5-20
TABLE 5-29 DESIGN CRITERIA FOR SLUDGE TRANSFER PUMPS 5-21
TABLE 5-30 DESIGN CRITERIA FOR SWRF DIGESTER BLOWERS 5-22
TABLE 5-31 BIOSOLIDS GENERATED AT SWRF FROM 2005 THROUGH 2009 5-22
TABLE 5-32 SWRF BIOSOLIDS QUALITY DATA FOR 2005 THROUGH 2007 5-23
TABLE 5-33 TOWN OF ERIE SOUTH WATER RECLAMATION FACILITY LEGAL DESCRIPTION AND CAPACITY 5-23

SECTION 6 ..... **6-1**

COLLECTION SYSTEM **6-1**

TABLE 6-1 SELECTED FUTURE FLOWS FOR PLANNING PURPOSES 6-6
TABLE 6-2 PROJECTED FUTURE AVERAGE AND MAXIMUM MONTH FLOWS 6-6
TABLE 6-3 ESTIMATED PEAK HOUR PEAKING FACTORS AND PEAK HOUR FLOWS BY FACILITY 6-7
TABLE 6-4 EVALUATION CRITERIA FOR WASTEWATER HYDRAULIC MODELING 6-9
TABLE 6-5 EXISTING / 2010 WASTEWATER HYDRAULIC MODELING RESULTS 6-10
TABLE 6-6 FUTURE WASTEWATER COLLECTION SYSTEM INTERCEPTORS FROM 2006 MASTER PLAN 6-11
TABLE 6-7 FUTURE WASTEWATER COLLECTION SYSTEM EXTENSIONS FROM 2006 MASTER PLAN 6-11
TABLE 6-8 FUTURE LIFT STATIONS FROM 2006 MASTER PLAN 6-12
TABLE 6-9 CAPACITY IMPROVEMENTS RECOMMENDED IN 2006 MASTER PLAN 6-13
TABLE 6-10 FUTURE / BUILD-OUT WASTEWATER HYDRAULIC MODELING RESULTS 6-13
TABLE 6-11 CAPITAL IMPROVEMENT RECOMMENDATIONS 6-14
TABLE 6-12 PEAK HOUR PEAKING FACTORS FOR COMMUNITIES IN COLORADO 6-19
TABLE 6-13 RECOMMENDED CLEANING AND INSPECTION INTERVALS FOR MANHOLES AND SEWER LINES 6-20

SECTION 7 ..... **7-1**

WATER QUALITY CHARACTERIZATION **7-1**

TABLE 7-1 WATER QUALITY STANDARDS FOR COAL CREEK AND BOULDER CREEK 7-6
TABLE 7-2 TVS-BASED METALS WATER QUALITY STANDARDS FOR ERIE SOUTH WRF 7-7
TABLE 7-3 TVS-BASED METALS WATER QUALITY STANDARDS FOR ERIE NORTH WRF 7-7
TABLE 7-4 AMBIENT WATER QUALITY FOR COAL CREEK 7-8
TABLE 7-5 AMBIENT WATER QUALITY FOR BOULDER CREEK 7-9
TABLE 7-6 LOW FLOWS FOR COAL CREEK AS CALCULATED IN THE BOULDER CREEK TMDL FOR AMMONIA 7-9
TABLE 7-7 EFFLUENT AMMONIA AND NITRATE LIMITS FOR SWRF FROM THE 2008 TMDL ANALYSIS 7-10
TABLE 7-8 TOWN OF ERIE SOUTH WRF EFFLUENT LIMITS AS OF JANUARY 2011 7-12
TABLE 7-9 TOWN OF ERIE NORTH WRF EFFLUENT LIMITS AS OF JANUARY 2011 7-13
TABLE 7-10 PERMITTED WASTEWATER DISCHARGERS 7-16
TABLE 7-11 WATER DIVERSIONS ON BOULDER AND COAL CREEKS 7-17

SECTION 8 ..... **8-1**

ALTERNATIVES ANALYSIS **8-1**

TABLE 8-1 SELECTED FUTURE FLOWS FOR PLANNING PURPOSES 8-1
TABLE 8-2 PROJECTED FUTURE AVERAGE AND MAXIMUM MONTH FLOWS 8-1
TABLE 8-3 FUTURE LOADS FOR TOWN OF ERIE FROM HISTORIC GENERATION RATES 8-2
TABLE 8-4 COST FACTORS USED TO DEVELOP ESTIMATED TOTAL PROJECT COSTS 8-2
TABLE 8-5 DIMENSIONS AND VOLUMES OF EXISTING BASINS 8-6
TABLE 8-6 COMPARISON OF SWRF DESIGN PARAMETERS TO STATE DESIGN CRITERIA 8-9
TABLE 8-7 ESTIMATED CONSTRUCTION COST TO CONVERT EXISTING BASINS TO ACTIVATED SLUDGE 8-10
TABLE 8-8 ESTIMATED CONSTRUCTION COST FOR EXTENDED AERATION ACTIVATED SLUDGE 8-11
TABLE 8-9 MEMBRANE CLEANING PROTOCOLS 8-14
TABLE 8-10 DESIGN CRITERIA FOR AN MBR SYSTEM 8-15
TABLE 8-11 ESTIMATED PROJECT COST FOR MEMBRANE BIOREACTOR (MBR) SYSTEM 8-16
TABLE 8-12 DESIGN CRITERIA FOR AN IFAS SYSTEM 8-19
TABLE 8-13 ESTIMATED PROJECT COST FOR INTEGRATED FIXED FILM ACTIVATED SLUDGE (IFAS) SYSTEM 8-21
TABLE 8-14 PROCESS TECHNOLOGY SELECTION SUMMARY 8-23
TABLE 8-15 REUSE WATER QUALITY REQUIREMENTS FROM REGULATION 84 8-24
TABLE 8-16 DESIGN CRITERIA FOR KRUGER DISCFILTER 8-25
TABLE 8-17 ESTIMATED CONSTRUCTION COST FOR KRUGER DISCFILTER. 8-26
TABLE 8-18 ESTIMATED CONSTRUCTION COSTS FOR MILLER-LEAHMAN ULTRAFILTRATION FILTER 8-27
TABLE 8-19 ESTIMATED CONSTRUCTION COST FOR VOLCANO FILTER 8-28
TABLE 8-20 ESTIMATED INCREASE IN SLUDGE PRODUCTION FROM CHEMICAL PHOSPHORUS REMOVAL 8-31
TABLE 8-21 CONSTRUCTION COST ESTIMATE FOR PREPARING PONDS FOR REUSE WATER STORAGE 8-32
TABLE 8-22 ROTARY DRUM THICKENER DESIGN CRITERIA 8-34
TABLE 8-23 ESTIMATED CONSTRUCTION COST FOR ROTARY DRUM THICKENER 8-34

SECTION 9 ..... **9-1**

MANAGEMENT AND FINANCIAL PLANS **9-1**

TABLE 9-1 PLANNED CAPITAL IMPROVEMENT PROJECTS 9-3
TABLE 9-2 TOWN OF ERIE WASTEWATER CASH FLOW 20 YEAR PROJECTION 9-4
TABLE 9-3 TOWN OF ERIE PROJECTED WASTEWATER EXPENSES 9-5
TABLE 9-4 2008 AND 2009 SEWER CONNECTION FEES 9-6
TABLE 9-5 EXISTING AND FUTURE SEWER SERVICE FEES 9-7
TABLE 9-6 COMPARISON OF 2010-2011 RESIDENTIAL WASTEWATER UTILITY SERVICE CHARGES. 9-8

FIGURES

SECTION 2 ..... **2-1**

FIGURE 2-1 EXISTING SERVICE AREA 2-1

INTRODUCTION **2-1**

SECTION 3 ..... **3-1**

GENERAL PLANNING **3-1**

FIGURE 3-1 WASTEWATER TREATMENT PLANTS NEAR TOWN OF ERIE 3-1
FIGURE 3-2 ECOREGIONS 3-7
FIGURE 3-3 SERVICE AREA TOPOGRAPHY 3-12
FIGURE 3-4 UNDERLYING GEOLOGY 3-12
FIGURE 3-5 BOULDER-WELD COUNTY UNDERGROUND COAL MINES NEAR ERIE, COLORADO 3-13
FIGURE 3-6 SOUTH WATER RECLAMATION FACILITY SOIL BORES 3-14
FIGURE 3-7 NORTH WATER RECLAMATION FACILITY SOIL BORES 3-15
FIGURE 3-8 100-YEAR FLOOD PLAIN FOR SOUTH WATER RECLAMATION FACILITY 3-16
FIGURE 3-9 100-YEAR FLOOD PLAING FOR NORTH WATER RECLAMATION FACILITY 3-17
FIGURE 3-10 WILDLIFE HABITAT RANGES 3-19
FIGURE 3-11 ENVIRONMENTAL CONSERVATION AREAS 3-19
FIGURE 3-12 SIGNIFICANT ENVIRONMENTAL COMMUNITIES 3-19
FIGURE 3-12B LEGEND FOR SIGNIFICANT ENVIRONMENTAL COMMUNITIES 3-19

SECTION 4 ..... **4-1**

WASTEWATER CHARACTERISTICS **4-1**

FIGURE 4-1 TOWN OF ERIE SERVICE AND PLANNING AREAS 4-1
FIGURE 4-2 TOWN OF ERIE PROPOSED 208 BOUNDARY 4-2
FIGURE 4-3 TOWN OF ERIE WATER AND SEWER LINES 4-3
FIGURE 4-4 HISTORIC POPULATION GROWTH 4-6
FIGURE 4-5 DEVELOPMENT MAP 4-7
FIGURE 4-6 INFLUENT FLOW AT SWRF ..... 4-**ERROR! BOOKMARK NOT DEFINED.**

SECTION 5 ..... **5-1**

EVALUATION OF WASTEWATER TREATMENT FACILITIES **5-1**

FIGURE 5-1 NWRF PROCESS DIAGRAM 5-1
FIGURE 5-2 NWRF HYDRAULIC PROFILE 5-1
FIGURE 5-3 KALDNES MEDIA 5-3
FIGURE 5-4 IFAS MEDIA RETENTION SCREENS 5-4
FIGURE 5-5 NWRF SITE LAYOUT 5-8
FIGURE 5-6 SWRF PROCESS DIAGRAM 5-9
FIGURE 5-7 SWRF HYDRAULIC PROFILE 5-9
FIGURE 5-8 SWRF SITE LAYOUT 5-24

SECTION 6 ..... **6-1**

COLLECTION SYSTEM **6-1**

FIGURE 6-1 EXISTING COLLECTION SYSTEM WITH CONTOURS AND DRAINAGE BASINS 6-1
FIGURE 6-2 SEWER FLOW DIAGRAM THROUGH DRAINAGE BASINS 6-2
FIGURE 6-3 DIVERSION STRUCTURE LOCATIONS 6-3
FIGURE 6-4 WEST SIDE INTERCEPTOR DIVERSION STRUCTURE 6-3
FIGURE 6-5 COAL CREEK AND OLD TOWN INTERCEPTORS DIVERSION STRUCTURE 6-3
FIGURE 6-6 DIURNAL CURVE 6-8

FIGURE 6-7 PEAK HOUR FOR 2010 - MINIMUM VELOCITY 6-10
FIGURE 6-8 WASTEWATER COLLECTION SYSTEM RECOMMENDED IMPROVEMENTS (B&V) 6-11
FIGURE 6-9 MONTHLY MAXIMUM AT BOULDOUT - MAXIMUM FLOW DEPTH/PIPE DIAMETER 6-13
FIGURE 6-10 PEAK HOUR AT BUILDOUT - MAXIMUM FLOW DEPTH/PIPE DIAMETER 6-13
FIGURE 6-11 PEAK HOUR AT BUILDOUT - MAXIMUM VELOCITY 6-13
FIGURE 6-12 RECOMMENDATIONS FOR COLLECTION SYSTEM 6-14

SECTION 7 7-1

WATER QUALITY CHARACTERIZATION 7-1

FIGURE 7-1 SOUTH PLATTE RIVER WATERSHED 7-2
FIGURE 7-2 BOULDER CREEK WATERSHED 7-3
FIGURE 7-3 WASTEWATER PLANTS NEAR TOWN OF ERIE 7-16
FIGURE 7-4 WELLS WITHIN ONE-MILE OF SWRF AND NWRF 7-16
FIGURE 7-5 WATER DIVERSIONS WITHIN 5-MILES OF SWRF AND NWRF 7-18

SECTION 8 8-1

ALTERNATIVES ANALYSIS 8-1

FIGURE 8-1 EXPANSION SCENARIO TIMING 8-5
FIGURE 8-2 SWRF EXISTING PROCESS LAYOUT 8-6
FIGURE 8-3 MEMBRANE BIOREACTOR 8-13
FIGURE 8-4 MEMBRANE BIOREACTOR SCHEMATIC 8-15
FIGURE 8-5 SITE LAYOUT FOR INTEGRATED FIXED FILM ACTIVATED SLUDGE (IFAS) 8-19
FIGURE 8-6 IFAS PROCESS SCHEMATIC 8-19
FIGURE 8-7 IFAS FUTURE DIRECTION SITE LAYOUT 8-22
FIGURE 8-8 KRUGER DISC FILTER 8-25
FIGURE 8-9 ULTRAFILTRATION UNIT 8-26
FIGURE 8-10 ROTARY DRUM THICKENER 8-33

APPENDICES

APPENDIX A Intergovernmental Agreements

APPENDIX B Selections from 2007 Non-Potable Municipal Water System Master Plan

APPENDIX C Geotechnical Reports and Soil Maps for SWRF and NWRF

APPENDIX D Environmental Report including Wetlands Survey and Wildlife Evaluation

APPENDIX E LOMR Application for Boulder Creek

APPENDIX F Discharge Permits and Preliminary Effluent Limits

APPENDIX G Stormwater Management Plans

APPENDIX H Selections from 2007 Erie Water and Wastewater Master Plan, dated April 2007

APPENDIX I Activated Sludge Calculations for SWRF

APPENDIX J Equipment Data

APPENDIX K Selections from Water and Wastewater Rates and Fees January 2009

APPENDIX L Capital Improvements Project List

APPENDIX M Legal Descriptions and Proof of Site Ownership for SWRF and NWRF

APPENDIX N Proof of Site Ownership

SECTION 1
EXECUTIVE SUMMARY

The Town of Erie is located along the Front Range, north of Denver and east of Boulder along Coal Creek. Erie's planning area is located in Boulder and Weld counties. The planning area is about 50 square miles. The boundary of the planning area is somewhat irregular but, in general, is formed by Baseline Road (Highway 7) on the south and by Interstate 25 on the east. The western boundary is about ½ mile west of Highway 287 and the northern boundary is approximately 1.7 miles north of State Highway 52. The Brownsville Water and Sanitation District is surrounded by Erie's planning area, but is not part of the planning area. The Town has experienced rapid population growth over the last ten years, nearly doubling in size to 18,135 residents as of the 2010 census.

The Town of Erie has two wastewater reclamation facilities. The South Water Reclamation Facility (SWRF) is located near the center of the service area just north of the intersection of Briggs Street and Evans Street. Briggs Street becomes County Road 1 ½ as it passes the SWRF. The SWRF is an extended aeration activated sludge plant and has a rated capacity of 1.6 mgd and 3,870 ppd of Biochemical Oxygen Demand (BOD). The North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The NWRF has a rated capacity of 1.5 mgd and 3,233 ppd of BOD. The NWRF is located north of Highway 52 and east of County Line Road along Boulder Creek.

OBJECTIVES FOR WASTEWATER UTILITY PLAN

The purpose of the 2010 Wastewater Utility Plan update is twofold: to consolidate relevant sections of past planning efforts into a single comprehensive document and to support a modification to Erie's 208 planning area boundary. The proposed 208 boundary will match Erie's existing planning area boundary. Updating the 208 boundary will facilitate water reuse, maintain water rights, and allow the Town to provide equivalent tap and service fees throughout their planning area.

Portions of Erie's historic planning area boundary overlap with the St. Vrain Sanitation District's 208 boundary. These areas include a section that is generally bounded to the north by County Road 10, the south by Erie Parkway, to the west by County Road 7, and to the east by I-25. A second area of overlap is located north of County Road 12 and east of County Road 7. A map of the proposed 208 area is shown in Figure 2-1. Sanitary sewer service to portions of such areas by the Town or the District could require extensive trunk line extensions. The Town and the District acknowledge that regional cooperation by local governments to achieve economical and efficient services is authorized by Sec. 29-1-203, C.R.S., and is in the best interests of public health and stream quality. The Town of Erie and St. Vrain Sanitation District have agreed to designate these overlapping areas as coordinated or shared 208 areas. Residents within these areas will have the option of receiving wastewater service from either the Town or the District depending on proximity of gravity sewer lines and overall cost of service. If these areas are annexed by the Town of Erie, Erie will provide service. A copy of the Intergovernmental Agency

Agreement (IGA) between the Town and the District, adopted on November 19, 2012, is included in Appendix A.

Erie has invested heavily in reuse water infrastructure including a new storage reservoir located at the NWRF site. Updating the 208 boundary will maximize capture of water for reuse by ensuring that areas receiving potable water from the Town also return wastewater to the Erie WWTFs. A portion of Erie's water portfolio may be used to extinction. Maximizing the water and sewer collection areas maximizes opportunities for reuse and ensures multiple cycles of "use to extinction" water. Currently, water and sewer fees differ depending on whether residents are located inside or outside of the designated service area. Updating the 208 Boundary will directly benefit users that are currently outside of the designated service area by making them eligible for in-service area rates thereby reducing their tap and service fees by one-half.

Updating the 208 boundary will make it consistent with Erie's historic planning area. Erie's internal planning area boundary has not changed since 1996 and has been referenced in numerous, publically available planning documents including Erie's 2001 Wastewater Utility Plan, 2005 Comprehensive Plan, 2006 Water and Wastewater Master Plan, and 2008 Update to the Wastewater Utility Plan. Much of the historic planning area is either already within the Town limits or has petitioned to be annexed.

Minor changes to the eastern boundary will bring the 208 Boundary edge up to I-25 which forms a natural barrier. Sewer and water lines may cross I-25; however, this is logistically difficult and can be costly. Erie has an Intergovernmental Agreement (IGA) with the City of Dacono stating that neither municipality will annex property on the opposing side of I-25. The IGA makes Erie the logical water and sewer provider for all parcels located between I-25 and Erie's Town limits. This is particularly true for Section 15 which is bordered by Erie on three sides. The 208 boundary will be extended west past Highway 287 and north past Niwot Road. Most of this area is designated as open space in Erie planning documents.

Changes to Erie's urban growth boundary (UGB) were approved by DRCOG in 2009. The updated UGB includes Sections 3, 10, and 15 which are located immediately west of Interstate-25. Generally, the wastewater utility service area (WUSA) is equivalent to the UGB. In some instances, the WUSA may be larger than the UGB. By expanding the UGB into these areas, the Town has already indicated an intent to provide wastewater service.

The primary reason for updating the 208 Boundary is to ensure recapture of wastewater flows for non-potable reuse. Maximizing reuse is consistent with the stated goals of NFRWQPA. Erie plans to use non-potable water to meet a portion of its irrigation and other non-potable water demands. Potential non-potable supplies include raw ditch water, raw water that would otherwise be treated for potable use, reservoir water, and treated wastewater return flows. Treated effluent may be recaptured at either of Erie's two wastewater treatment facilities (WWTFs). Flows captured at the NWRF may be transmitted to the SWRF. Reusable return flows from the SWRF that do not coincide with seasonal non-potable demands could be conveyed via Coal Creek and Boulder Creek to the NWRF storage facility.

BASIS OF PLANNING

Erie experienced modest growth until 1990. Between 1990 and 2000, population increased by a factor of five from 1,258 to 6,291 residents. Since 2000, growth has averaged ten percent per year and has nearly tripled the Town's population. As of the 2010 Census, Erie had 18,135 residents.

In December 2005, Erie's Planning Commission adopted the Town's Comprehensive Plan. The Comprehensive Plan included input from diverse sources including citizens, the Erie Board of Trustees, the Comprehensive Plan Advisory Committee, and several planning consultants: Clarion Associates, LSA Associates, BBC Research, and EDAW. One of the many tasks that the comprehensive plan took on was a detailed analysis of future population growth in Erie's planning area. The Town considers it more accurate than projections by the State Demographers Office, DRCOG, and other estimates because the analysis was so detailed.

Table 1-1 Future Population Growth

Date	Population
2010	18,135
2015	26,525
2020	33,525
2025	40,680
2030	49,625
Buildout	68,820

In past years, growth has been as high as 22% in one year and has averaged 10% per year since 2000. Future growth will also vary from year to year, but is expected to average 6% per year until the year 2017 followed by growth at 4% per year until buildout. Population projections from the Comprehensive Plan are presented in Table 1-1. The buildout population of 68,820 residents will not completely fill all land-use parcels to their maximum capacities as defined by land use.

IMPLEMENTATION SCHEDULE AND TIMING ISSUES

The Wastewater Utility Plan (WUP) will move through the approval processes at North Front Range Water Quality Planning Association (NFRWQPA) beginning in mid-2011. Final approval of the proposed 208 Boundary Change and Wastewater Utility Plan is expected in 2012. The WUP will then be filed with the State.

APPLICABLE COMPLIANCE SCHEDULES OR CONSENT DECREES

The Town received a new discharge permit for the South Water Reclamation Facility (SWRF) in September 2011. The new permit became effective on October 1, 2011. The permit contains compliance schedules for installing temperature monitoring equipment and for meeting new effluent ammonia limits. It does not contain a phosphorus limit. The SWRF must be able to meet the permitted ammonia limits by April 30, 2017.

WASTEWATER FLOW AND LOAD PROJECTIONS

For planning purposes, future flows and loads were estimated using both historic generation rates and higher per capita flow generation rates that agree with planning criteria recommended by DRCOG and the State of Colorado. Historically, average wastewater generation has been 62 gpcd which is extremely low even for Colorado communities. Currently, the Town is a bedroom community with few commercial accounts and no major industrial users. This may change in the future. More commercial and industrial

users will change the characteristics of the influent wastewater and will result in higher observed per capita generation rates. For these reasons, the flow projections presented in Table 1-2 were selected to ensure adequate future capacity in both the collection system and the treatment facilities. Future loads are given in Table 1-3.

Table 1-2 Selected Future Flows for Planning Purposes

Parameter	2025	Buildout
Annual Average Flow Generation Rate, gpcd	90	90
Maximum Month Flow Generation Rate, gpcd	118	118
Annual Average Daily Flow, mgd	3.70	6.26
Maximum Month Flow, mgd (MM PF = 1.3)	4.80	8.12
Peak Day Flow, mgd (PD PF = 1.7)	6.30	10.66
Maximum Month Flow to SWRF, mgd	1.60	1.60
Annual Average Flow to SWRF, mgd	1.23	1.23
Peak Hour Peaking Factor for SWRF	3.52	3.52
Peak Hour Flow to SWRF, mgd	4.35	4.35
Maximum Month Flow to NWRF, mgd	3.20	6.52
Annual Average Flow to NWRF, mgd	2.46	5.02
Peak Hour Peaking Factor for NWRF	3.14	2.79
Peak Hour Flow to NWRF, mgd	7.74	14.02

MM PF = Maximum Month Peaking Factor, PD PF = Peak Day Peaking Factor

Future annual average loads for BOD, TSS, and ammonia were estimated by multiplying the projected population for each year by the per capita generation rates calculated from historic data. The per capita generation rates used for BOD, TSS, and ammonia are 0.16 ppd, 0.19 ppd, and 0.018 ppd, respectively. These generation rates are the highest annual average generation rates observed over the last ten years. Future maximum month loads were estimated by multiplying the calculated annual average loads by the maximum month peaking factors determined from historic data.

Table 1-3 Future Loads for Town of Erie from Historic Generation Rates

		BOD, ppd		TSS, ppd		NH_3-N, ppd	
Year	Population[1]	AA	MM	AA	MM	AA	MM
2010	18,135	2902	3773	3446	4135	326	414
2015	26525	4244	5517	5040	6048	477	606
2020	33525	5364	6973	6370	7644	603	766
2025	40680	6509	8462	7729	9275	732	930
2030	49625	7940	10322	9429	11315	893	1134
Buildout	68820	11011	14314	13076	15691	1239	1574

AAF = Annual Average Flow, MM = Maximum Month Average, AA = Annual Average
AA per capita generation rates utilized are 0.16 ppd BOD, 0.19 ppd TSS, and 0.018 ppd NH_3N
Maximum Month Peaking Factors Applied are BOD = 1.30, TSS = 1.20, Ammonia = 1.27

NWRF PROCESS SYSTEM

The NWRF was constructed in 2010 and became operational in early 2011. The NWRF has a rated capacity of 1.5 mgd and 3,233 ppd of BOD. A new interceptor with bypass structures at the SWRF allow the Town to distribute flows in any proportion between the SWRF and NWRF.

The NWRF headworks consists of a mechanical screen, manual bar screen in a bypass channel, lift station, and grit basin. The secondary process is a three-stage biological nutrient removal integrated fixed film activated sludge (IFAS) process with anaerobic zones, anoxic zones, and aerated zones followed by secondary clarification. The secondary process is designed for biological phosphorus removal, nitrification, and denitrification. Secondary clarifier effluent flows to the disinfection room where a splitter structure diverts flow to either a tertiary cloth media filter or directly to the UV disinfection units. The tertiary filter may treat up to 3.6 MGD of flow to produce reuse quality water which will be stored in an on-site reservoir. Residual solids from the activated sludge process will be stabilized with lime prior to dewatering with a screw press. The NWRF will produce Class A biosolids using lime stabilization and time/temperature criteria.

SWRF PROCESS SYSTEM

The SWRF was constructed in 1998 with a design capacity of 0.6 mgd to replace an aging lagoon treatment facility. Two of the lagoon cells remain, but are no longer in service. Four years later, the facility was rated at 0.8 mgd of capacity. In 2003, the SWRF was expanded for a hydraulic capacity of 1.2 mgd and an organic capacity of 2,900 ppd of BOD. The expansion included an influent pump upgrade, replacement of a mechanical screen, increased capacity for the aeration system, blower replacement, construction of a clarifier splitter structure, construction of a second secondary clarifier, modifications to the RAS and WAS pumping systems, replacement of the UV disinfection unit, piping modifications, and a decant system for the digesters (HDR Engineering, Inc., 2003). The SWRF was rerated again in 2006 at 1.6 mgd and 3,870 ppd of BOD (Burns and McDonnell, 2006). A new discharge permit which incorporates the rerated capacity was issued in September 2011 and became effective October 1, 2011. The current permit lists capacity at 1.6 mgd and 3,870 ppd of BOD.

The SWRF headworks consists of an influent pump station, step screen, manual bar screen, and two grit basins operated in series. Wastewater is treated to remove organic matter and suspended solids with an activated sludge process followed by secondary clarification and ultra-violet disinfection. The treatment process was not originally designed to nitrify or denitrify; however, effluent ammonia concentrations are typically below 3 mg/L and effluent nitrate concentrations are typically below 15 mg/L. Residual solids from the activated sludge process are stabilized with aerobic digestion prior to land application. Stabilized biosolids are thickened within the digester though multiple decant cycles. The headworks and disinfection equipment are enclosed in buildings. All other processes are outdoors.

COLLECTION SYSTEM

Within the Town of Erie's planning area, most of the existing wastewater collection system flows by gravity either from south to north, or west to east, to either the South or North Water Reclamation Facilities (WRFs). Land elevations in the service area vary from 5,250 feet on the south end, to 4,950 at the north border; a drop of 300 feet.

The collection system contains 107,109 linear feet or 20.29 miles of pipes with 10-inch diameters and larger and 336,040 linear feet or 63.64 miles of pipes with 8-inch diameters and smaller. There are 2,199 known manholes. The collection system contains two diversion structures located near the SWRF. The

diversion structures allow plant staff to control the proportion of flow received at the SWRF. The existing sewer collection system serves ten of the fifteen drainage basins. The remaining five drainage basins are not developed and therefore do not currently generate wastewater flow.

The collection system was previously modeled with H2OMap Sewer (MWHSoft, GIS Professional Suite 8.0, Update #4) in 2007 (Black and Veatch April 2007). An addendum to the 2007 Water and Wastewater Masterplan was issued in 2008. The addendum discussed an alternative alignment for portions of the Highway 52 and Interstate interceptors that would eliminate the need to construct lift stations. Several capital improvement projects (CIPs) identified in the 2006 Water and Wastewater Master Plan have been completed or are under construction. The model was updated using drawings issued for construction including: 1) addition of diversion structures, Kenosha Farms Lift Station bypass line, NWRF interceptor, and NWRF; 2) removal of the Kenosha Farms Lift Station.

The existing wastewater collection system (2010) was modeled at annual average flow (AAF), maximum month (MM), and peak hour conditions. Modeling indicates that under current conditions, the existing collection system is under 70% capacity. Flow velocities at peak hour range between 0.023 and 6.914 fps. No surcharging was identified.

The existing collection system will need to be expanded to accommodate future flows generated within the existing service area and to extend service into currently undeveloped drainage basins. Small diameter interceptors and laterals can be constructed by developers; however, the Town may need to take an active role in the development, construction, and funding of major interceptors. Two new major interceptors will be needed to serve currently undeveloped portions of the planning area. These interceptors are the Interstate interceptor and the Highway 52 interceptor. Both of these interceptors were modeled as traditional gravity sewers with lift stations and as deep gravity sewers. In some locations, the deep gravity sewers were up to 35 feet deep. Two small lift stations are recommended for future. The Northeast lift station which will serve the Northeast drainage basin and the Northwest lift station which will serve the Northwest drainage basin. These lift stations are relatively small and will serve defined areas. Both of these lift stations will likely be constructed and paid for by developers. The Town anticipates construction of several temporary lift stations to serve outlying developments until gravity interceptors can be constructed. These lift stations and other recommendations are discussed in Section 6.

Modeling did not identify any surcharging under 2010, 2025, or buildout flows when the recommended improvements were in place. Four areas were identified for capacity improvements including: Arapahoe Ridge, West Side, Old Town, and the Austin Industrial Line.

ALTERNATIVES ANALYSIS

The NWRF is an Integrated Fixed Film Activated Sludge System (IFAS) with an initial treatment capacity of 1.5 MGD and 3,233 ppd of BOD; expandable to 3.6 MGD and 7,750 ppd of BOD. Expansion will require additional IFAS media as well as additional equipment and another secondary clarifier; although a paper rerating to 1.75 or 2.0 mgd may be possible based on performance data. The NWRF was designed to remove ammonia, nitrate, and phosphorus. Phosphorus removal will be done

biologically through an anaerobic zone. The Town of Erie could receive effluent permit limits as low as 1.0 mg/L as P as early as 2022. Phosphorus removal may be done biologically down to 1.0 mg/L or chemically. Chemical precipitation with either alum or ferric chloride followed by tertiary filtration will be required to remove phosphorus below 0.8 mg/L as P. Chemical addition can be done in the secondary clarifier wet wells or after clarification followed by filtration with the existing cloth media filter.

The SWRF has a rated capacity of 1.6 MGD and 3,870 ppd of BOD. For a variety of reasons discussed throughout this WUP, the SWRF will be required to remove ammonia to lower levels in the future as well as both nitrate and phosphorus. The SWRF will not be able to meet future permit limits as it is currently configured.

The final permit for the SWRF was issued in September 2011 and became effective on October 1, 2011. The permit contains compliance schedules for installation of temperature monitoring equipment and for meeting lower effluent ammonia limits. Effluent ammonia limits range between 1.9 and 4.4 mg/L. The SWRF must be able to meet the new limits by April 30, 2017. The new permit does not contain limits for nitrate or phosphorus.

Five alternatives were evaluated for modifying the SWRF secondary treatment process including: decreasing the organic and hydraulic loads to maximize nitrification, adding aeration basin space by converting the existing anaerobic digesters to aeration basins, construction of additional aeration basins, conversion to a membrane bioreactor activated sludge process, and conversion to an integrated fixed film activated sludge (IFAS) process. Options for phosphorus removal were also evaluated. The recommended alternative for the SWRF is to convert the existing activated sludge basins to IFAS and either construct an anaerobic zone for biological phosphorus removal or add facilities for chemical phosphorus removal followed by tertiary filtration. This alternative would mirror the recently completed NWRF. Converting the SWRF to IFAS will remove ammonia to less than 1 mg/L as N and total nitrogen below 10 mg/L as N. The existing aeration basins would be retrofitted to accommodate the IFAS equipment, so no new basins or clarifiers will be needed. Additional blower capacity may be required.

The estimated construction cost to convert the SWRF from activated sludge to a 1.6 mgd IFAS process is $4.6 million. Costs include modifications to the existing aeration basins and process piping, the IFAS media, pumps, aeration system, and screens, and modifications to the existing blowers to increase capacity. Costs do not include modifications to the solids handling process, anaerobic zones, chemical addition equipment for phosphorus removal, tertiary filtration, or upgrades to the site electrical service.

The aerobic digesters are preserved under this alternative; however, to maintain a rated capacity of 1.6 mgd, some type of solids thickening would likely be needed. Operations staff thicken solids in the aerobic digesters by allowing the solids to settle and decanting supernatant back to the plant headworks. The finished biosolids are between 1.5 and 2.5 percent solids. Thicker biosolids could be obtained by adding flocculants and increasing the settling time; however, extended settling times generate noxious odors that are unacceptable to the Town and nearby residents. The only technology evaluated for thickening was a rotary drum thickener although the solids screw press selected for the NWRF could also be applied at the SWRF. The drum thickener could be placed next to the existing digesters within a small

building. Thickening may be done as WAS enters the digester or by withdrawing digested solids from the digester for thickening and then returning the thickened solids to the digester. In either case, the filtrate would be returned to headworks. Estimated construction costs for a drum thickener and containment building are $0.94 million.

Tertiary filtration was evaluated for producing reuse quality water at the SWRF. Erie plans to use non-potable supplies consisting of raw water and reclaimed treated wastewater effluent to meet a portion of its current and projected water needs. Erie completed a Non-Potable Municipal Water System Master Plan in September 2007 (CDM September 2007). The Master Plan predicted that the highest non-potable demands at buildout would occur in the southeast portion of the service area, within 2 to 3 miles of Interstate I-25 and/or Highway 7 and around Old Town. The SWRF is better located to serve these areas than the NWRF. The Lower Boulder Ditch passes within 250 feet of the SWRF and the Cottonwood Extension Ditch (aka Erie Coal Creek Ditch) is within 2000 feet. Either or both of these canals could be used to transfer reuse water to the north central and eastern portions of the service area by gravity.

Three alternatives were considered for tertiary filtration: the Kruger Hyrdotech Disc-Filter, Miller-Leahman's ultrafiltration filter, and Lighthouse Filters' Volcano Continuous Downflow Filter. Each filter was sized based on an average daily flow of 1.0 mgd and a peak hour flow of 2.0 mgd. Each filter is capable of producing reuse quality water in accordance with Regulation 84. Each filter is compatible with chemical phosphorus removal and could be expanded for that purpose when the SWRF receives phosphorus limits. The Kruger disc-filter is the least costly of the three alternatives evaluated at $0.57 million installed. It has been used in Colorado for phosphorus removal and for generating reuse quality water. Two major advantages of this filter are that it fits within the hydraulic profile of the SWRF so a pump station is not needed and that it does not need to be inside a building. Erie selected the same technology for the NWRF. Utilizing it at the SWRF will reduce operator training requirements and spare parts inventories.

The SWRF location has two, unused lagoon pond cells from the original treatment plant. The first cell has a volume of 2.5 million gallons and the second pond has a volume of 2.1 million gallons. The ponds have been off-line since the original SWRF mechanical facility was constructed in 1998. The ponds could be repurposed for reuse water storage. The ponds will need some earthwork to remove vegetation, reshape the slopes, and prepare the surface for a synthetic liner. The estimated cost for repurposing the lagoons is $0.79 million.

MANAGEMENT AND FINANCIAL PLANS

The Town of Erie owns, operates, and manages its own wastewater reclamation facilities and collection system. The wastewater utility is a self-supporting enterprise fund with funding for annual operations and maintenance expenses, capital projects, and debt service met primarily through wastewater service rates and tap fees.

The Town completed a Water and Wastewater Rate Study in January 2009 (Red Oak Consulting January 2009). The primary goal of the study was to ensure that an adequate level of revenue from wastewater

service revenue is maintained to finance Erie's daily operations as well as future capital improvements and expansions. In December 2008, the Board of Trustees adopted the wastewater rates recommended in the study. Tap fees for single family residences increased slightly from $4,000 to $4,290. A single family home generating 6,000 gallons of wastewater in a month, paid $39 per month in 2009 and can expect to pay $65 per month in 2013 in service fees.

The North Water Reclamation Facility (NWRF) debt was financed through revenue bonds. At this time, it is anticipated that the debt financing for projects at the South Water Reclamation Facility (SWRF) will also be financed through revenue bonds. The Town anticipates the majority of new interceptors, sewer lines, and lift stations will be financed by private developers as development occurs.

SECTION 2
INTRODUCTION

The Town of Erie is located along the Front Range, north of Denver and east of Boulder along Coal Creek. Erie's planning area is located in Boulder and Weld counties. Figure 2-1 shows the current service area and planning area boundaries. The planning area is about 50 square miles. The boundary of the planning area is somewhat irregular but, in general, is formed by Baseline Road (Highway 7) on the south and by Interstate 25 on the east. The western boundary is about ½ mile west of Highway 287 and the northern boundary is approximately 1.7 miles north of State Highway 52. The Brownsville Water and Sanitation District is surrounded by Erie's planning area, but is not part of the planning area.

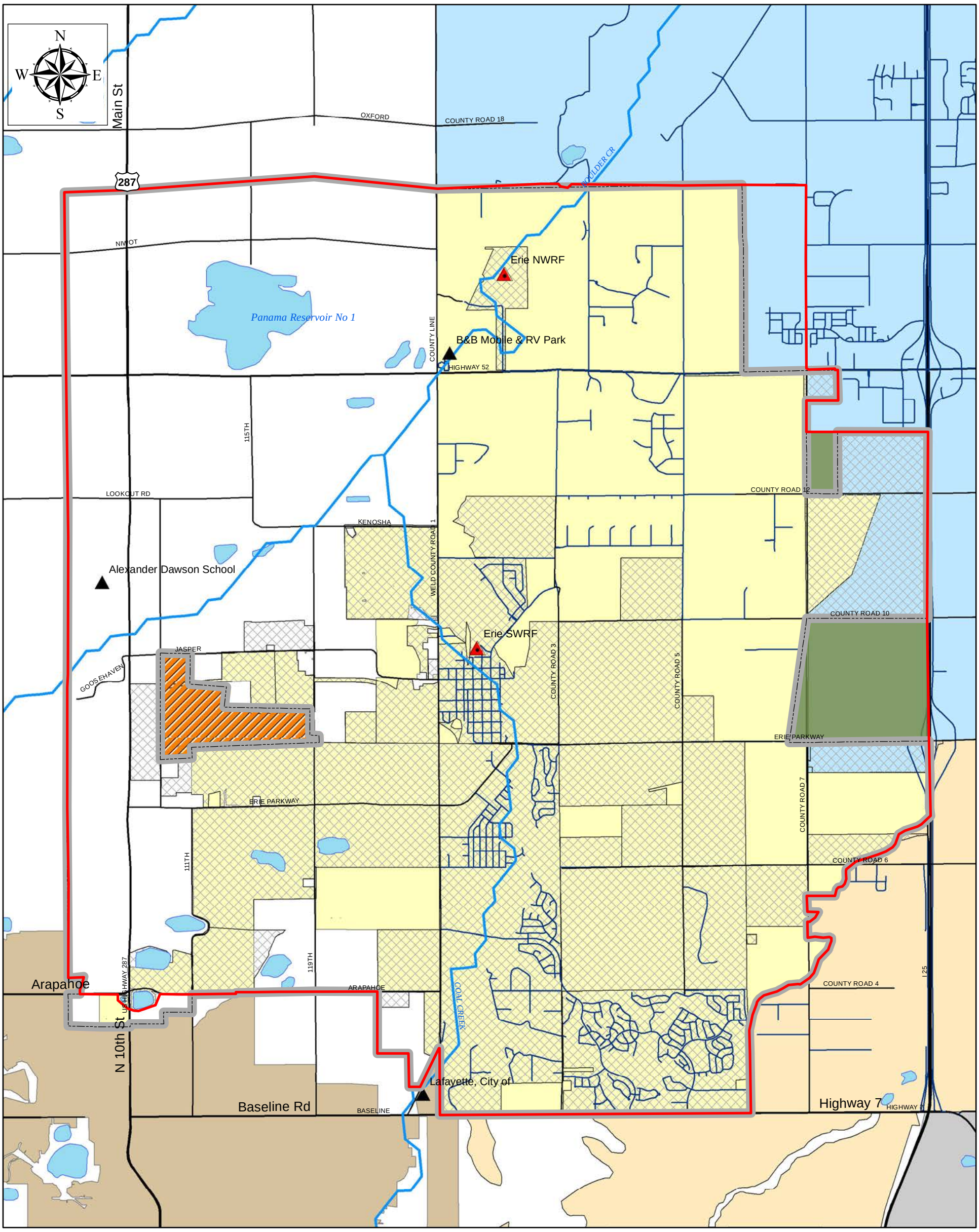
The Town of Erie prepared a Wastewater Facility Plan in 2001. The plan was updated in 2008 to reflect improvements made to the wastewater infrastructure between 2001 and 2008 and to support approval and construction of the new North Wastewater Reclamation Facility (NWRF). The Town has completed numerous other planning studies related to collection, distribution, and water reuse. The purpose of the 2011 Wastewater Utility Plan update is twofold: to consolidate relevant sections of past planning efforts into a single comprehensive document and to support a modification to Erie's 208 planning area boundary.

BACKGROUND AND HISTORY

The South Water Reclamation Facility (SWRF) was constructed in 1998 with a design capacity of 0.6 mgd to replace an aging lagoon treatment facility. Two of the lagoon cells remain, but are no longer in service. Four years later, the facility was rated at 0.8 mgd of capacity. In 2003, the SWRF was expanded for a hydraulic capacity of 1.2 mgd and an organic capacity of 2,900 ppd of BOD. The expansion included an influent pump upgrade, replacement of a mechanical screen, increased capacity for the aeration system, blower replacement, construction of a clarifier splitter structure, construction of a second secondary clarifier, modifications to the RAS and WAS pumping systems, replacement of the UV disinfection unit, piping modifications, and a decant system for the digesters (HDR Engineering, Inc., 2003). The SWRF was rerated again in 2006 at 1.6 mgd and 3,870 ppd of BOD (Burns and McDonnell, 2006). The rerating was approved by the State of Colorado in October 2006 and was incorporated into the new discharge permit issued in September 2011.

The NWRF was constructed in 2010 and became operational in early 2011. A new interceptor along with bypass structures allows the Town to distribute flow between the SWRF and NWRF.

The Town has completed numerous planning studies over the years. The most current and relevant planning documents are summarized in the following paragraphs. The Town's overall vision is set forth in its Comprehensive Plan. The Comprehensive Plan was last updated in 2005. It defines the planning area and sets guidelines for land use, residential density, and build-out populations for different areas. The planning area boundary is larger than the current 208 planning area (Figure 2-1). Erie's planning boundary has not changed since 1996 and has been used to define the study area for all of the following



Legend

- Historic Planning Boundary
- Proposed 208 Boundary
- Coordinated Area

0 0.25 0.5 1 Miles

- *Wastewater Treatment Plants*
- *Erie Water Reclamation Facilities*
- Colorado Major Rivers
- Lakes
- Minor Streets
- *Brownsville W&SD*
- Erie Incorporated

NFRWQPA 208 - 2009

- BROOMFIELD
- ERIE
- NORTHGLENN
- ST. VRAIN S.D.

DRCOG WUSA

- *Broomfield*
- *Erie*
- *Lafayette*
- *Northglenn*
- *Thornton*

NOTE: The proposed 208 bounday is Erie's historic planning area with modifications to accomodate the coordinated 208 planning area between the Town of Erie and St. Vrain Sanitation District.

Figure 2-1 Town of Erie Historic Planning Area and Proposed 208 Boundary

water and wastewater planning documents. Future population growth projections used within this Utility Plan were taken directly from Erie's 2005 Comprehensive Plan (Erie Planning Commission, 2005). The Town has policies in place to control and guide growth within the planning area.

The Town of Erie completed a Wastewater Utility Plan (WUP) in 2001 (HDR Engineering, Inc., 2001). The WUP followed the DRCOG outline and met basic requirements for planning purposes such as population, flow, and load projections. The 2001 WUP did not include an evaluation of the collection or distribution systems, a comprehensive performance evaluation of the existing treatment facility, an operations review, on-site laboratory evaluation, or a review of user rates. The 2001 WUP recommended rerating the South Water Reclamation Facility (SWRF) to 0.8 mgd from 0.6 mgd with an ultimate expansion up to 1.6 mgd. The 2001 WUP also recommended construction of a new North Water Reclamation Facility (NWRF) to capture future flows generated to the north and downstream of the SWRF. The NWRF was constructed in 2010 and became operational in early 2011.

The Town completed a Water and Wastewater Master Plan in 2006 (Black and Veatch, April 2007). The 2006 plan included hydraulic modeling of both the water distribution system and the wastewater collections system. The water distribution system was modeled using WaterGEMS and the collection system was modeled using MWHSoft Sewerpro. The 2006 plan updated population, water use, and wastewater generation projections, but did not evaluate either the water treatment plant or SWRF. The 2006 Master Plan made multiple recommendations for new sewer and water lines to serve areas of future population growth.

In September 2007, the Town completed a Non-Potable Municipal Water System Plan (CDM, 2007) to determine how much of future water needs could be met through reuse. Water rights issues prevent reuse of some water supplies, but the Town has a substantial amount of water rights that can be used to extinction. The Non-Potable Plan recommended capture of Windy Gap return flows at both the SWRF and NWRF for immediate reuse. The plan did not discuss modifications to the WRFs that may be needed to produce reuse quality water.

In June 2008, the Town hired Burns and McDonnell to perform an update of the 2001 WUP to facilitate design of the new NWRF and to ease passage of the required Site Application through NFRWQPA and the State of Colorado. This engineering report updated population forecasts, historic wastewater flows and loads, and the twenty-year projections for flows and loads (Burns and McDonnell, 2008). In April 2009, Burns and McDonnell submitted a Process Design Report to the State of Colorado for the NWRF. The Process Design Report includes detailed design information and engineering calculations for the NWRF (Burns and McDonnell, 2009). The NWRF design includes extensive infrastructure for wastewater reuse including a reuse water storage reservoir and pump station.

OBJECTIVES FOR WASTEWATER UTILITY PLAN

The purpose of the 2010 Wastewater Utility Plan update is twofold: to consolidate relevant sections of past planning efforts into a single comprehensive document and to support a modification to Erie's 208 planning area boundary. The proposed 208 boundary, shown in Figure 2-1, will match Erie's existing planning area boundary. Updating the 208 boundary will facilitate water reuse, maintain water rights, and allow the Town to provide equivalent tap and service fees throughout their planning area.

Portions of Erie's historic planning area boundary overlap with the St. Vrain Sanitation District's 208 boundary. These areas include a section that is generally bounded to the north by County Road 10, the south by Erie Parkway, to the west by County Road 7, and to the east by I-25. A second area of overlap is located north of County Road 12 and east of County Road 7. A map of the proposed 208 area is shown in Figure 2-1. Sanitary sewer service to portions of such areas by the Town or the District could require extensive trunk line extensions. The Town and the District acknowledge that regional cooperation by local governments to achieve economical and efficient services is authorized by Sec. 29-1-203, C.R.S., and is in the best interests of public health and stream quality. The Town of Erie and St. Vrain Sanitation District have agreed to designate these overlapping areas as coordinated or shared 208 areas. Residents within these areas will have the option of receiving wastewater service from either the Town or the District depending on proximity of gravity sewer lines and overall cost of service. If these areas are annexed by the Town of Erie, Erie will provide service. A copy of the Intergovernmental Agency Agreement (IGA) between the Town and the District, adopted on November 19, 2012, is included in Appendix A.

FACILITIES PLAN SUMMARY

The facilities plan is summarized in the executive summary.

IMPLEMENTATION SCHEDULE AND TIMING ISSUES

The Town received a new discharge permit for the South Water Reclamation Facility (SWRF) in September 2011. The new permit became effective on October 1, 2011. The permit contains compliance schedules for installing temperature monitoring equipment and for meeting new effluent ammonia limits. The SWRF must be able to meet the permitted ammonia and nitrate limits by April 30, 2017.

On May 14, 2012, the Water Quality Control Commission made changes to Regulation 31 which incorporate nutrient criteria into the basic water quality standards. The water quality standards must be adopted through the regular basin hearing process before they can be incorporated into discharge permits. The first possible adoption will be the South Platte River Basin in 2017. The Commission also adopted a new regulation, Regulation 85 – Nutrients Management Control, which sets interim limits for total inorganic nitrogen and phosphorus in discharge permits. The permitted capacities of both of Erie's WWTPs are below 2 mgd which make them eligible for delayed implementation. The earliest either the SWRF or NWRD will have to meet the requirements under Regulation 85 is May 31, 2022.

The Wastewater Utility Plan will move through the approval processes at North Front Range Water Quality Planning Association (NFRWQPA) beginning in mid-2011. Final approval of the proposed 208

Boundary Change and Wastewater Utility Plan is expected in 2012. The approved WUP will then be submitted to the State.

SUMMARY OF DIFFERENCES BETWEEN WUP AND NFRQWPA / DRCOG OUTLINES

The preferred utility plan checklists vary slightly between North Front Range Water Quality Planning Association (NFRWQPA) and the Denver Regional Council of Governments (DRCOG). In places where sections were preferred for one plan, but not the other, those sections were included as specified by their respective plans. During the review stages of this WUP, DRCOG abdicated its authority to review WUPs to the State Water Quality Control Division.

The Wastewater Characterization section has been divided into three distinct sections. Section 4 of the utility plan includes influent flow and characteristics information, population projections, and projected future flows and loads. Section 4 describes all of the influent flow and load characteristics at the beginning of the section followed by future flow and load projections rather than estimating future flows at the beginning of the section. Section 5 includes design information for the existing South Water Reclamation Facility (SWRF) and the new North Water Reclamation Facility (NWRF). Section 6 details the collection system and collection system modeling effort.

SECTION 3
GENERAL PLANNING

The Town of Erie is located southeast of Boulder and north of the Denver Metropolitan Area. The Town has two wastewater reclamation facilities. The South Water Reclamation Facility (SWRF) is located on S. Briggs Street close to the older part of the Town. It was upgraded from a lagoon to a mechanical facility in 1998. The North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The Town has experienced rapid population growth over the last ten years, nearly doubling in size to approximately 18,000 residents.

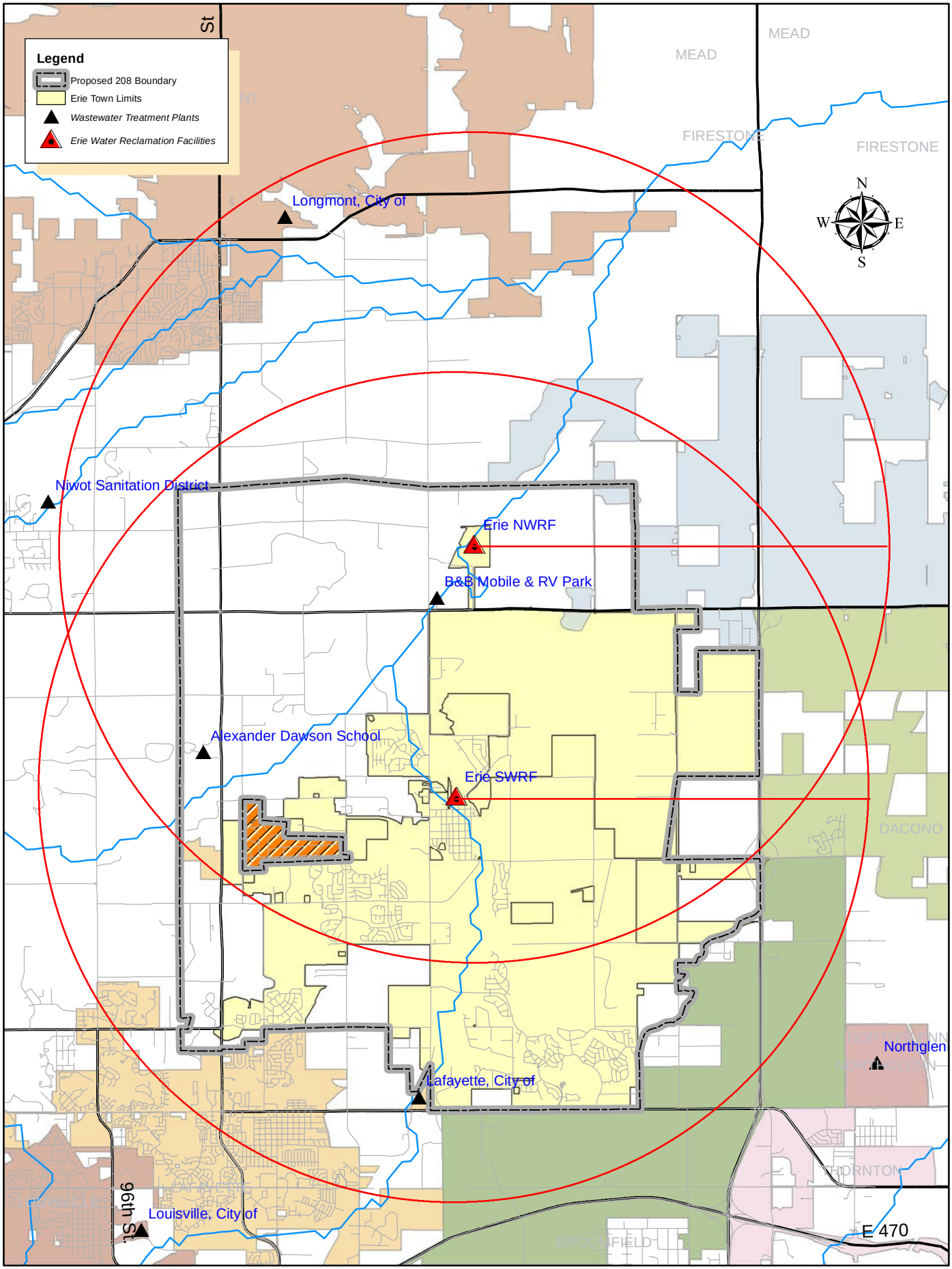
CONSOLIDATION OF FACILITIES

There are multiple wastewater treatment plants located within five miles of Erie's wastewater treatment facilities including: The City of Lafayette Water Reclamation Plant (WRP), Alexander Dawson School, B&B Mobile & RV Park, and the City of Longmont WWTP. Nearby treatment facilities are shown in Figure 3-1. The WWTFs at the B&B Mobile & RV Park and the Alexander Dawson School process only a few thousand gallons of flow each day and are not capable of absorbing flows from the Town of Erie. These smaller facilities may be served by the Erie NWRF in the future provided their wastewater infrastructure meets or exceeds Erie's construction standards and the system owners supply the necessary infrastructure for making the connection and treatment capacity is available. System investment (tap) and service fees would apply to each connection.

Lafayette's WRP is located at the extreme southern edge of Erie's service area and is uphill from the entire service area. To transfer flows to this location would require a large lift station and a ten-mile long force-main. Force mains and lift stations are prone to failure. A long force main could leak into the environment for an extended period of time undetected and cause serious environmental damage. A large lift station would cost roughly one million dollars to construct. The force main would be even more costly. At a planning cost of $120 per linear foot of force main, the estimated cost would be $6.3 million dollars. This does not include costs for acquiring the necessary easements or system investment (tap) fees. Lafayette's tap fees for single family homes within their service area were $5,300 in 2010 (City of Lafayette, Colorado 2010). Assuming an in-service area rate, the estimated cost for tap fees would be in excess of $122 million[1]. The Lafayette WRP has a permitted capacity of 4.4 MGD. Current average daily flows are near 2.0 MGD. To accommodate the additional flows from Erie, the Lafayette WRP would have to significantly increase its capacity as well as improve its ability to meet nutrient limits. Consolidation with Lafayette is not a viable alternative.

Longmont's wastewater treatment plant is located northwest of Erie's service area. It has a permitted capacity of 17 MGD and an average daily flow of 8 MGD. The Longmont WWTP has enough excess capacity to comfortably absorb wastewater flows generated within Erie's service area. There are two large ridges between the NWRF and the Longmont WWTP. Each ridge has an elevation gain of more

[1] Tap fees assume a buildout population of 68,820 residents at three persons per household.



Circles show 5-mile radius from each reclamation facility.

Figure 3-1 Wastewater Treatment Plants Near Town of Erie

than 100 feet. Consolidation with Longmont would require a minimum of two lift stations and approximately four miles of force main. The estimated cost for consolidation, not including easement acquisition or system investment fees, would be $5.2 million dollars. Consolidation with Longmont has the same environmental concerns of long force mains and lift stations. Longmont's tap fees for single family homes within their service area were $2,940 in 2010. Assuming an in-service area rate, the estimated cost for tap fees would be in excess of $67 million. Consolidation with Longmont is not a viable alternative.

The next closest treatment facility is the Saint Vrain Sanitation District WWTF. Wastewater generated within Erie could potentially be conveyed by gravity to the Saint Vrain SD WWTF by following the natural drainage created by Boulder Creek. Consolidation with the Saint Vrain SD WWTF would require a 9.8 mile long gravity sewer. At an estimated cost of $120 per linear foot, consolidation is estimated at $6.2 million. This cost does not include easement acquisition or system investment fees.

As of October 2009, the Town of Erie had 6,123 wastewater taps including light commercial and schools. According to the St. Vrain Sanitation District website, system investment fees are $5,050 per single family equivalent. System investment fees for commercial and industrial taps are based on the size of the water meter and range from $7,575 for a 5/8-inch meter up to $80,800 for a 3-inch meter. Investment fees are slated to increase in January 2011 (Saint Vrain Sanitation District 2010). Assuming all taps are residential taps, the plant investment fees associated with consolidation would be $30.92 million if it were done in 2010. This estimate is low as it does not include higher fees for commercial accounts. The total cost of tap fees, assuming a buildout population of 68,820 and three residents per household, would exceed $115 million.

The environmental concerns associated with pump stations and force mains are not an issue with this alternative. However, the Saint Vrain SD WWTF is only permitted for 3.0 MGD and does not have the capacity to absorb Erie's flows. It would need to be expanded significantly. Consolidation with Saint Vrain Sanitation District is not a viable alternative.

In addition to the considerable technical challenges and financial costs associated with consolidation, there is also the issue of water rights. Erie is able to reclaim and reuse a substantial portion of its treated effluent. This reduces demand for raw and potable supplies. If Erie consolidated, the reuse water would be lost to the Town. For these reasons, consolidation is not a viable alternative.

WASTEWATER REUSE

The Town of Erie has developed a diverse water portfolio to meet its needs under a variety of conditions and prides itself on being stewards for water wise practices. Erie uses both potable and non-potable water in its water management plan, with non-potable supplies consisting of raw or untreated water and reclaimed treated wastewater effluent. Non-potable water effectively reduces the demand on Erie's potable water system and can be used for irrigation in parks, as well as for landscape irrigation in commercial, industrial, and multi-family settings. Non-potable water use also promotes water sustainability through the efficient use and reuse of a valuable resource. The most recent update of Erie's

Non-Potable Municipal Water System Master Plan was completed by CDM in September 2007 and can be found in Appendix B. The following is a summary of that plan.

Water Rights and Water Supply Sources

The water supply inventory for the Town of Erie is a collection of sources including transbasin sources, such as Colorado-Big Thompson (CBT) units and Windy Gap Project (Windy Gap) units; reservoirs, such as Thomas, Erie, and Prince; as well as Mutual Irrigation Company shares such as, Leyner Cottonwood Ditch, South Boulder Canyon Ditch, Erie Coal Creek Ditch and Reservoir Company, and FRICO-Marshall Lake Division. Erie's current water rights portfolio provides an approximate yield of 8,260 acre-feet per year (afy.) Table 3-1 shows Erie's existing water rights as of January 2010. Erie is currently able to recapture and reuse water from Windy Gap shares.

Erie also has 1.9 cubic feet per second of Coal Creek junior water right, although adjudication is still pending. Most of Erie's water supply comes from the CBT and the Windy Gap project. As Erie continues to grow and develop, additional water rights will be needed to meet future demands. Erie is a participant of two significant Front Range coalitions working on securing future water needs, the Northern Integrated Water Supply Project (NISP) and The Windy Gap Firming Project.

The Northern Integrated Water Supply Project (NISP), sponsored by the Northern Colorado Water Conservancy District, is a regional cooperative water supply project. Its goal is to provide participating water providers with approximately 40,000 acre-feet of new, reliable water supply. NISP includes construction of two new reservoirs: Glade and Galeton Reservoirs. Glade reservoir will be located northwest of Fort Collins and north of Horsetooth Reservoir. It will have 170,000 acre-feet of storage capacity (Northern Integrated Supply Project n.d.). Glade Reservoir will be filled by diverting water from the Poudre River using the already existing Poudre Valley Canal. There will be no new structures on the river. Galeton Reservoir will be located east of Ault and northeast of Greeley. It will hold about 40,000 acre-feet of water at full capacity (Northern Integrated Supply Project n.d.). Galeton Reservoir will be filled by diverting water from the South Platte River downstream from Greeley. Galeton water will be delivered to two agricultural irrigation companies in exchange for the Poudre River water they currently use. Erie has requested 6,000 afy of firm yield, although Erie could increase its NISP request to 6,500 afy of firm yield. Water obtained through NISP may be recaptured and reused. The original NISP delivery has fallen behind schedule due to technical and political uncertainty, although public interest remains high. It is estimated to be delivered sometime around 2020.

The Windy Gap Project near Granby, CO, diverts water from the Colorado River to the Front Range via the Colorado-Big Thompson Project (CBT) on a space-available basis. During wet years when water is available for Windy Gap diversions, Lake Granby is often full with little or no space for the water. During dry years, the water right can be too junior to come into priority, so no water is available to pump. The Windy Gap Firming Project was proposed to store available water in a new Front Range reservoir to ensure reliable or "firm" future deliveries. The Windy Gap Firming Project will enable Erie to receive its full Windy Gap allotment, which is currently not always possible. Erie also plans to purchase additional CBT shares, ditch and reservoir shares, and Windy Gap shares, as well as extend its supplies by using

non-potable raw and reclaimed water. The availability of non-potable supplies depends on the hydrology of a given year, as well as how and when the shares are used.

Table 3-1 Existing Water Rights, April 27, 2010

Water Right Name	No. of shares or Units Owned	Avg. Annual Yield[1]	Annual Yield[2]	Dry Year Annual Yield[1]	Dry Year Annual Yield[2]	Notes and Comments
Transbasin Sources						
CBT Project						
Total units owned	4680					
Erie Financial Corp. (Lease/Purchase)	2639					
CBT - Variable quota	7,319	0.7	5123.3	1.0	7319	
Windy Gap Project	14	100	1400	0	0	This resource can be recaptured and used to extinction, although table shows amount available at first use
Reservoir Storage						
Erie Reservoir[4]	239	1.0	239	0.3	71.7	
Prince Reservoir[4]	80	1.0	80	0.3	24.0	physical capacity = 213 ac-ft
Thomas Reservoir[4]	148	1.0	148	0	0	physical capacity = 200 ac-ft
Mutual Irrigation Company Ownership						
Leyner Cottonwood Ditch	311.5	0.54	168.2	0.21	65.4	Available during irrigation season
South Boulder Canyon Ditch	203	2.9	588.7	0	0.00	Available during irrigation season (610 shares total)
Erie Coal Creek Ditch and Res. Co.	98	4.9	480.2	0.56	54.9	Available during irrigation season
FRICO – Marshall Lake Div.	8.24	4	33	0.5	4.1	Available during irrigation season
Total			8,260.4		7,539.1	

[1]acre feet per share
[2]acre feet total
[3]One acre-foot equals 325,851 gallons
[4]CBT water may be stored in Erie, Prince and Thomas Reservoirs.

Non-potable and Raw Water Supplies

Erie plans to use non-potable water to meet a portion of its irrigation and other non-potable water demands. Potential non-potable supplies include raw ditch water, raw water that would otherwise be treated for potable use (e.g., CBT or Windy Gap), reservoir water, and treated wastewater return flows.

Erie's ditch and reservoir company shares turn out approximately 1,733 AFY in an average year and 219 AFY in a dry year (Table 3-1). Raw ditch and reservoir company shares are preferred for irrigation use over using treated potable water because ditch and reservoir company shares used for irrigation may not

need to be converted to municipal use. Value may be lost in the conversion process. Secondly, ditch water doesn't require treatment. Unlike reuse of reclaimed treated wastewater effluent, ditch water is not subject to Regulation 84 – Reclaimed Water Control Regulation (Colorado Department of Public Health and Environment Water Quality Control Commission 2007).

There are disadvantages to using ditch water, such as variability in yield and the need for storage, higher conveyance losses, and gravity as opposed to a pressurized delivery system which restricts the potential service area. Erie could also use reservoir, raw CBT, or Windy Gap water for non-potable irrigation. CBT and Windy Gap water could either be directly delivered to non-potable customers or stored in Thomas Reservoir for later use. Ditch and reservoir company shares owned by Erie and legally reusable treated wastewater flows are currently not enough to meet all future non-potable demands. Raw CBT, Windy Gap, and reservoir water could be used to meet the remaining portion of Erie's non-potable demands.

Reuse of Windy Gap Return Flows

The return flows from the use of Windy Gap water in the potable water system may be recaptured after treatment at one of Erie's two wastewater treatment facilities (WWTFs). Windy Gap water may be used to extinction. These return flows may either be diverted and directly conveyed to non-potable demands or be temporarily stored in the 1,000 acre-foot storage facility currently under construction at the NWRF. Reusable return flows from the SWRF that do not coincide with seasonal non-potable demands can be conveyed via Coal Creek and Boulder Creek to the NWRF storage facility. Because winter water use is primarily non-consumptive indoor use, Erie can maximize the capture of reusable Windy Gap effluent by using the majority of its Windy Gap water for potable use in the winter. Erie could then store the reusable effluent at the NWRF reservoir and later use it to meet summer irrigation demands. Windy Gap shares not used during the winter could be used for potable supplies in the summer and recaptured as treated effluent at the WWTFs for direct reuse. As previously noted, Erie may acquire additional Windy Gap shares, thus increasing the amount of water that Erie could recapture for non-potable reuse. The Windy Gap Firming Project must be implemented for treated effluent to be reliably available during dry years.

Water obtained through the NISP may also be recaptured and reused. Erie anticipates having 6,000 acre-feet per year available for capture through NISP plus 1,400 acre-feet per year available through Windy Gap. All of Erie's other existing water rights are limited to a one-time use.

Transmission and Treatment Systems

Prince, Erie, and Thomas Reservoirs, located in the southwestern portion of the service area, store much of Erie's raw water. The water is then piped to Lynn R. Morgan Water Treatment Facility (WTF) and to and from Thomas Reservoir. Raw CBT and Windy Gap water are conveyed to Erie from Carter Lake via a pipeline to WTF. The water is distributed from the WTF to residential and commercial customers. The Southern Water Reclamation Facility (SWRF) near Old Town currently treats Erie's wastewater and has a capacity of 1.6 mgd. Treated effluent is discharged into Coal Creek. The Northern Water Reclamation Facility (NWRF) broke ground November 9th, 2009, was completed in 2010, and became operational in

early 2011. Treated effluent is discharged to South Boulder Creek. The NWRF includes a Reuse Pump Station and 1,000 acre-foot reclaimed water storage reservoir. Reuse water may be pumped back to the SWRF through a reclaimed water pipeline.

Non-potable Transmission Systems

Erie is currently using its shares in the Leyner-Cottonwood Ditch and shares from the Erie Coal Creek Ditch and Reservoir Company to irrigate parks and open space in the Erie Commons development. Non-potable water demands associated with irrigation in the Erie Commons development will increase as additional landscaping and recreational facilities are developed. Non-potable water is also used to irrigate the Vista Ridge golf course. This non-potable water supply currently consists of Windy Gap water, Coal Creek raw water, leased Marshall Lake (Community Ditch) shares, and leased treated effluent from Superior and Louisville delivered via Coal Creek.

A raw water conveyance system conveys non-potable supplies owned by Erie from Thomas Reservoir to Erie Commons via the Leyner-Cottonwood Ditch. This system consists of two raw water supply lines, a non-potable storage facility, and the Leyner-Cottonwood Ditch. Raw water is transported from Thomas Reservoir through a raw water pipeline to Leyner-Cottonwood Ditch and then diverted east of County Line Road to a raw water pipeline that conveys the water to a non-potable storage site east of the Erie Commons development.

Raw water could potentially be conveyed through existing raw water pipelines and ditches. Erie's raw ditch water shares could be conveyed via the Leyner-Cottonwood Ditch, South Boulder Canyon Ditch, Erie Coal Creek Ditch, and Community Ditch. It may be possible to convey other non-potable supplies, such as reclaimed effluent, via the ditch system to non-potable demand points if agreements could be negotiated between Erie and the ditch companies.

APPLICABLE COMPLIANCE SCHEDULES OR CONSENT DECREES

The Town received a new discharge permit for the South Water Reclamation Facility (SWRF) in September 2011. The new permit became effective on October 1, 2011. The permit contains compliance schedules for installing temperature monitoring equipment and for meeting new effluent ammonia limits. The SWRF must be able to meet the permitted ammonia and nitrate limits by April 30, 2017.

ENVIRONMENTAL COMPONENTS EVALUATION

The following environmental components descriptions apply to the Town of Erie's planning area and to the SWRF and NWRF site envelopes. A thorough summary of various environmental components is an essential part of acquiring grant funding and loans from State and Federal sources.

Ecoregions

Ecoregions denote areas of general similarity in ecosystems and in the type, quality, and quantity of environmental resources. A Roman numeral hierarchical scheme has been adopted for different levels of ecological regions. Level I is the coarsest level, dividing North America into 15 regions. Level II divides

the continent into 52 regions. At level III, the continental United States contains 104 ecoregions. The Town is located in two ecoregions, Level II 25l: Front Range Fans and 25d: Flat to Rolling Plains (Chapman, et al. 2006). Erie's planning area is in Ecoregion 25 – High Plains – and includes two different subecoregions: 25l – Front Range Fans and 25d – Flat to Rolling Plains.

EPA describes the High Plains Ecoregion as being "comprised [of] smooth to slightly irregular plains having a high percentage of cropland. Grama-buffalo grass is the potential natural vegetation in this region as compared to mostly wheatgrass and needle grass to the north, Trans-Pecos shrub savanna to the south, and taller grasses to the east. The northern boundary of this ecological region is also the approximate northern limit of winter wheat and sorghum and the southern limit of spring wheat. In Colorado, gas and oil fields are scattered throughout the region, with the greatest concentration found in the Denver Basin area" (Chapman *et. al* 2006).

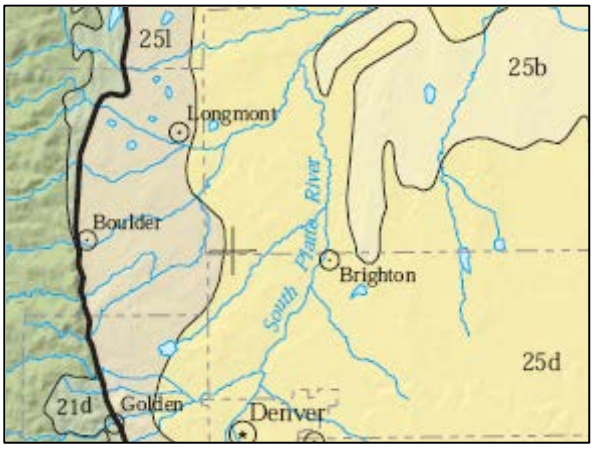


Figure 3-2 Ecoregions

The *Front Range Fans* ecoregion (25l) is described as "flanking the northern Front Range of the Southern Rockies in Colorado. Streams tend to be cooler than in other High Plains regions and contain many Front Range aquatic species. The soils of the region have more outwash gravels than regions farther east and occupy old terraces, benches, and alluvial fans. The soils are formed from materials weathered from arkosic sedimentary rock, gravelly alluvium, and redbed shales and sandstone. Some soils have a high shrink-swell potential. Land use is changing from mostly cropland and rangeland to more extensive urban development. Development has led to an increase in manmade lakes and gravel pits dotting the region" (Chapman *et. al* 2006).

The *Flat to Rolling Plains* ecoregion (25d) is described as "more level and less dissected than the adjacent Moderate Relief Plains (ecoregion 25c). Soils are generally silty with a veneer of loess. Dryland farming is extensive, with areas of irrigated cropland scattered throughout the ecoregion. Winter wheat is the main cash crop, with a smaller acreage in forage crops" (Chapman *et. al* 2006).

The EPA developed the Ecoregion concept as a way to more effectively manage water bodies. Currently, rivers are regulated by segment rather than as a cohesive ecosystem. As rivers cross State borders, regulations and water quality standards can change significantly. Under the Ecoregion umbrella, rivers in a particular area are expected to meet specific criteria for nutrients and other parameters. Essentially, EPA is setting nutrient criteria for impacted water bodies to ensure that the concentrations of nutrients in those water bodies don't exceed the typical background concentrations for that ecoregion. Suggested water quality goals for Ecoregion 25 are given in Table 3-2. The goals represent the 25th percentile concentrations for nutrients for unimpacted lakes, reservoirs, rivers, and streams within ecoregion 25.

The implication is that nutrient levels measured in streams in the top 75th percentile are excessive and potentially damaging to designated uses.

Although EPA has been working on the ecoregion concept for almost twenty years, the proposed ecoregion nutrient criteria have not been enforced until recently. The Florida Wildlife Federation (FWF) filed a lawsuit in 2008 seeking to require EPA to promulgate numeric water quality standards for Florida waters. Florida had been in the process of developing its own nutrient criteria for many years, but due to the wide variety of soil types and ecosystems present, had fallen years behind schedule. Florida was relying on narrative water quality standards in the interim. EPA determined that Florida's narrative criteria were insufficient to protect water quality and that issuing water quality standards was necessary to meet the requirements of the Clean Water Act (Environmental Protection Agency 2010).

Table 3-2 Suggested Water Quality Criteria Goals for Ecoregion 25

Parameter	Lakes and Reservoirs	Rivers and Streams
Total Phosphorus, mg/L	0.033	0.067
Total Nitrogen, mg/L	0.56	0.88
Chlorophyll a, ug/L	2.3	3
Secchi, m	1.3	NA
Turbidity, FTU/NTU	NA	7.83

In early 2009, EPA sent a letter to the Florida Department of Environmental Protection (FDEP) proposing draft nutrient criteria for that state. In August 2009, EPA entered into a Consent Decree with FWF to settle the 2008 litigation. The Consent Decree committed EPA to proposing numeric nutrient criteria for lakes and flowing waters by January 2010. Final standards must be set by October 2010 for lakes and flowing waters and by October 2011 for estuarine and coastal waters (Environmental Protection Agency 2010). Although there are legal and procedural challenges in process, the success of the Florida lawsuit sets a legal precedent that may be repeated in other States. The State of Wisconsin is currently being sued by environmental groups over its water quality standards.

On May 14, 2012, the Water Quality Control Commission adopted changes to Regulation 31 to incorporate nutrient criteria into the basic water quality standards. The water quality based nutrient standards are presented in Table 3-3. Colorado's water quality based nutrient criteria differ slightly from EPA's ecoregion criteria and reflect both cold and warm water habitats. Early adoption of site-specific criteria may help Colorado to avoid lawsuits like the ones brought in Florida and Wisconsin. The water quality standards must be adopted through the regular basin hearing process before they can be incorporated into discharge permits. The first possible adoption will be the South Platte River Basin in 2017. Basin hearings will continue through 2022.

Both EPA's and Colorado's proposed water quality standards for nutrients are lower than the practical treatment limits for most wastewater treatment technologies. The limit of technology for total nitrogen removal has been cited as 5 mg/L as N (Jeyanayagam 2005) and more recently as 3 mg/L as N (Cadmus Group, Inc. August 2010). Removal mechanisms and limits for nitrogen and phosphorus species are presented in Table 3-4. To move beyond the limits of technology, treatment facilities that cannot rely on

dilution by the receiving water will have to resort to extraordinary measures such as reverse osmosis to meet permit limits.

Table 3-3 Colorado's Nutrient Water Quality Standards

Parameter	Lakes and Reservoirs > 25 Acres		Rivers and Streams	
	Cold	Warm	Cold	Warm
Total Phosphorus, mg/L	0.025	0.083	0.110	0.160
Total Nitrogen, mg/L	0.426	0.910	1.250	2.010
Chlorophyll a, ug/L	8	20	N/A	N/A
Chlorophyll a, mg/m^{2*}	N/A	N/A	150	150

Source: Regulation 31 Nutrient and Chlorophyll Interim Values
*mg/m^2 chlorophyll of attached algae, not to exceed.
Note: Direct Use Water Supply Lakes and Reservoirs have a chlorophyll α limit of 5 ug/L.

Table 3-4 Technological Limits for Biological Nutrient Removal

Form	Common Removal Mechanism	Technology Limit, mg/L
Total Nitrogen		
Ammonia-N	Nitrification	<0.5
Nitrate-N	Denitrification	1 - 2
Particulate Organic-N	Solids Separation	0.5 – 1.5
Soluble Organic-N (non-biodegradable)	None	0.5 – 1.5
Total Phosphorus		
Soluble P	Microbial uptake and/or chemical precipitation	0.1
Particulate	Solids Removal	<0.05

Source: (Jeyanayagam 2005)

The TN in reclaimed water consists of inorganic nitrogen (ammonia and nitrates) and organic nitrogen. The ammonia and nitrates can be removed down to low levels by conventional BNR processes – nitrification and denitrification with supplemental carbon (e.g. methanol) if necessary. These systems routinely remove total nitrogen down to levels of 5 mg/L (Mulholland 2007). The presence in reclaimed water of soluble nitrogen and phosphorus compounds that are not biodegradable (also known as refractory compounds) ultimately sets the lowest concentrations possible at treatment plants that rely on biological treatment methods. Researchers call the refractory dissolved organic nitrogen (RDON). RDON is present in many raw water supplies and thus is present in the influent to water reclamation facilities. RDON is also a byproduct of biological treatment and will be found in reclaimed water even if not found in raw sewage. Recent studies suggest that effluent organic nitrogen is typically around 1 mg/L (Mulholland 2007).

The EPA recognizes that conventional treatment technologies capable of removing nitrogen down to the proposed nutrient criteria limits (Total Nitrogen <1.3 mg/L) do not exist today and that attempting to meet these criteria with exotic, add-on technologies such as reverse osmosis will be very costly. EPA is considering allowing variances in treatment standards based on a community's financial ability to fund WWTF improvements. It has been proposed that, if a community's average monthly residential user fee for sewer service is at or above 1 to 2 percent of the median household income, the community will not be

expected to make improvements to meet nutrient criteria. Erie's mean household income is $87,284 (Town of Erie 2010). Before EPA would consider a variance, monthly sewer fees would need to be between $72 and $144 per month. Monthly service fees were $39 per month in 2010. If a variance were allowed, it would result in a temporary water quality standard for the Town for up to twenty years.

The State of Colorado Water Quality Control Division also recognizes the limitations of conventional treatment and potential costs associated with trying to meet the proposed nutrient criteria at the final effluent. The Commission also adopted a new regulation, Regulation 85 – Nutrients Management Control, which sets limits for total inorganic nitrogen (TIN) and total phosphorus (TP) in discharge permits. The TIN and TP limits shown in Table 3-5 will be implemented during normally scheduled permit reviews.

Table 3-5 Regulation 85 Nutrients Management Control Discharge Limits

Category	Parameter	Annual Median[1], mg/L	95th Percentile[2], mg/L
Existing Dischargers	Total Phosphorus (as P)	1.0	2.5
	Total Nitrogen (as N)	15	20
New Dischargers	Total Phosphorus (as P)	0.7	1.75
	Total Nitrogen (as N)	7	14

[1]The annual median is defined as the median of all samples taken in the most recent 12 calendar months.
[2]The 95th percentile is defined as the 95th percentile of all samples taken in the most recent 12 calendar months.
[3]Existing dischargers are defined as treatment facilities discharging prior to May 31, 2012 or that have submitted a request for preliminary effluent limits prior to that date.

Dischargers that meet one of the following criteria are exempt from the effluent nutrient limits: 1) lagoon facilities that have rated capacities of 1 mgd or less, 2) facilities owned by disadvantaged communities, and 3) facilities with rated capacities of 0.5 mgd or less. Deferrals for delayed implementation are available for treatment facilities with capacities between 1 and 2 mgd. The permitted capacities of both of Erie's WWTPs are below 2 mgd which make them eligible for delayed implementation. The earliest either the SWRF or NWRF will have to meet the requirements under Regulation 85 is May 31, 2022.

Finally, upgrades to existing facilities may not have to be done if the cost of improvements isn't reasonable with respect to the environmental benefit gained. Regulation 85 contains a provision that may be used by permittees to cap expenditures on nutrient control technology when the nutrient reduction benefits do not bear a *reasonable relationship* to the economic, environmental, or energy impacts resulting from meeting those effluent limitations. The reasonable relationship calculation takes into account the percentage of nutrients in a receiving water coming from permitted sources, the cost of mitigating the nutrient load, and the median household income of the permittee's service area. In the case of the SWRF which discharges into an effluent dominated stream, it is likely that more than 50 percent of the incremental nutrient load is from point sources. For this category, there can be more than a 10 percent change in the level of profitability for the permittee and the annual treatment cost may be more than 2 percent of the median household income. For the SWRF, monthly sewer fees would need to be higher than $144 per month before the State would consider an economic variance.

The effluent limits set forth in Regulation 85 can be met using conventional treatment technologies. Phosphorus may be removed either biologically with an anaerobic zone in activated sludge processes or with chemical precipitation using iron or alum salts. Tertiary filtration may not be necessary. Total

inorganic nitrogen limits can be met using biological nitrification and denitrification. The NWRF is currently capable of meeting requirements for both phosphorus and nitrogen. The SWRF will require capital improvements to meet these treatment goals.

Regulation 85 includes provisions for nutrient trading from a nonpoint or point source to a point source on a stream segment or watershed basis where the WQCD has determined that the trade achieves a net water quality or environmental benefit and does not cause adverse localized impacts. The NWRF is designed to remove nitrogen and phosphorus to low levels. The Town of Erie could potentially trade nutrient credits between the NWRF and SWRF which could make upgrades to the SWRF unnecessary.

All domestic wastewater facilities are required to begin monitoring for nutrients no later than March 1, 2013. Minor dischargers will generally be required to sample every other month and major dischargers will generally be required to sample monthly. Samples must be collected upstream of the outfall, from the discharge, and downstream of the outfall at the nearest USGS gage station where one is available. Monitoring is designed to assess the effectiveness of Regulation 85 and to help determine whether the water quality based nutrient standards described in Regulation 31 need to be implemented. If in-stream water quality standards are met solely by Regulation 85 effluent limitations, more stringent limits calculated using the water quality based standards and in-stream dilution may not be necessary.

Climate

Climate data for Erie, Colorado is presented in Table 3-6. Climate data was obtained from the Western Regional Climate Center. The Longmont 2 ESE, Colorado (055116) weather station was selected as the most representative weather station in the general area and with the longest period of record. It has operated continuously from January 1, 1893 to November 30, 2004.

Erie has a moderate climate with winter daytime temperatures averaging between 44 and 53 degrees Fahrenheit (6.7 to 11.7 degrees Celsius). Winter evening temperatures average between 14 and 23 degrees Fahrenheit (-10 to -5 degrees Celsius). Summer daytime temperatures average between 72 and 88 degrees Fahrenheit (20 to 31 degrees Celsius) with evening temperatures dropping into the fifties. Like most of the Front Range, Erie receives an average of 13.5 inches of precipitation each year.

Table 3-6 Climate Data for the Town of Erie, Years 1893 to 2004

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	43	46	53	62	71.6	82	88	87	78	67	52	44	64.5
Average Min. Temperature (F)	12	16	23	32	41.7	50	55	53	44	33	22	14	32.7
Average Total Precipitation (in.)	0.4	0.5	1	1.7	2.45	1.7	1.4	1.2	1.2	1	0.6	0.5	13.53
Average Total Snowfall (in.)	4.8	5.5	7.1	4.9	0.7	0	0	0	0.5	1.8	5.3	5.3	35.9
Average Snow Depth (in.)	1	1	1	0	0	0	0	0	0	0	0	1	0

Source: http://www.wrcc.dri.edu/cgi-bin/cliMAIN.pl?co5116, Station Longmont 2 ESE, Colorado (055116). Period of record is January 1, 1893 to November 30, 2004 (Western Regional Climate Center 2009)

Topography

The topography of the service area is defined by the Boulder Creek and Coal Creek drainages. Boulder Creek runs northeast across the Town as shown in Figure 3-3. Erie's service area ranges in elevation from 5,200 feet down to 4,920 feet; a span of 280 feet. In general, the service area slopes from southeast to northwest towards the creeks.

Underlying Geology

The underlying geology of Erie's planning area is dominated by the historic stream bed and flood plains for Boulder Creek and Coal Creek. The underlying geology consists primarily of alluvial soils and eolium. Alluvial soils are deposited by water and are characterized by sand, pebbles, and loose cobbles. Eolium consists of windblown clay, silt, and granules and tends to be light- brown to reddish-brown to olive-gray in color. The majority of the planning area is underlain by Eolium (Qe) which is shown in light tan in Figure 3-4 (Colton 2003).

Pinery Creek Alluvium (Qp) of Holocene age, shown in bright yellow in Figure 3-4, is present along Boulder Creek and Coal Creek. Piney creek alluvium is dark-grey humic sandy to gravelly alluvium containing organic matter. This type of alluvium tends to be underlie terraces whose surfaces are nine to eighteen feet above a nearby flood plain (Colton 2003). Eolium Sand (Qes) and Broadway Alluvium (Qb) are in the surrounding areas of the site. These strata are clay to gravel sized soils that have been water or wind deposited. The bedrock under this area is the Larimer Formation (KI) to the southeast and the Fox Hill Sandstone to the northwest.

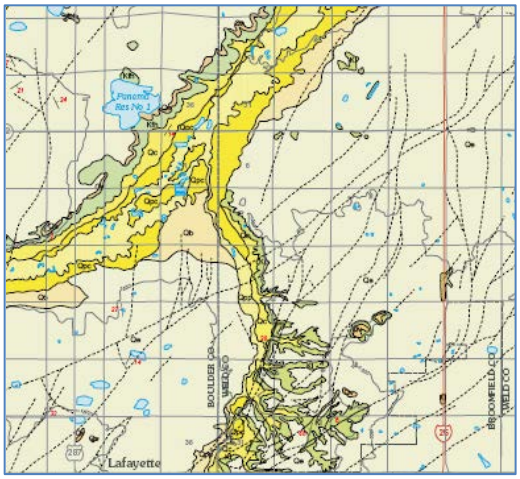


Figure 3-4 Underlying Geology

Coal Mining

The original plat for Erie was filed in 1871, following the establishment of the Briggs Coal Mine. The Briggs Coal Mine was the first commercial coal mine in Weld County. The Boulder-Weld coal field is located north and northwest of the Denver metropolitan area. The coal field extends for some twenty to twenty-five miles from Marshall in the southwestern part of the coal field to areas just north and east of the tri-cities metroplex of Dacono, Frederick, and Firestone. The coal field is mostly within Boulder and Weld Counties, and includes parts of the Niwot, Erie, Frederick, Gowanda, Louisville, Lafayette, and Eastlake 7.5' quadrangles (S.B. Roberts 2001). Figure 3-5 shows where coal mining has taken place in the portion of the coal field surrounding Erie.

Mining in the coal field began in the early 1860's in the Marshall area and continued into the 1970's with the last mine (Lincoln Mine) closing in 1979 because of fire. The extent of the coal mines shown on the map in Figure 3-5 does not include areas where coal was "poached" from beyond a mine's boundaries. In some areas, mining may have been more extensive than documented in historic records.

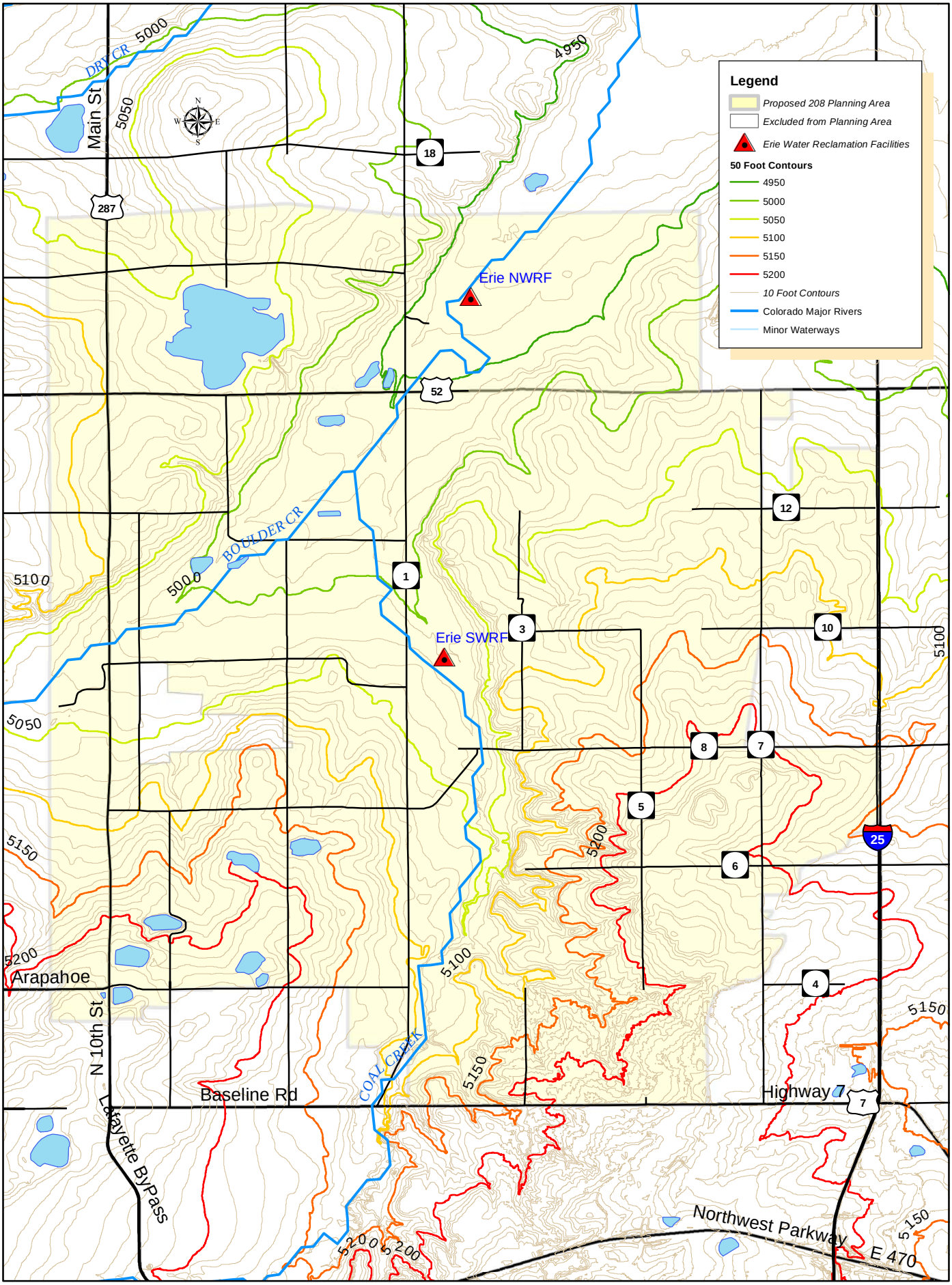


Figure 3-3 Service Area Topography

Historic coal bed mining was confined to the lower 265 feet of the Laramie formation (S.B. Roberts 2001). As many as sixteen individual coal beds may be locally present. Coal beds or seams range in thickness from three to fourteen feet. Seams lie in layers that are separated by layers of rock as thick as one-hundred feet.

There are no known coal mines under either the SWRF or NWRF sites.

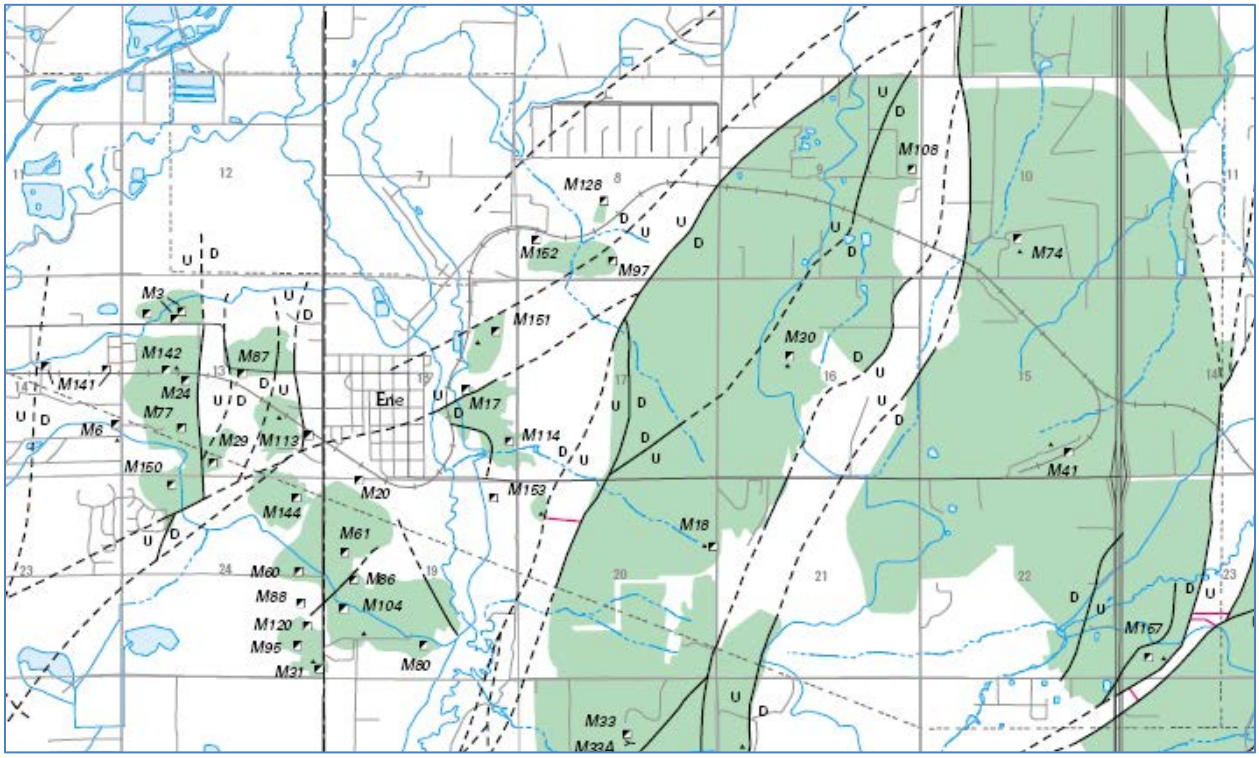


Figure 3-5 Boulder-Weld County Underground Coal Mines Near Erie, Colorado
Source: (S.B. Roberts 2001) Small boxes indicate mine shafts.

Local Soils

The local soil types were mapped using the National Resources Conservation Service Web Soil Survey. Soil types are classified according to the American Association of State Highway and Transportation Officials (AASHTO) system. The AASHTO system was originally developed to classify soils based on suitability for highway and airfield construction.

South Water Reclamation Facility. The SWRF is constructed on soil type 4 – Aquolls and Aquepts, flooded. Soil types are mapped in Appendix C and described in Table 3-7. Normally found more than 80 inches below the surface, Aquolls and Aquepts soils drain poorly can be found on drainage ways, plains and depressions. Its parent material is recent alluvium. Slopes are between 0 to 3 percent. Aquolls and Aquepts soils are found at elevations of 3,600 to 4, 700 feet.

A typical profile is varied from 0 to 8 inches, while 8 to 60 inches consist of stratified sandy loam to clay. It has a high frequency of flooding with no ponding potential. The maximum calcium carbonate content is 10 percent, and has a water capacity of 4.7 inches.

Table 3-7 Local Soil Types at SWRF Site by AASHTO Group Classification

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
3	Aquolls and Aquents, gravelly substratum	12.7	20.40%
4	Aquolls and Aquepts, flooded	33	53.30%
85	Water	16.3	26.30%
	Totals for Area of Interest	62	100.00%

North Water Reclamation Facility. The NWRF is also constructed on soil type 4 – Aquolls and Aquepts, flooded. Soil types are mapped in Appendix C and described in Table 3-8. Normally found more than 80 inches below the surface, Aquolls and Aquepts soils drain poorly can be found on drainage ways, plains and depressions. Its parent material is recent alluvium. Slopes are between 0 to 3 percent. Aquolls and Aquepts soils are found at elevations of 3,600 to 4,700 feet.

Table 3-8 Local Soil Types at NWRF Site by AASHTO Group Classification

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
4	Aquolls and Aquepts, flooded	20.4	99.00%
20	Colombo clay loam, 1 to 3 percent slopes	0.2	1.00%
	Totals for Area of Interest	20.6	100.00%

A typical profile is varied from 0 to 8 inches, while 8 to 60 inches consist of stratified sandy loam to clay. It has a high frequency of flooding with no ponding potential. The maximum calcium carbonate content is 10 percent, and has a water capacity of 4.7 inches.

Site Specific Geotechnical Data

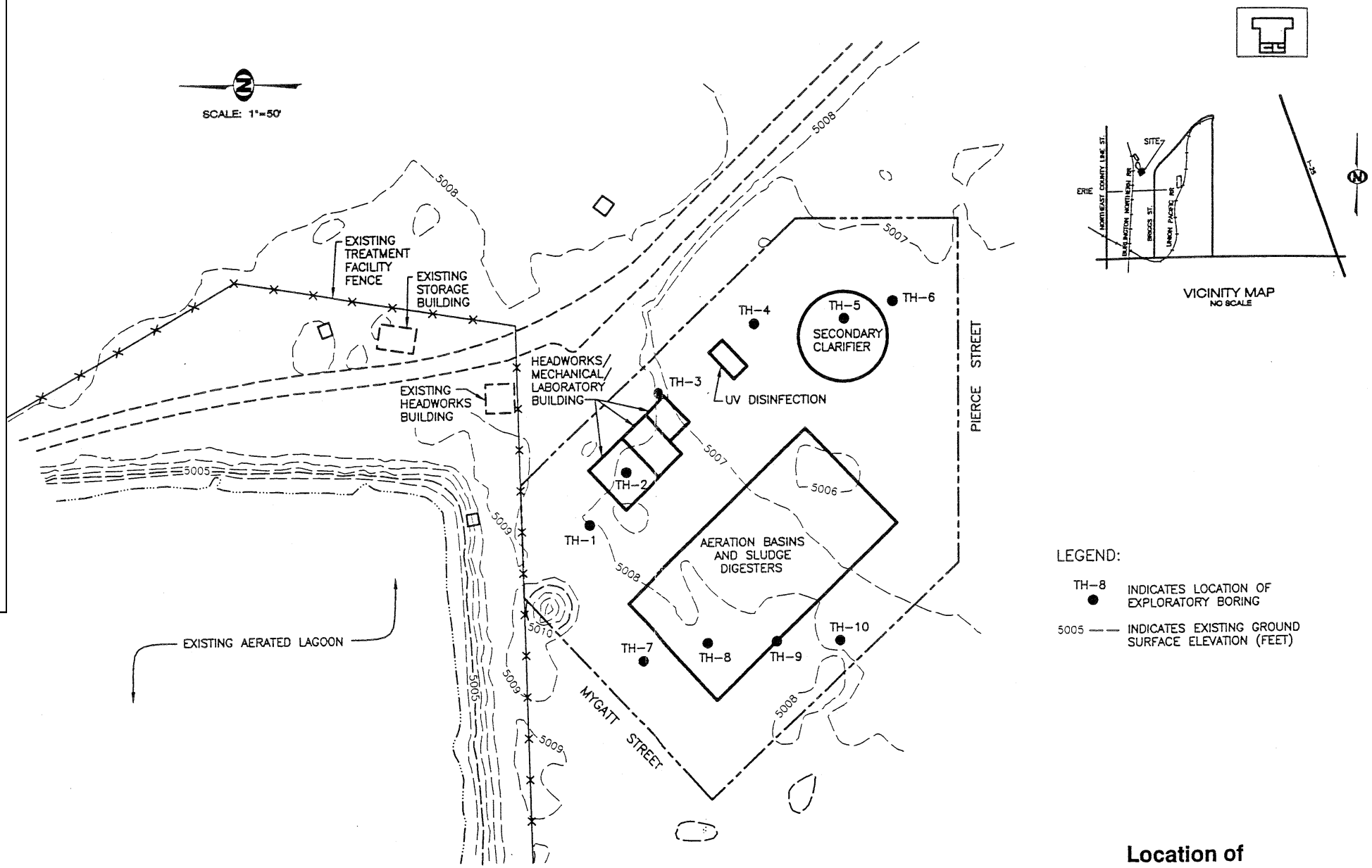
South Water Reclamation Facility. CTL/Thompson Inc. conducted a geotechnical investigation of the SWRF site in September 1997. A complete copy of the final geotechnical report is included in Appendix C. Subsurface conditions were investigated by drilling ten exploratory test holes at the locations shown in Figure 3-6 (CTL/Thompson, Inc. Consulting Engineers September 1997).

The ground surface at the SWRF slopes gently from northwest to southeast with about a two foot elevation drop across the site. Soil bores showed between five and twenty-eight feed of sandy clay over clean, silty, and clayey sands and interlayered clays and sands underlain by sedimentary sandstone at depths between 24.5 and 31 feet. The clays are described as soft to stiff. Sands are described as loose to medium dense. The two clayey sand samples tested had unconfined compressive strengths of 400 and 700 psf. Gradation tests showed samples of the sands contained 7 to 46 percent passing the No. 200 screen (CTL/Thompson, Inc. Consulting Engineers September 1997). The geotechnical report cites nil to low swell potential.

Sulfate was present at concentrations ranging from 0.04 to 0.12 percent. Modified Type II concrete with sulfate resistance was recommended for all below grade construction (CTL/Thompson, Inc. Consulting Engineers September 1997).

Very hard sandstone bedrock was penetrated by all test holes at depths between 24.5 and 31 feet below the existing ground surface. The sandstone samples tested had 19 to 30 percent fines (passing No. 200 sieve) (CTL/Thomson, Inc, 1997).

Figure 3-6 South Water Reclamation Facility Soil Bores



Location of Exploratory Borings

Fig. 1

Rocky Mountain Consultants, Inc.
Wastewater Treatment Facility
Job No. 25,353

25,353_01 12/13/96 MD

North Water Reclamation Facility. Kumar and Associates, Inc. conducted a geotechnical investigation of the NWRF site on November 30, 2005. A complete copy of the geotechnical report is included in Appendix C. The field exploration program consisted of seven (7) exploratory borings and three (3) exploratory pits, shown in Figure 3-7, to obtain information on subsurface conditions (Kumar and Associates, Inc. May 6, 2009). The site is located within the limits of a previous aggregate mining operation. The western one-third of the NWRF site was mined of aggregate, backfilled with concrete rubble, and covered with a thin layer of soil. The site has been reclaimed by placing fill to an elevation above the 100 year flood plain (Kumar and Associates, Inc. May 6, 2009).

The overall topography of the NWRF site is nearly level with a slight slope down to west northwest. Concrete rubble was encountered in the three exploratory pits and in one boring. This concrete rubble was encountered to depths of 11 feet below existing grade. The rubble varied in size to a maximum size of 4 to 5 feet. There was approximately 2 feet of organic and clay soil overlaying the concrete rubble. Fill soils were encountered in six of the seven borings at depths of approximately two to seven feet. The fill consisted of clay with layers of silt and gravel. Granular material was encountered below the fill material and ranged from silty sand to poorly graded gravel. Claystone bedrock was encountered at depths ranging from thirteen to sixteen feet in four of the borings. Samples taken of the Claystone indicated that it exhibited low swell potential. Cemented hard sandstone bedrock was encountered at a depth of approximately 25 feet below grade (Burns & McDonnell, 2009). Bedrock continued down through the explored depths of twenty to forty feet (Kumar and Associates, Inc. May 6, 2009).

Sulfate was present at concentrations ranging from 0.02 to 0.37 percent. Modified Type II concrete with sulfate resistance was recommended for all below grade construction

During construction, structural fill was used to reconstruct the NWRF site, fill in the gravel pits, and raise the ground level above the 100-year flood plain. Approximately 10 feet depth of structural fill was placed over the site.

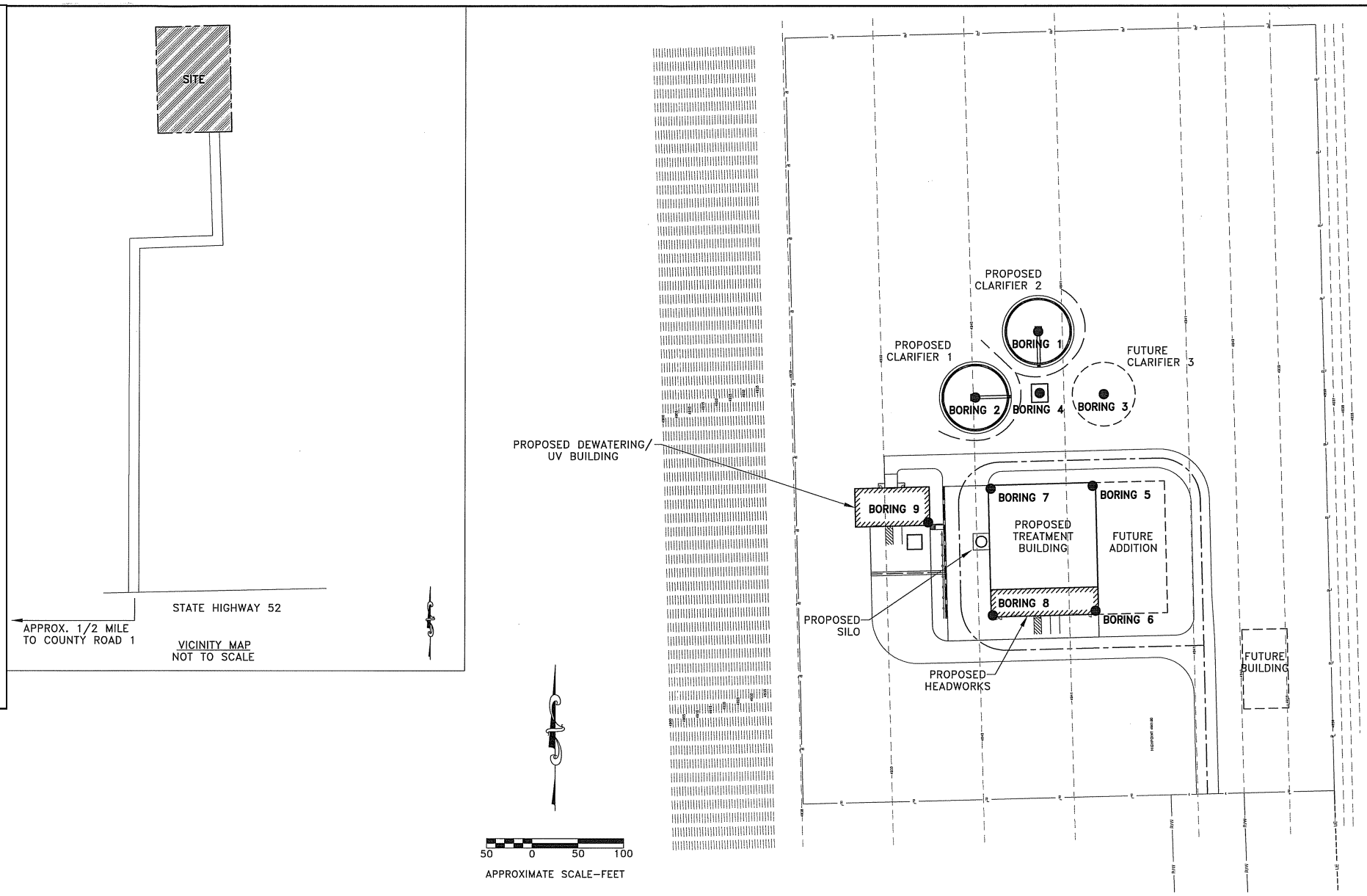
Groundwater

Ten test holes were drilled at the SWRF site as part of the geotechnical investigation (CTL/Thompson, Inc. Consulting Engineers September 1997). Groundwater was measured in all test holes at depths between three and eleven feet below grade. Groundwater was encountered in all nine test borings at the NWRF site at depths of seven to seventeen feet below grade (CTL/Thompson, Inc. Consulting Engineers September 1997). For both sites, the geotechnical reports noted that construction would be impacted by high groundwater and that temporary and permanent dewatering may be needed for below grade structures. All below grade structures should be made water tight and be able to withstand buoyant force.

At the NWRF, approximately 10 feet of structural fill was placed over the site. Consequently, groundwater will be at least ten feet below grade.

Average depth to ground water maps from the National Resources Conservation Service Web Soil Survey are included in Appendix C for both the SWRF and NWRF sites.

Figure 3-7 North Water Reclamation Facility Soil Bores



09–1–112	Kumar & Associates	TOWN OF ERIE NORTH WATER RECLAMATION FACILITY	LOCATIONS OF EXPLORATORY BORINGS	Fig. 1

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Wetlands

Wetlands are transitional areas between aquatic ecosystems and uplands. True wetlands are defined by three criteria: vegetation, hydrology, and soils. Wetlands must, at least periodically, support predominantly hydrophytic vegetation. Wetlands are characterized by free water for at least part of the year and by soil type.

The presence/absence of wetlands for the NWRF and SWRF sites was determined by searching the National Wetlands Inventory Database and by searching the NRCS for the presence of hydric soils on and around the sites. The National Wetlands Inventory database did not show any wetlands on or around the sites; however, digital maps are not available for most of Colorado. It is highly likely that riparian wetlands extend along the banks of both Boulder and Coal Creeks. Wetlands dominated by sandbar willow (*Salix exigua*) and reed canarygrass (*Phalaris arundinacea*) occur along Boulder Creek. On the terraces above Boulder Creek, many large mature plains cottonwoods (*Populus deltoides*) occur.

The U.S. Fish and Wildlife's National Wetland Inventory (NWI) describes potential wetlands found within Erie's planning area. Boulder Creek flows through the western portion of the project area and the northern portion of the project area is within the historical floodplain. Eight (8) potential wetlands were identified and described in the environmental report for the NWRF, which is included in Appendix D. Out of the eight potential wetlands described, it was determined that five of the eight wetlands were most likely non-jurisdictional because there is no connection to waters of the U.S (Burns and McDonnell, Inc. April 29, 2009).

Wetlands may also be partially defined by the presence of hydric soils. The definition of a hydric soil is a soil that formed under conditions of saturation, flooding or ponding long enough during the growing season to develop anaerobic conditions in the upper part. The concept of hydric soils includes soils developed under sufficiently wet conditions to support the growth and regeneration of hydrophytic vegetation. The NWRF and SWRF sites are underlined by soil types aquolls and aquepts. Both are hydric soils. Hydrologic soil groups are based on estimates of runoff potential. They are then assigned to one of four groups, A through D, based on the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms. Aquolls and aquaepts are Group D soils. Group D soils have very slow infiltration rates (high runoff potential) when thoroughly wet. Additional information about hydric soils as well as hydric soil maps of the SWRF and NWRF sites are included in Appendix C.

The presence of hydric soils does not automatically translate into the presence of wetlands. Hydrophytic plants must also be present as well as standing water for at least part of the year.

Floodplain

Figure 3-8 shows the Flood Insurance Rate Map (FIRM) of the 100-year flood plain for Coal Creek (Federal Emergency Management Agency December 2, 2204). The SWRF site is within the 100-year flood plain for Coal Creek. Treatment facilities were protected from flooding by importing fill to raise the elevation of the site surface above the 100-year flood level prior to constructing facilities (HDR

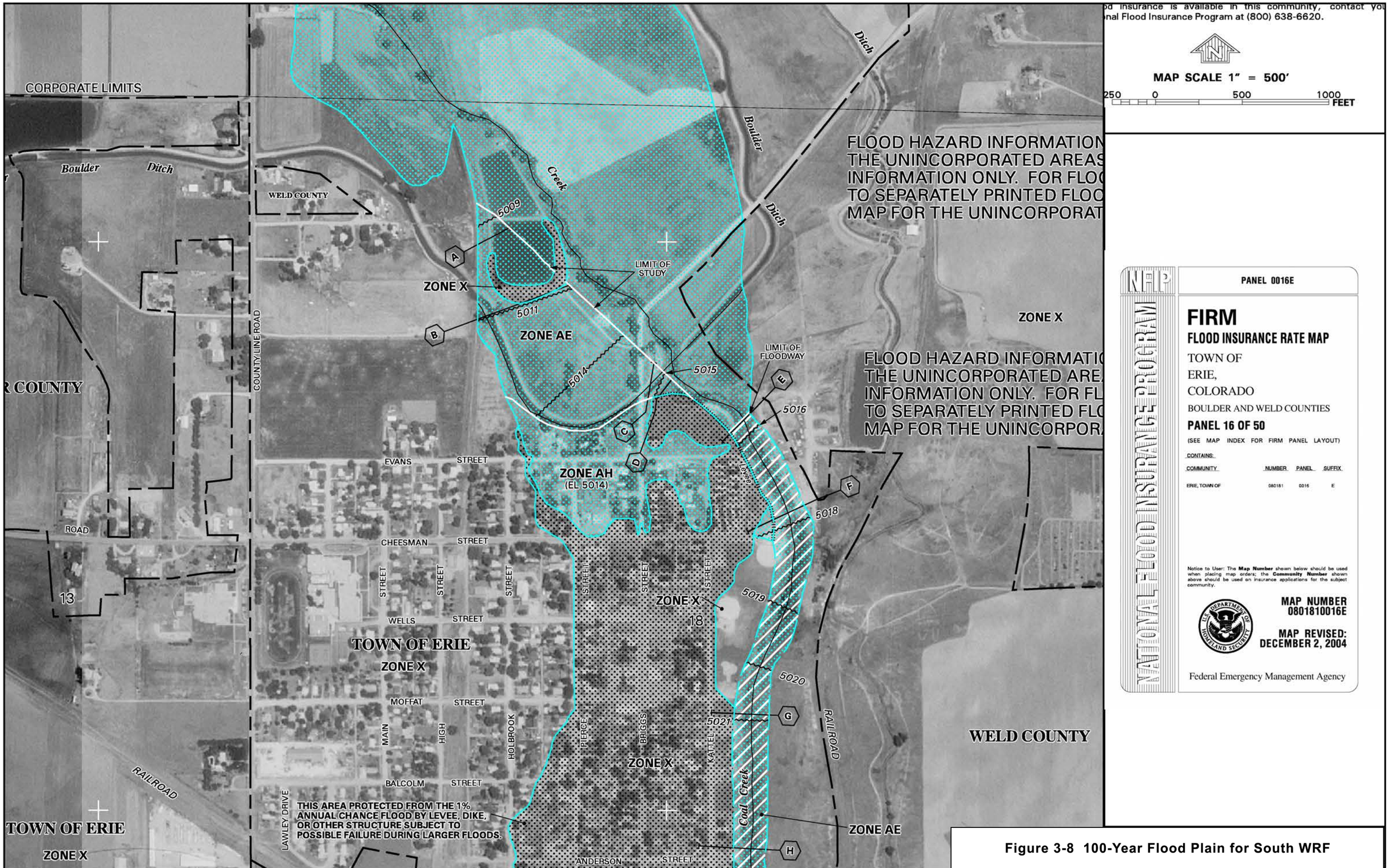


Figure 3-8 100-Year Flood Plain for South WRF

Engineering, Inc. June 2001). No part of the SWRF mechanical treatment plant is within the 100 year flood plain for Coal Creek.

Figure 3-9 shows the Flood Insurance Rate Map (FIRM) for the 100-year flood plain in relation to the NWRF. The FIRM used is community panel number 080266 0850 C, revised September 28, 1982. The floodplain has been altered by aggregate mining activity at the NWRF site and is not accurately described by the FIRM. The Martin and Martin report, titled "Preliminary Floodplain Study for Lower Boulder Creek, E. County Line Rd. to County Rd. 16-1/2, Weld County", was completed in 2009. An application for a Letter of Map Revision (CLOMR) for Lower Boulder Creek was filed with FEMA in June 2011, but has not received final approval. A copy of the application without Appendices has been included in Appendix E.

As part of the land agreement with the town of Erie, the mining company was required to fill the NWRF site to an acceptable elevation for the treatment plant. Martin/Martin has performed hydraulic modeling as part of the floodplain analysis to ensure that the floodway is not altered by this project and that no adjacent properties will be adversely affected due to fill on the NWRF property. The study determined that the 100-year floodplain elevation at the southern edge of the NWRF property corresponding to Boulder Creek is 4934.7 (using the 1988 NAVD datum). This elevation is lower than the pre-mining elevation at the drive entrance to the plant site, which supports the limited amount of fill in the floodplain required for the plant site. It should be noted that a datum change has occurred since the original floodplain mapping was completed. When comparing the older and new models, it is important to take this into consideration (Burns & McDonnell, 2009).

Knowing the 100 year floodplain elevation (4934.7), the NWRF site was graded to 4940.0 at a minimum. This is a stipulation of the land agreement between the mining company and the Town of Erie. As such, the site is located well above the 100 year floodplain, reducing flood risk for the site. No part of the NWRF site is within the floodway of Boulder Creek (Burns & McDonnell, 2009).

Further complicating the NWRF treatment site is an adjacent parcel of land that will be used as a storage reservoir. This storage reservoir site is located adjacent to the treatment site, closer to the floodway. As part of the project analysis, the reservoir site was reviewed for the appropriate bank height. It was determined through preliminary studies by another consultant that raising the reservoir banks above the 100 year floodplain elevation would affect adjacent or downstream properties. The reservoir will be used as part of the Town's augmentation plan by periodically releasing stored water into Boulder Creek. Water will also be used for irrigation of Town parks and public areas. Since the reservoir will only be used for non-potable water, it was decided to keep the bank elevations closer to historic levels, thus the reservoir site will have no impact on adjacent properties (Burns & McDonnell, 2009).

The Letter of Map Revision (CLOMR) for Lower Boulder Creek that was filed with FEMA in June 2011 notes that in the event of a flood, flows will split into two streams upstream of the NWRF and reservoir. An existing berm located along the east bank of Boulder Creek forms a natural barrier and will divide the stream under flood conditions. A portion of the flow will be conveyed within Boulder Creek and the remainder will be conveyed through the reservoir.

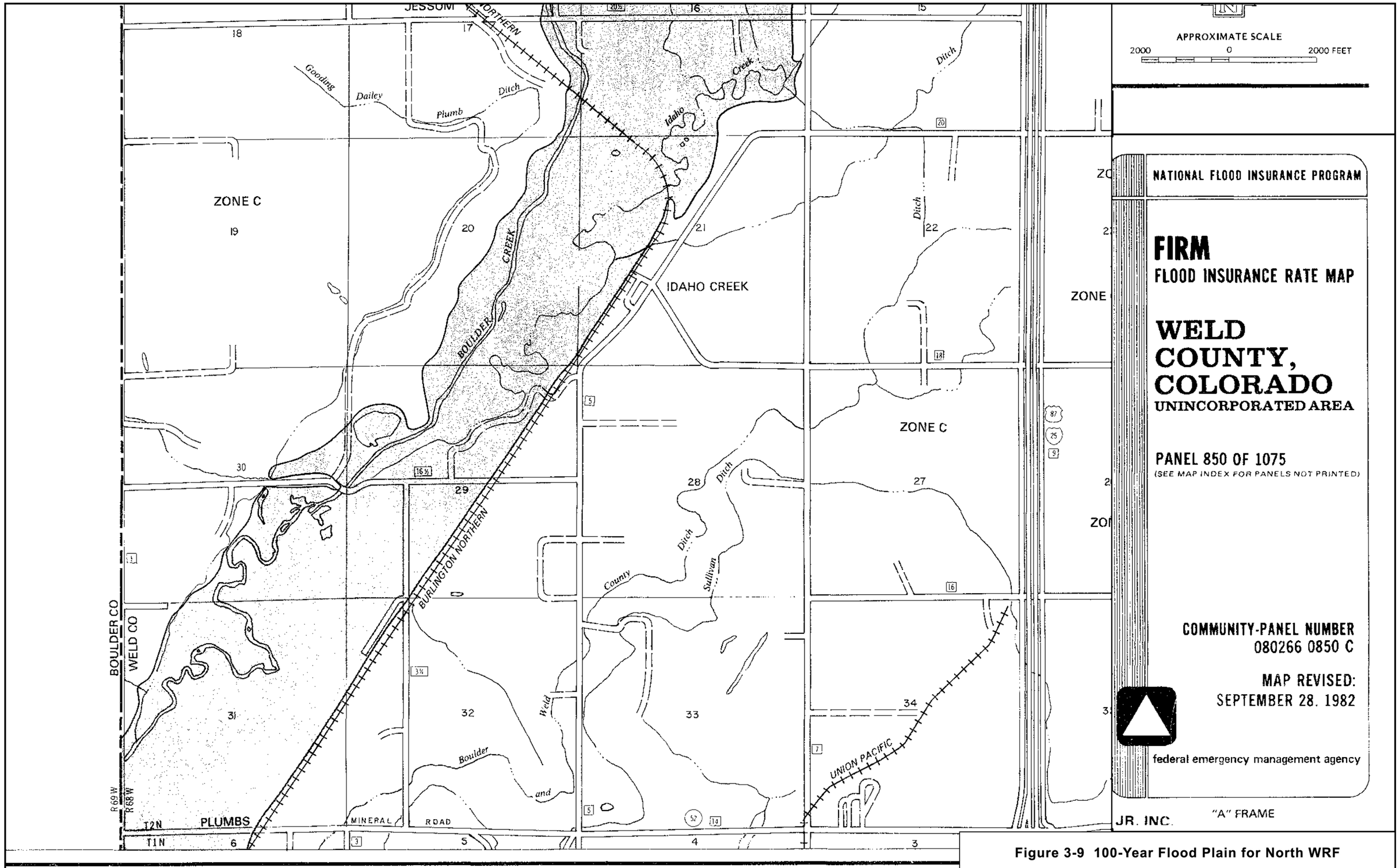


Figure 3-9 100-Year Flood Plain for North WRF

Water stored in the reservoir will meet the NWRFs discharge permit limits and reuse standards under Regulation 84. In the event of a flood, water may move through the reservoir and push stored water into Boulder Creek. No negative impacts to water quality in Boulder Creek are expected as water quality in the reservoir is based on low flow analysis of the creek. Following a flood, water quality in the reservoir may need to be assessed to determine suitability for discharge and/or reuse.

Wildlife

Wildlife range maps were obtained through the Colorado Division of Wildlife. Wildlife ranges for the SWRF and NWRF sites is shown in Figure 3-10. Boulder Creek serves as a corridor between the mountains, foothills, and lower elevations for many different types of wildlife including Geese, White Pelicans, Great Blue Herons, and White Tail Deer. Bald Eagle communal roosts are located northeast and west of the NWRF site. In addition to the wildlife ranges shown in Figure 3-10, the entire area is home to pheasants, black tailed prairie dogs, and mule deer.

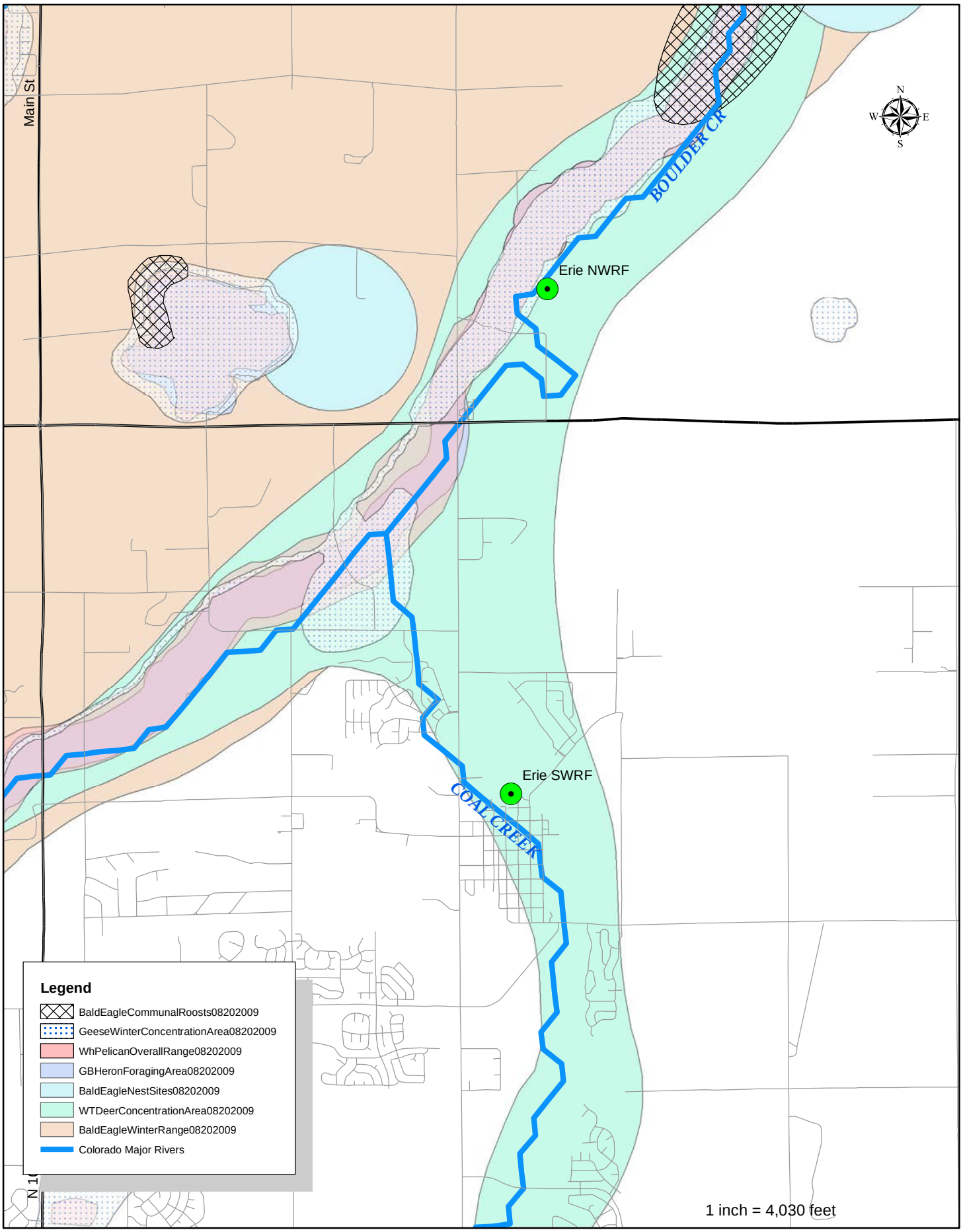
Boulder County publishes maps of Environmental Conservation Areas and Environmental Resources. These maps are included as Figures 3-11 and 3-12, respectively. The Environmental Resources map denotes natural communities, rare plants, riparian corridors, and critical wildlife habitat. The Environmental Conservation Areas map shows Boulder Creek and parts of Coal Creek as “Stream Habitat Connectors”. Erie’s planning area contains some environmental conservation areas around Panama Reservoir. Erie’s master plan calls for these areas to remain as open space and is consistent with the Environmental Conservation Areas map. There are two places within the planning area that are designated as critical wildlife habitat: Panama Reservoir and B-J Acres Ranch. No portion of either the SWRF or the NWRF sites is designated as critical wildlife habitat.

Noise and physical presence mitigation efforts must be implemented when certain threatened and endangered species inhabit a certain land area. A report was prepared in December 2005 which discusses federally threatened, endangered and candidate species potentially found in Boulder and Weld counties (Burns and McDonnell, Inc. April 29, 2009). The report is included in Appendix D. Table 1 of the report summarizes the mammals, birds, fish, and plants that are listed and may or may not be within the project boundaries within Weld or Boulder County. The mammals include the Black-footed ferret, the Canada lynx, and the Preble’s meadow jumping mouse. The birds include the Bald eagle, Interior least tern, Mexican spotted owl, Piping plover, and the Whooping crane. The fish include the Greenback cutthroat trout and pallid sturgeon. The plants include the Colorado butterfly plant and Ute ladies’-tresses orchid. The report determined that only the Bald eagle appears to inhabit the project area (Burns and McDonnell, Inc. April 29, 2009). The other named species are unaffected. Recommendations are included in the report in Appendix D.

SUMMARY OF ENVIRONMENTAL INFORMATION OR SPECIAL STUDIES

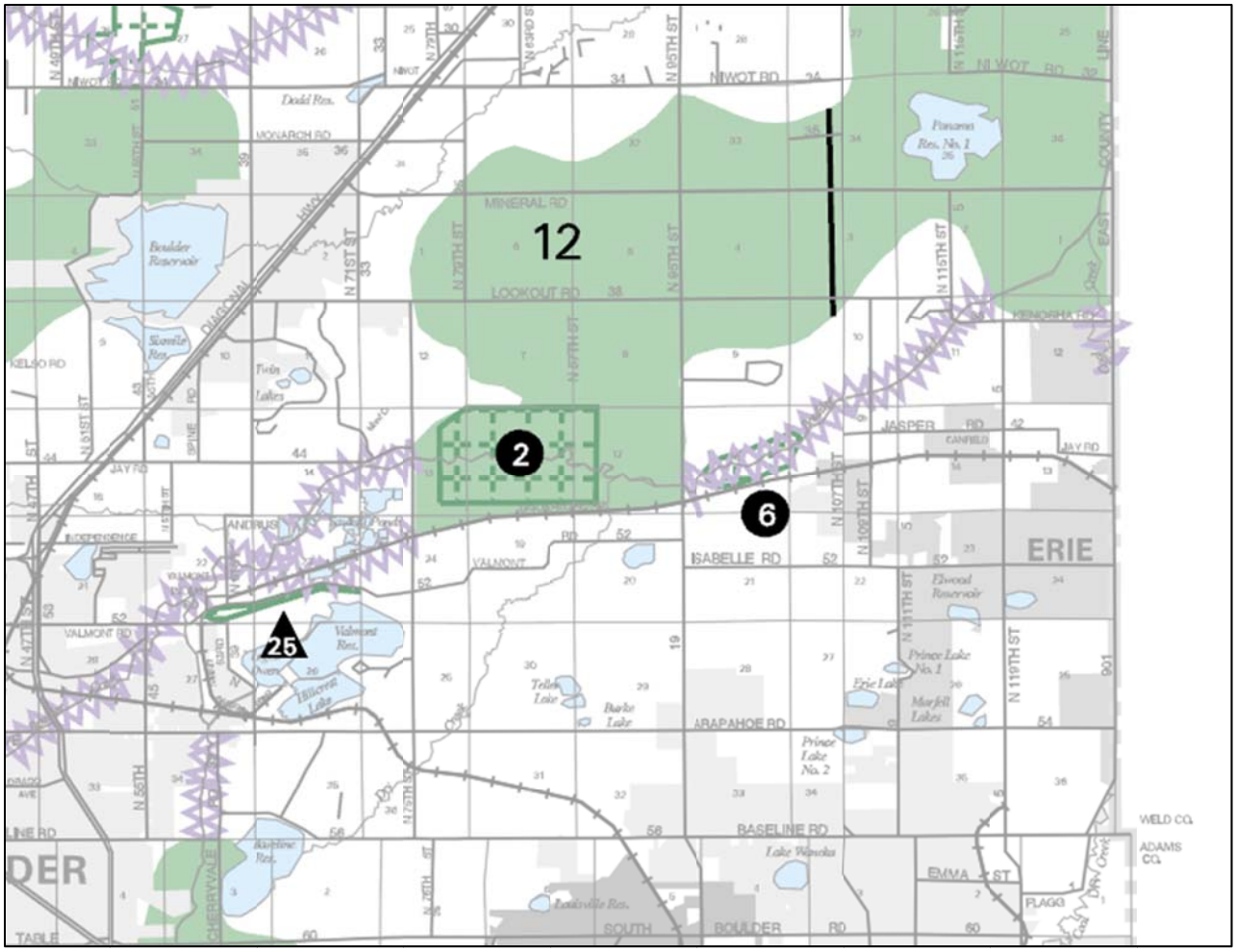
A Total Maximum Daily Load (TMDL) study was completed for Segment three of the Saint Vrain Watershed, including Boulder and Coal Creeks, in July 2003. Segment three begins at Hygiene Road and continues to the South Platte River. The TMDL utilized the Colorado Ammonia Model to determine monthly acute and chronic daily loads at critical points throughout the watershed and at each wastewater

treatment facility discharge point. Thirteen wastewater treatment facilities as well as some smaller permitted facilities and non-point sources were included in the analysis. The TMDL endpoint was 0.06 mg/L of unionized ammonia (chronic) (United States Environmental Protection Agency 2003).



INDIGO WATER GROUP

Figure 3-10 Wildlife Habitat Ranges



Environmental Conservation Areas
Environmental Resources

Areas where conservation/preservation is encouraged

Legend

Environmental Conservation Areas

0 (See Back for Descriptions)

Overland Habitat Connectors

Stream Habitat Connectors

Natural Landmarks & Natural Areas

Natural Landmarks

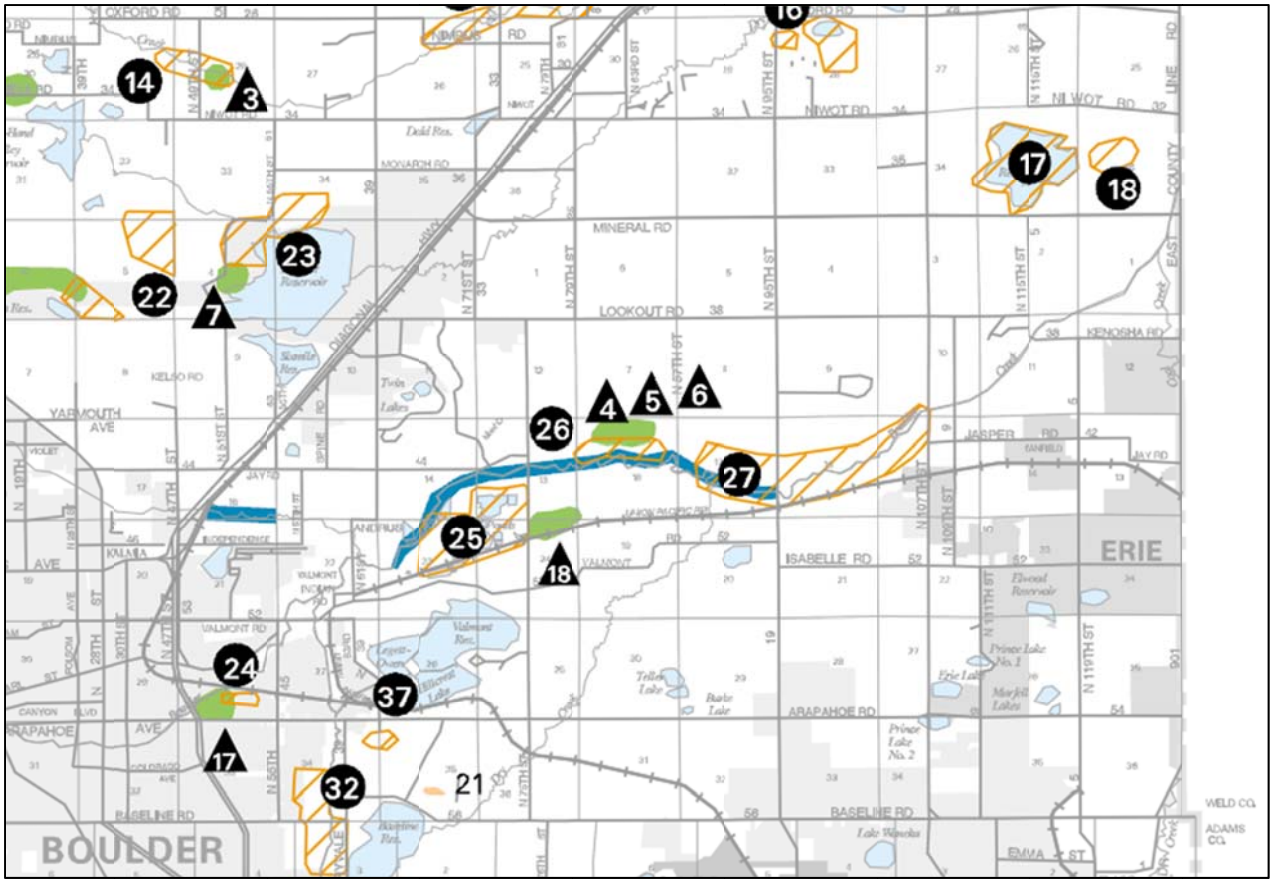
0 (See Back for Descriptions)

Natural Areas

0 (See Back for Descriptions)

Incorporated Areas
(As of April 20, 1999)

Figure 3-11 Environmental Conservation Areas in Boulder County
Source: Boulder County Comprehensive Plan



Natural Communities, Rare Plants Riparian Corridors, and Critical Wildlife Habitats
Environmental Resources

Legend

Significant Natural Communities
0 (See Back for Descriptions)

Rare Plant Areas
0 (See Back for Descriptions)

Critical Wildlife Habitats
0 (See Back for Descriptions)

*Significant Riparian Corridors

*The significant riparian corridors were identified as part of a limited study conducted in 1986-87, and do not necessarily represent the full extent of significant riparian corridors Countywide.

Incorporated Areas
(As of April 20, 1999)

Figure 3-12 Significant Environmental Communities in Boulder County
Source: Boulder County Comprehensive Plan

Significant Natural Communities

1 Montane Willow Carr

2 Foothills Ponderosa Pine Scrub Woodland (Purshia)

3 Mixed Foothills Shrubland (Purshia)

4 Foothills Ponderosa Pine Savanna

5 Mixed Foothills Shrubland (Cercocarpus)

6 Xeric Tallgrass Prairie

7 Foothills Ponderosa Pine Scrub Woodland (Cercocarpus)

8 Foothills Ponderosa Pine Scrub Woodland (Cercocarpus)

9 Foothills Ponderosa Pine Scrub Woodland (Cercocarpus)

10 Great Plains Mixed Grass Prairie (Stipa comata)

11 Great Plains Mixed Grass Prairie (Stipa neomexicana)

12 Great Plains Salt Meadow

13 Great Plains mixed Grass Prairie (Stipa comata)

14 Alpine Wetlands

15 Montane Wet Willow Carr

16 Montane Grasslands

17 Foothills Ponderosa Pine Savanna

18 Montane Grasslands

19 Foothills Ponderosa Pine Savanna

20 Xeric Tallgrass Prairie

21 Great Plains Mixed Grass Prairie

22 Montane Willow Carr

23 Montane Grasslands

24 Wet Prairie

25 Wet Prairie

26 Wet Prairie

27 Xeric Tallgrass Prairie

Rare Plant Areas

1 *Physaria bellii*

2 *Physaria bellii*

3 *Physaria bellii*

4 *Aristida basiramea*

5 *Apios americana*

6 *Asplenium adiantum-nigrum*

7 *Physaria bellii*

8 *Physaria bellii*

9 *Phippsia algida*

10 *Physaria bellii*

11 *Listera convallarioides*

12 *Eustoma grandiflorum*

13 *Physaria bellii*

14 *Physaria bellii*

15 *Selaginella weatherbiana*

16 *Amorpha nana, Betula papyrifera, Carex saximontana, Listera convallarioides, Malaxis brachypoda, Pyrola picta, Selaginella weatherbiana*

17 *sensitive species*

18 *Eustoma grandiflorum*

19 *Botrychium echo, Botrychium hesperium, Botrychium lanceolatum, Botrychium pallidum, Listera convallarioides, Botrychium minganense*

20 *Selaginella weatherbiana*

21 *Amorpha nana*

22 *Amorpha nana*

Critical Wildlife Habitats

1 Ouzel Falls

2 Copeland Willow Carr (+wetlands)

3 South Sheep Mountain

4 Deadman Gulch and South St. Vrain

4 Steamboat Mountain

6 St. Vrain Creek (+wetlands)

7 St. Vrain Corridor (+wetlands)

8 Rabbit Mountain

9 Old Apple Valley

10 Middle St. Vrain Willow Carr (+wetlands)

11 Tumbleson Lake (+wetlands)

12 Marietta Canyon

13 Lefthand Palisades

14 Lefthand Creek Cottonwood Groves (+wetlands)

15 Lagerman Reservoir (+wetlands)

16 Gaynor Lakes (+wetlands)

17 Panama Reservoir (+wetlands)

18 B-J Acres Ranch

19 City of Boulder Watershed (Special Consideration)

20 Como Creek (Special Consideration)

21 Boulder Falls area

22 Boulder Valley Ranch (+wetlands)

23 Boulder Reservoir (+wetlands)

24 Cottonwood Grove on Boulder Creek (+wetlands)

25 Walden and Sawhill Ponds (+wetlands)

26 White Rocks (+wetlands)

27 Cottonwood Grove & Heron Rookery (+wetlands)

28 Diamond Lake Outlet

29 Chittenden Meadows (+wetlands)

30 Arapaho Ranch - Tucker Homestead (+wetlands)

31 Boulder Mountain Parks - Eldorado Mountain

32 South Boulder Creek (+wetlands)

33 Tallgrass Prairie

34 Marshall Mesa

35 Stearns Lake (+wetlands)

36 Carolyn Holmberg Preserve at Rock Creek Farm

37 Sombrero Marsh (+wetlands)

38 Lazy H. Ranch Willow Carr (+wetlands)

39 Coney Flats Willow Carr (+wetlands)

40 Mitchell Lake Willow Carr (+wetlands)

41 Coney Lake Willow Carr (+ wetlands)

42 Long Lake Willow Carr (+wetlands)

43 South St. Vrain Willow Carr (+wetlands)

44 Lefthand Reservoir Willow Carr (+wetlands)

45 Boulder Watershed Willow Carr (+wetlands)

46 Lake Albion Willow Carr (+wetlands)

47 Triple Lakes Willow Carr (+wetlands)

48 Horseshoe Creek Willow Carr (+wetlands)

49 Caribou Park Willow Carr (+wetlands)

50 Upper Caribou Park Willow Carr (+wetlands)

51 Delonde Creek Willow Carr (+wetlands)

52 Caribou Ranch Willow Carr (+wetlands)

53 Woodland Flats Willow Carr (+wetlands)

54 Buckeye Basin Willow Carr (+wetlands)

55 Los Lagos Willow Carr (+wetlands)

56 Roaring Fork Willow Carr (+wetlands)

57 Peterson Lake (+wetlands)

58 Hunter's Creek

Figure 3-12b Legend for Significant Environmental Communities

SECTION 4
WASTEWATER CHARACTERISTICS

The capacity and type of treatment facilities needed to provide treatment for the Town of Erie will depend on future development of the service area, flow variations, wastewater characteristics, and regulatory requirements. Future needs have been estimated based on demographic trends in the service area, population based growth projections, and analysis of historic wastewater flows and loads at the south water reclamation facility (SWRF). Wastewater characteristics for the Town are based on a eight-year record of flows and waste strength from 2003 through 2010. The draft WUP was submitted to NFRWQPA for review in early 2011 which is why data from 2011 and 2012 is not included in the summary tables.

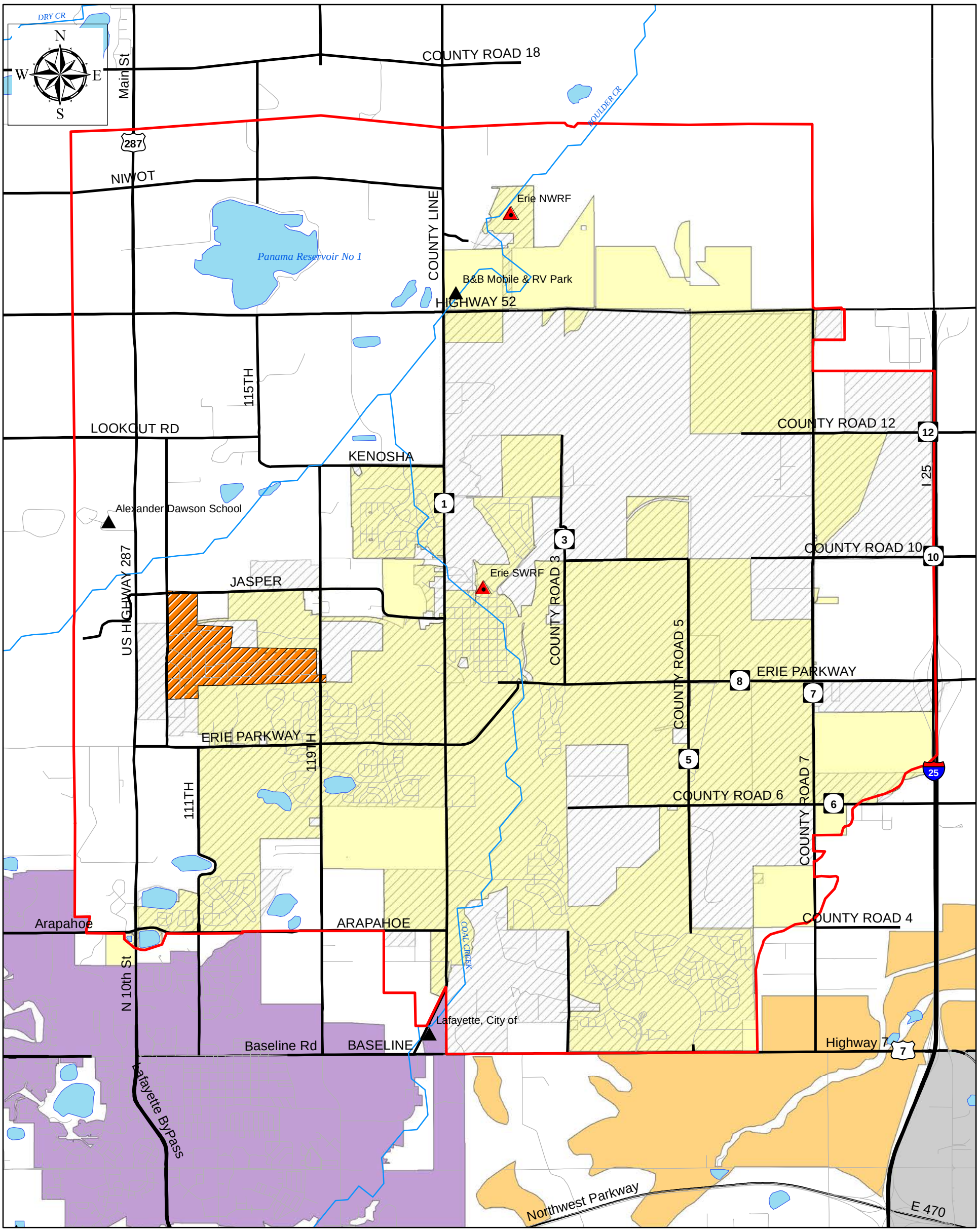
SERVICE AREA DESIGNATIONS

The Town of Erie (Town) is located along the Front Range, north of Denver and east of Boulder along Coal Creek. Erie's planning area is located in Boulder and Weld counties. Figure 4-1 shows the current service area and planning area boundaries. The planning area is about 50 square miles. The boundary of the planning area is somewhat irregular but, in general, it is formed by Baseline Road (Highway 7) on the south and by Interstate 25 on the east. The western boundary is about ½ mile west of Highway 287 and the northern boundary is approximately 1.7 miles north of State Highway 52. The Brownsville Water and Sanitation District is surrounded by Erie's planning area, but is not part of the planning area.

The Town of Erie has two wastewater reclamation facilities. The South Water Reclamation Facility (SWRF) is located near the center of the service area just north of the intersection of Briggs Street and Evans Street. Briggs Street becomes County Road 1 ½ as it passes the SWRF. The SWRF is an extended aeration activated sludge plant and has a rated capacity of 1.6 mgd and 3,870 ppd of Biochemical Oxygen Demand (BOD). The North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The NWRF is an Integrated Fixed Film Activated Sludge Plant (IFAS) and has a rated capacity of 1.5 mgd and 3,233 ppd of BOD. The NWRF is located north of Highway 52 and east of County Line Road along Boulder Creek. Legal descriptions for each site are given in Tables 4-1 and 4-2. Copies of the discharge permits are included in Appendix F.

Table 4-1 Town of Erie South Water Reclamation Facility Legal Description and Capacity

Characteristic	Description
WWRF Mailing Address	Town of Erie 645 Holbrook P.O. Box 100 Erie, Colorado 80516
WWRF Location	1000 Briggs Street Erie, Colorado 80516
Site Legal Description	In the NE ¼ of the NW ¼ of Section 18, T1N, R68W
Permitted Hydraulic Capacity, mgd	1.6
Permitted Organic Capacity, ppd BOD_5	3,870



Legend

- Historic Planning Area

NFRWQPA 208 Planning Areas

- *Broomfield*
- *Erie*
- *Northglenn*
- *St. Vrain Sanitation District*
- *Brownsville W&SD*

DRCOG WUSA

- *Broomfield*
- *Erie*
- *Lafayette*
- *Northglenn*
- *Thornton*

- Erie Town Boundary
- *Wastewater Treatment Plants*
- *Erie Water Reclamation Facilities*
- Streets
- Colorado Major Rivers
- Minor Waterways
- Lakes

NOTE: Erie's historic planning boundary is shown with a bold red outline.
The historic planning boundary does not match the proposed 208 boundary.

Figure 4-1 Town of Erie Historic Service and Planning Areas

Table 4-2 Town of Erie North Water Reclamation Facility Legal Description and Capacity

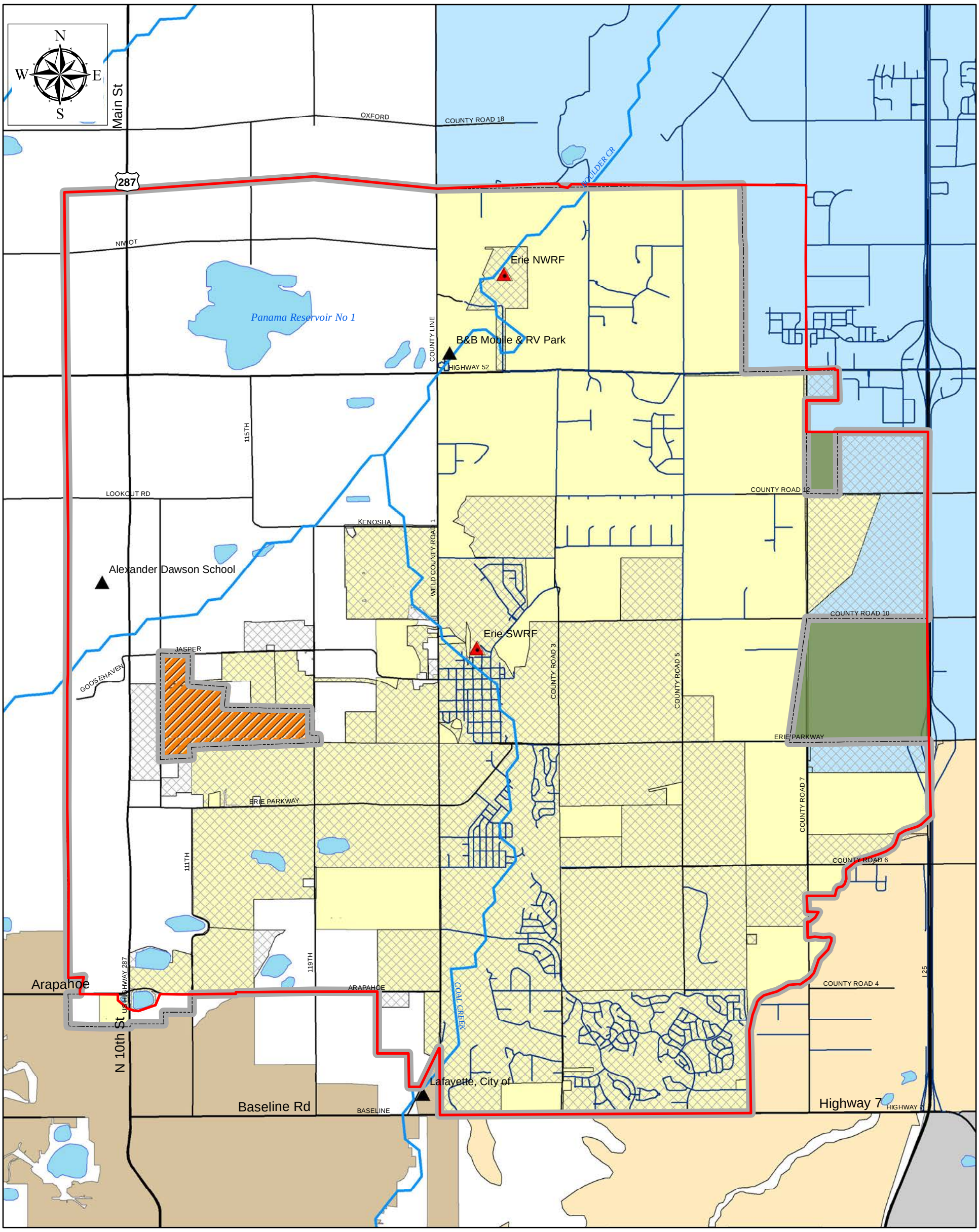
Characteristic	Description
WWRF Mailing Address	Town of Erie 645 Holbrook P.O. Box 100 Erie, Colorado 80516
WWRF Location	501 State Highway 52 Erie, Colorado 80516
Site Legal Description	In the east half of Section 31, T2N, Range 68 West of the 6th PM of Weld County
Permitted Hydraulic Capacity, mgd	1.5
Permitted Organic Capacity, ppd BOD_5	3,233

WUSA and CWP Planning Areas

The Town of Erie is requesting a formal change to its 208 Planning Area Boundary. The proposed 208 boundary, shown in Figure 4-2 – Town of Erie Proposed versus Existing 208 Boundary, will match Erie's existing internal planning area boundary. Updating the 208 Boundary will facilitate water reuse and allow the Town to provide equivalent tap and service fees throughout their planning area. Erie has invested heavily in reuse water infrastructure including a new storage reservoir located at the NWRF site. Updating the 208 boundary will maximize capture of water for reuse by ensuring that areas receiving potable water from the Town also return wastewater to the Erie WWTFs. A portion of Erie's water portfolio may be used to extinction. Maximizing the water and sewer collection areas maximizes opportunities for reuse and ensures multiple cycles of "use to extinction" water. Currently, water and sewer fees differ depending on whether residents are located inside or outside of the designated service area. Updating the 208 Boundary will directly benefit users that are currently outside of the designated service area by making the eligible for in-service area rates thereby reducing their tap and service fees by one-half.

Updating the 208 boundary will make it consistent with Erie's historic planning area. Erie's internal planning area boundary has not changed since 1996 and has been referenced in numerous, publically available planning documents including Erie's 2001 Wastewater Utility Plan, 2005 Comprehensive Plan, 2006 Water and Wastewater Master Plan, and 2008 Update to the Wastewater Utility Plan. Much of the historic planning area is either already within the Town limits or has petitioned to be annexed. Figure 4-2 – Proposed 208 Boundary – shows the existing Town limits in light yellow with orange cross-hatching indicating areas containing Current Development Activities (CDA) that have petitioned to be annexed.

Changes to the eastern boundary of Erie's 208 Boundary will bring the 208 Boundary edge east to I-25 south of Highway 52. I-25 forms a natural service area barrier. Moving the 208 Boundary edge east will bring the southern half of Section 3 and all of sections 10, 15, and 22 into Erie's 208 Planning Area. Section numbers are shown in purple on Figure 4-1. Sewer and water lines may cross I-25; however, this is logistically difficult and can be costly. Erie has an Intergovernmental Agreement (IGA) with the City of Dacono stating that neither municipality will annex property on the opposing side of I-25. The IGA makes Erie the logical water and sewer provider for all parcels located between I-25 and Erie's Town limits. This is particularly true for Section 15 which is bordered by Erie on three sides. Additional



Legend

- Historic Planning Boundary
- Proposed 208 Boundary
- Coordinated Area

0 0.25 0.5 1 Miles

- *Wastewater Treatment Plants*
- *Erie Water Reclamation Facilities*
- Colorado Major Rivers
- Lakes
- Minor Streets
- *Brownsville W&SD*
- Erie Incorporated

NFRWQPA 208 - 2009

- BROOMFIELD
- ERIE
- NORTHGLENN
- ST. VRAIN S.D.

DRCOG WUSA

- *Broomfield*
- *Erie*
- *Lafayette*
- *Northglenn*
- *Thornton*

NOTE: The proposed 208 bounday is Erie's historic planning area with modifications to accomodate the coordinated 208 planning area between the Town of Erie and St. Vrain Sanitation District.

Figure 4-2 Town of Erie Historic Planning Area and Proposed 208 Boundary

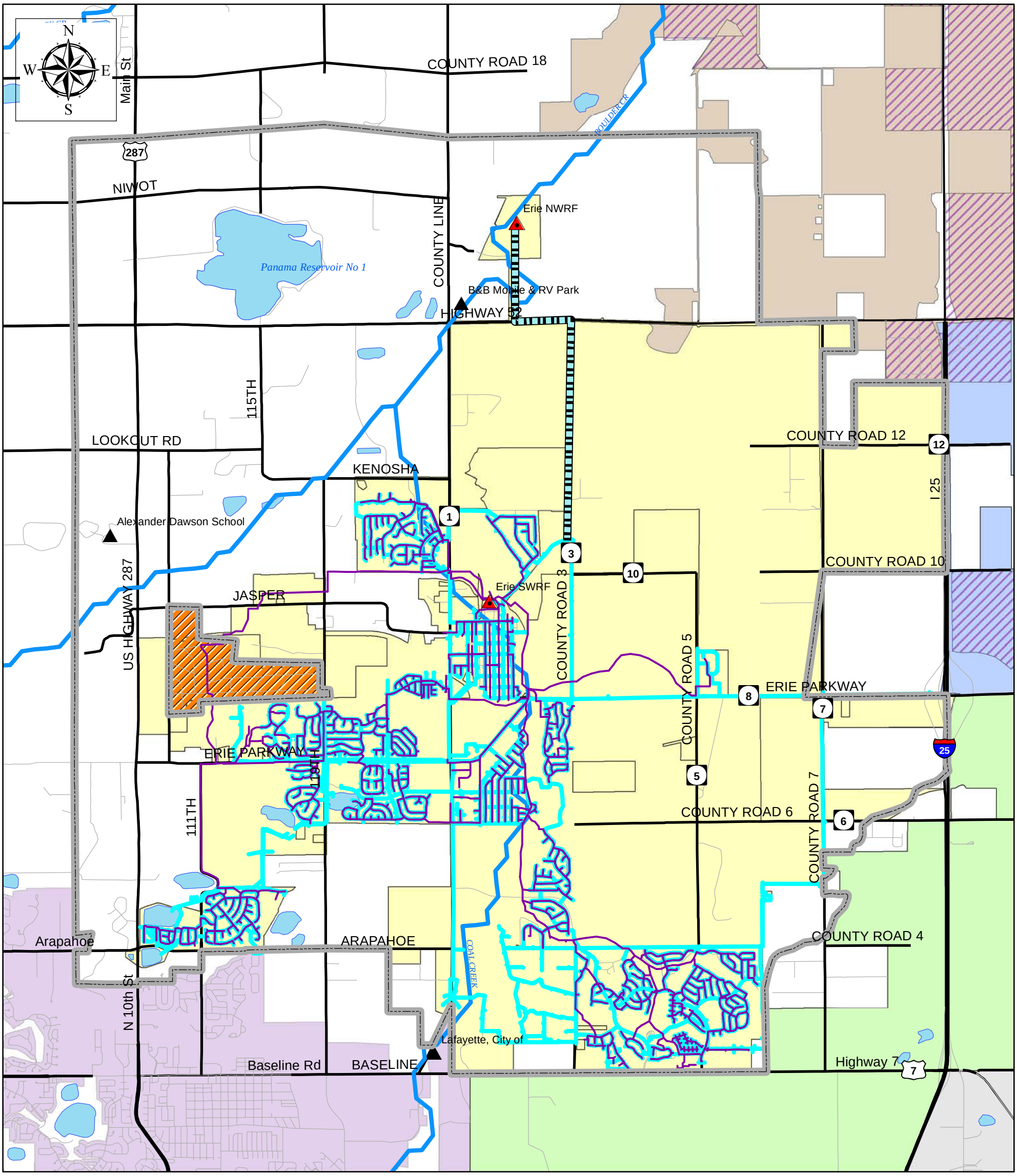
changes to the eastern 208 boundary border will bring the northwest quarter of the northwest quarter of Section 3 and the southeast quarter of Section 28 into Erie's 208 Planning Area. The 208 boundary will be extended west past Highway 287 and north past Niwot Road. Most of this area is designated as open space in Erie planning documents.

Changes to Erie's urban growth boundary (UGB) were approved by DRCOG in 2009. The updated UGB includes Sections 3, 10, and 15 which are located immediately west of Interstate-25 and are shown on Figure 4-1. Generally, the wastewater utility service area (WUSA) is equivalent to the UGB. In some instances, the WUSA may be larger than the UGB. By expanding the UGB into these areas, the Town has already indicated an intent to provide wastewater service.

The Brownsville Sanitation District is located within Erie's historic planning area. At one time, the District considered relinquishing control of its sewer lines to the Town of Erie. Citizens voted to maintain the District rather than consolidate. The Brownsville Sanitation District is not included in the proposed 208 boundary.

Portions of Erie's historic planning area boundary overlap with the St. Vrain Sanitation District's 208 boundary. These areas include a section that is generally bounded to the north by County Road 10, the south by Erie Parkway, to the west by County Road 7, and to the east by I-25. A second area of overlap is located north of County Road 12 and east of County Road 7. A map of the proposed 208 area is shown in Figure 4-1. Sanitary sewer service to portions of such areas by the Town or the District could require extensive trunk line extensions. The Town and the District acknowledge that regional cooperation by local governments to achieve economical and efficient services is authorized by Sec. 29-1-203, C.R.S., and is in the best interests of public health and stream quality. The Town of Erie and St. Vrain Sanitation District have agreed to designate these overlapping areas as coordinated or shared 208 areas. Residents within these areas will have the option of receiving wastewater service from either the Town or the District depending on proximity of gravity sewer lines and overall cost of service. If these areas are annexed by the Town of Erie, Erie will provide service. A copy of the Intergovernmental Agency Agreement (IGA) between the Town and the District, adopted on November 19, 2012, is included in Appendix A.

Water and Wastewater Service Erie's existing distribution and collection systems infrastructure are shown in Figure 4-3 – Town of Erie Water and Sewer Service Lines. Strategic placement of water mains allows Erie to provide potable water north and east of existing development. The Town has recently constructed a potable water line to the new Northern Water Reclamation Facility (NWRF). This line runs north on County Road 3 to Highway 52 and then west to the NWRF. Construction was completed in December 2010. A second potable water line is currently in design that will run east along Highway 52 to County Road 5. This line will provide potable water to the northeastern portion of Erie's planning area. Existing water lines running east to I-25 along Erie Parkway and north to south on County Road 7 will provide potable water to the eastern and southeastern portions of the service area. Additional distribution pipes will be added in accordance with the 2006 Water and Wastewater Master Plan (Black and Veatch)



Legend

- Proposed 208 Boundary
- *Wastewater Treatment Plants*
- *Erie Water Reclamation Facilities*

Collection System

- *Existing Sewer Lines*

Distribution System

- *Existing Water Line*
- *Water Line Complete Dec 2010*
- *Water Line in Design Stage*
- Brownsville Water & Sanitation District
- St. Vrain Sanitation District

- *Lafayette DRCOG WUSA*
- *Thornton DRCOG WUSA*

Selected City Boundaries

- Broomfield
- Dacono
- Erie
- Frederick

Figure 4-3 Town of Erie Collection and Distribution System

as development occurs. The placement of existing water pipes makes it cost effective to extend service to developing parcels.

The new NWRF was constructed in 2010 and became operational in early 2011. This facility was designed to serve areas downstream of the existing South Water Reclamation Facility (SWRF). The NWRF and associated infrastructure were sized to accommodate population growth within Erie's historic planning area. Erie's planning area has been consistent since 1996. The first stage of construction was permitted for 1.5 mgd and 3,223 ppd of BOD. Because the facility is sized for the historic planning area, reevaluation of population growth, future wastewater flows, and loads is not required. Population growth was most recently evaluated and presented to NFRWQPA in the 2008 Update to the Wastewater Utility Plan (Burns and McDonnell). Erie anticipates a buildout population of 68,820 residents.

The NWRF is capable of servicing the bulk of the planning area by gravity flow. In the 2006 Water and Wastewater Master Plan (Black and Veatch, 2007), collection system modeling suggested that a small lift station will be needed to serve portions of Sections 28 and 33. All other flows generated within the planning area can be conveyed by gravity. Black and Veatch issued an update to the 2006 Plan in July 2008 that recommended alternative alignments for the Interstate and Highway 52 interceptors. The alternative alignments call for deep gravity sewers instead of lift stations. Additional details regarding the collection system are discussed in Section 6 of this WUP.

Water Rights and Wastewater Reuse The primary reason for updating the 208 Boundary is to ensure recapture of wastewater flows for non-potable reuse. Maximizing reuse is consistent with the stated goals of NFRWQPA and DRCOG. Erie plans to use non-potable water to meet a portion of its irrigation and other non-potable water demands. Potential non-potable supplies include raw ditch water, raw water that would otherwise be treated for potable use, reservoir water, and treated wastewater return flows. Treated effluent may be recaptured at either of Erie's two wastewater treatment facilities (WWTFs). Flows captured at the NWRF may be transmitted to the SWRF. Reusable return flows from the SWRF that do not coincide with seasonal non-potable demands could be conveyed via Coal Creek and Boulder Creek to the NWRF storage facility.

Erie is investing heavily in reuse infrastructure. The new NWRF includes tertiary filtration to produce Category 2 reuse water and a non-potable pump station. A 1,000 acre-ft storage reservoir is currently being constructed adjacent to the NWRF site for reuse water storage. A splitter structure located downstream of the treatment plant will divert reuse eligible water to the storage pond and non-eligible water to Boulder Creek. Reuse water will be applied to unrestricted public access areas such as parks and greenways, but will not be under residential control.

The water supply inventory for the Town of Erie is a collection of sources including transbasin sources, such as Colorado-Big Thompson (CBT) units and Windy Gap Project (Windy Gap) units; reservoirs, such as Thomas, Erie, and Prince; as well as multiple irrigation company shares. Erie's current water rights portfolio provides an approximate yield of 8,260 acre-feet per year (afy). The water portfolio contains about 1,400 acre-ft of Windy Gap flows that may be used to extinction. The Town plans to acquire additional Windy Gap shares. When the Northern Integrated Water Supply Project (NISP) is completed,

Erie will have an additional 6,000 acre-ft available for reuse. The original NISP delivery has fallen behind schedule due to technical and political uncertainty, although public interest remains high. It is estimated to be delivered sometime around 2020.

Summary Erie will provide potable water service within its historic planning area. The historic planning area has remained unchanged since 1996 and has been published in many publically available documents. Erie has entered into an IGA with St. Vrain Sanitation District designating coordinated areas where Erie's historic planning area overlaps with St. Vrain's 208 planning area. Coordinated areas may be served by either the Town or the District depending on a variety of factors. Modifying the 208 Boundary to match will ensure recapture of wastewater for non-potable reuse and maximize use of a valuable resource. Maximizing reuse is consistent with the stated goals of NFRWQPA and DRCOG. The proposed 208 Boundary includes areas that are already within the Town limits or have petitioned to be annexed. Erie's existing distribution system is configured to easily provide potable water anywhere within the planning area. Additional lines will be added as development occurs. Construction of the NWRF allows the Town to service virtually all of the planning area by gravity. Erie has invested heavily in reuse infrastructure. Maximizing recapture of wastewater flows is an essential part of their water planning.

Historic Population Growth

The Town of Erie was founded by Reverend Richard Van Valkeburg, a Methodist preacher, who named the town after Erie, Pennsylvania. Like its namesake, Erie is dotted with coal mines dating from the 1880's. The Briggs Coal Mine was the first commercial coal mine in Weld County. The original plat for Erie was filed in 1871 and the Town was incorporated three years later in 1874. By 1877, Erie was home to approximately 600 residents (Town of Erie Historic Preservation Board, 2009).

Census data for the Town is presented in Table 4-1 and Figure 4-4. Erie experienced modest growth until 1990. Between 1990 and 2000, population increased by a factor of five from 1,258 to 6,291 residents. Since 2000, growth has averaged ten percent per year and tripled the Town's population. The Department of Local Affairs (DOLA) estimated the population in June 2007 at 14,189 residents (Colorado Department of Local Affairs 2009). Census data from 2010 indicates a population of 18,135 residents. Erie's rapid growth can be attributed to its convenient location, panoramic views, small town feel and rural atmosphere, reasonable real estate prices, reputable schools, and community amenities. Erie is a short twenty-five minute drive from either downtown Denver or nearby Boulder, Colorado.

Average home prices in Erie hovered around $231,000 at the end of 2009 which is nearly half the average price of $560,000 for a home in Boulder (AOL Real Estate, 2010). Home prices in nearby Broomfield and Lafayette averaged $270,000 and $312,000, respectively, while homes in Frederick were closer to $162,000 (AOL Real Estate 2010). Many of the single family homes in Erie are newer, having been built in the last ten years. Reasonable home prices coupled with an award winning, 63,000 square foot Community Center and 20,000 square foot Community Library make Erie a desireable place to live.

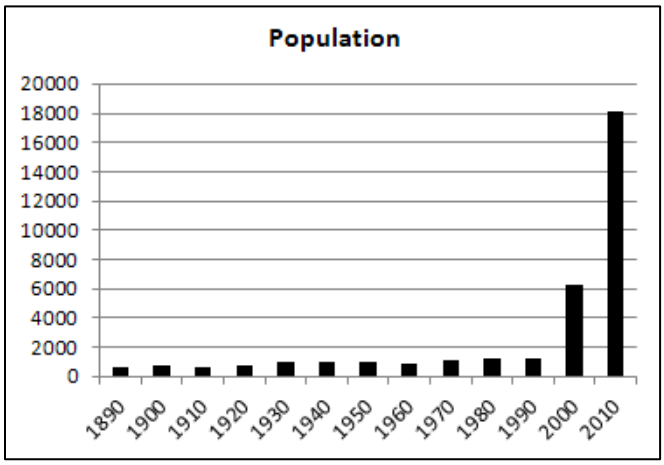


Figure 4-4 Historic Population Growth

Table 4-3 Historic Population Growth

Year	Population	Year	Population
1890	662	2000	6,291
1900	697	2001[a]	8,150
1910	596	2002[a]	8,591
1920	697	2003[a]	9,109
1930	930	2004[a]	10,264
1940	1,019	2005[a]	11,872
1950	937	2006[a]	13,478
1960	875	2007[a]	14,189
1970	1,090	2008[a]	16,030
1980	1,254	2009[a]	16,408
1990	1,258	2010	18,135

[a]Estimated by State Demographers Office

Estimated population data from DOLA is presented in Table 4-3 for years 2001 through 2008. The number of new water and wastewater taps issued by the Town from 2002 through 2009 are shown in Table 4-4. Growth has slowed over the last few years in response to the overall economic downturn in Colorado and the rest of the United States. Growth is cyclical and is expected to average six percent per year for through 2017 and four percent growth per year until buildout.

Table 4-4 New Taps Issued for Town of Erie, 2002 through 2009

	2002	2003	2004	2005	2006	2007	2008	2009
Water Taps	247	446	724	766	447	243	198	172
% Increase	7.3%	11.6%	15.8%	14.3%	7.7%	4.0%	3.2%	2.7%
Wastewater Taps	282	430	670	727	481	238	193	141
% Increase	8.7%	11.7%	15.4%	14.3%	8.7%	4.1%	3.2%	2.3%

Future Population Growth

In December 2005, Erie's Planning Commission adopted the Town's Comprehensive Plan. The Comprehensive Plan included input from diverse sources including citizens, the Erie Board of Trustees, the Comprehensive Plan Advisory Committee, and several planning consultants: Clarion Associates, LSA Associates, BBC Research, and EDAW. One of the many tasks that the comprehensive plan took on was a detailed analysis of future population growth in Erie's planning area. The Town considers it more accurate than projections by the State Demographers Office, DRCOG, and other estimates because the analysis was so detailed.

In past years, growth has been as high as 22% in one year and has averaged 10% per year since 2000. Future growth will also vary from year to year, but is expected to average 6% per year until the year 2017 followed by growth at 4% per year until buildout. Population projections from the Comprehensive Plan are presented in Table 4 -5. The buildout population of 68,820 residents will not completely fill all land-use parcels to their maximum capacities as defined by land use.

Knowing the estimated population for future years is not the same as knowing where and when growth will occur within the town limits. This information is critical for selecting alignments and diameters for future water and sewer pipes. Population was distributed among various land parcels using data from three sources: the 2009 land use map, the 2009 development map, and a spreadsheet of current and future development projects provided by Erie's planning department. The 2009 development map is shown in Figure 4-5. The spreadsheet contained a list of development names, number of dwelling units approved, number of units with certificates of occupancy, and number of unused or open units. The following assumptions were made prior to allocating future population:

Table 4-5 Future Population Growth

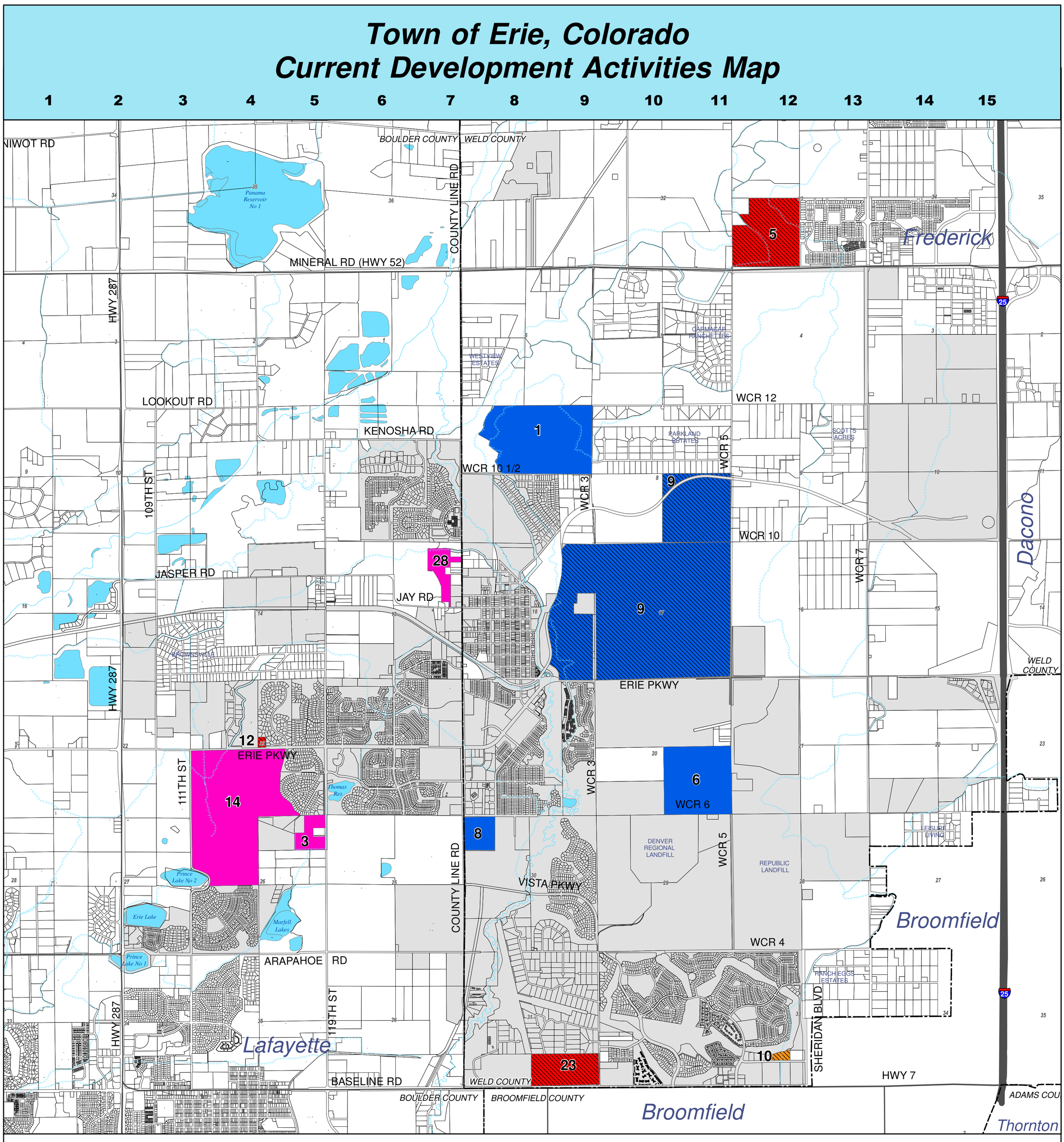
Date	Population
2010	18,135
2015	26,525
2020	33,525
2025	40,680
2030	49,625
Buildout	68,820

- Building permits issued as of January 2010 will be constructed and occupied prior to 2015.
- For developments with annexation agreements, but no building permits issued, dwelling units will begin construction in the 2015 to 2020 time frame.
- For areas without annexation agreements, growth will not occur until sometime after 2020.
- Dwelling units have an average of 2.8 residents.

The Town's land use GIS polygons were used for projecting population growth. A given polygon was calculated with a population in each year of concern -- 2010, 2015, 2020, 2025, 2030 and buildout. Land use polygons were "populated" at various dates. The various developments were identified in terms of GIS land use polygons and data were input into the polygons. A given polygon had a limit for number of units, and once the limit was reached, no additional population was allocated to that polygon. Note that a fixed value of 2.8 persons per dwelling unit was used in calculating the populations. For example, the Candlelight Ridge development had 67 building permits approved, 65 Certificates of Occupancy issued, and 31 units remaining. With 31 remaining units in 2009, this development was considered to reach build out by 2015.

In terms of the addition of population through time, existing developments were filled first, then areas with approved dwelling units were filled, then named developments with no data except number of dwelling units. From the development activities map, final plats were populated earlier than preliminary plats. Then, the annexation areas were populated. Finally areas with no named developments were populated based on acreage according Table 4-6. At each 5-year increment, population was added to individual polygons. When the population limit for that polygon or for the 5-year period was reached, population addition ceased.

Town of Erie, Colorado
Current Development Activities Map



LOCATION NUMBER	MAP NUMBER	PROJECT NAME	FILE NUMBER	PROJECT STATUS	DWELLING UNITS	NON-RESIDENTIAL SQUARE FOOTAGE	NUMBER ACRES	CURRENT ZONING	PROPOSED ZONING
ANNEXATION									
E-11	**5**	SMT INVESTORS 52/5	AN06-010	Active	415	To be determined	148	AG	CC/MR/LR
L-4	**12**	RICHTER	AN10-003	Active	2	n/a	3	AG	ER
Q-9	**23**	SIERRA VISTA	AN08-001	Active	80	To be determined	80	AG	CC/CMU/MR
COMPREHENSIVE PLAN AMENDMENT									
ENTIRE PLANNING AREA - COMPREHENSIVE PLAN UPDATE			CP10-001	Active	n/a	n/a	n/a	n/a	n/a
INITIAL ZONING/REZONING									
E-11	**5**	SMT INVESTORS 52/5	ZR07-002	Active	415	To be determined	148	AG	CC/MR/LR
L-4	**12**	RICHTER	ZR10-003	Active	2	n/a	3	AG	ER
Q-9	**23**	SIERRA VISTA	ZR08-001	Active	80	To be determined	80	AG	CC/CMU/MR
PD/PUD REZONING									

LOCATION NUMBER	MAP NUMBER	PROJECT NAME	FILE NUMBER	PROJECT STATUS	DWELLING UNITS	NON-RESIDENTIAL SQUARE FOOTAGE	NUMBER ACRES	CURRENT ZONING	PROPOSED ZONING
SPECIAL REVIEW USE									
Q-12	**10**	VISTA RIDGE F2, AMDMT 1 (PARCEL 35C)	SR10-005	Active	n/a	Telecom/Cellular Site	1	PD	n/a
PRELIMINARY PLAT									
G-8	**1**	MORGAN HILL	SK08-005	Active	490	n/a	216	LR/SR/AG-OS	n/a
L-10	**6**	SUNSET	PP08-002	Active	250	n/a	157	LR	n/a
M-7	**8**	COUNTY LINE BUSINESS PARK	PP08-003	Active	n/a	To be determined	40	LI	n/a
J-10	**9**	BRIDGEWATER	PP-11-00005	Active	700	n/a	240	LR/PUD	n/a
FINAL PLAT									
M-5	**3**	REX RANCH FIL NO 1	FP09-002	Active	57	n/a	54	LR	n/a
J-10	**9**	BRIDGEWATER MASTER PLAT	MS-11-00029	Active	n/a	n/a	933	Multiple	n/a
M-3	**14**	FLATIRON MEADOWS FIL NO 1	FP08-002	Active	72	n/a	53	SR/PUD	n/a
M-3	**14**	FLATIRON MEADOWS FIL NO 2	FP08-003	Active	117	n/a	69	SR/PUD	n/a
I-6	**28**	LOST CREEK FARM	Old Code	Active	46	n/a	30	SR	n/a
SITE PLAN									
Q-12	**10**	VISTA RIDGE F2, AMDMT 1 (PARCEL 35C)	SPA10-005	Active	n/a	Telecom/Cellular Site	1	PD	n/a

NOTES:
1. This map is intended to serve as a guide for future land use patterns within the Town of Erie's Planning Area Boundary and is advisory in nature. Land Use patterns depicted on the map are generalized, recognizing that developement proposals may contain a mixture of land uses and density levels which achieve the intent of the Town of Erie Comprehensive Plan.
2. Developments located on this map have completed and accepted applications in the development review process.
3. Cross-hatched areas indicate multiple development applications are being processed for a property, and the color indicates where they first appear on the procedure charts.

CURRENT AS OF: JULY 20, 2011

PREPARED BY: DEPARTMENT OF COMMUNITY SERVICES, D. JENKINS

0 0.25 0.5 1 Miles

Figure 4-5 2011 Development Map

Table 4-6 Maximum Dwelling Units per Acre

Land UseType	Units per Acre
Rural Residential	1
Low Density Residential	4
Medium Density Residential	8
High Density Residential	16
Multi-Use	8 -- using 50% of acreage

Given the priority of developments on the "List of Residential Developments by County," the named developments "filled up" during the years 2010 to 2025. Other areas, including the annexation areas on the developments map, were populated from 2025 to buildout. Some of the areas calculated by acreage did not "fill up" by the time a population of 68,820 was reached.

Populations were distributed based on the best available information in January 2010. Certain developments may proceed faster or slower than predicted. Similarly, some areas of the Town may ultimately reach higher population densities than predicted by the allocation methods used.

WASTEWATER FLOWS

Historic wastewater flows at the SWRF are shown in Figure 4-6 and Tables 4-7 and 4-8. Annual average influent flows have nearly doubled between 2003 and 2010 increasing from 0.578 mgd to 1.007 mgd. The highest recorded annual average flow was 1.007 mgd in year 2010. Increased flows correspond to population growth within the service area. The maximum month flow of 1.091 mgd recorded in June 2010 represents sixty-eight percent (68%) of the rated hydraulic capacity of 1.6 mgd.

The maximum month peaking factor is calculated by dividing the average daily flow for the year by the average daily flow of the highest flow month. The maximum month peaking factor has ranged between 1.05 and 1.13 over the last eight years. The maximum month flow peaking factor is used for sizing of chemical storage and biological secondary treatment processes. The maximum month peaking factor can be indicative of excessive inflow and infiltration (I&I) flows and/or a large number of seasonal residents or tourists. For Erie, influent flows do not vary substantially from month to month which suggests low I&I and a stable population base.

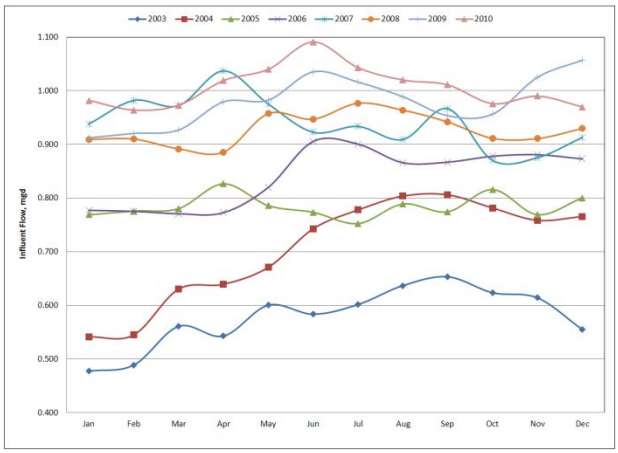


Figure 4-6 Influent Flow at SWRF

Minimum and maximum daily flows measured at the SWRF are summarized in Table 4-8 for the period from 2003 through 2010. The minimum and maximum daily flows are the lowest and highest daily flows measured for single days during an entire year. Peaking factors are calculated for each year by dividing the average daily flow for the year by the lowest and highest flows for individual days. Peaking factors for minimum daily flows ranged between 0.46 and 0.864 while peaking factors for the maximum daily

flows ranged between 1.18 and 1.57. Maximum day peaking factors for the 95th and 99th percentile frequency over the seven-year record are 1.34 and 1.38, respectively.

Table 4-7 Monthly Average Influent Flow, mgd

	2003	2004	2005	2006	2007	2008	2009	2010
January	0.477	0.541	0.769	0.777	0.938	0.909	0.912	0.982
February	0.489	0.545	0.776	0.775	0.982	0.910	0.920	0.964
March	0.561	0.630	0.780	0.771	0.972	0.892	0.927	0.973
April	0.543	0.639	0.827	0.773	1.037	0.886	0.979	1.019
May	0.600	0.671	0.786	0.820	0.975	0.957	0.983	1.040
June	0.584	0.743	0.773	0.906	0.923	0.947	1.035	1.091
July	0.601	0.778	0.752	0.901	0.933	0.977	1.016	1.043
August	0.637	0.804	0.789	0.866	0.909	0.964	0.989	1.020
September	0.653	0.806	0.774	0.867	0.967	0.942	0.953	1.011
October	0.623	0.782	0.816	0.878	0.870	0.911	0.956	0.976
November	0.614	0.758	0.769	0.881	0.875	0.911	1.025	0.990
December	0.555	0.766	0.800	0.873	0.913	0.930	1.057	0.970
Average	0.578	0.705	0.784	0.841	0.941	0.928	0.979	1.007
MM	0.653	0.806	0.827	0.906	1.037	0.977	1.057	1.091
PF	1.13	1.14	1.05	1.08	1.10	1.05	1.09	1.08

A previous planning effort calculated peak day peaking factors between 1.34 and 2.78 (Burns and McDonnell, 2008). The 2.78 peaking factor was considered not representative of actual influent conditions. Instead, a peaking factor of 1.82 was selected; being representative of the middle of the range. The 2.78 peaking factor is based on single day of flow measured on May 10, 2003 of 1.61 mgd. Flood records for Coal Creek and Boulder Creek indicate that the creeks flooded on May 30, 2003, but did not reach the 50-year flood level (Daily Camera 2010). It is unlikely that flooding impacted the flow measurement on May 10, 2003. This flow was more than three times the daily flow observed either before or after that date. It appears that this data point is a data entry error. After discussing the data point with Erie staff, it was eliminated from the data set. Eliminating the single erroneous data point dropped the peak day peaking factor to 1.38.

Table 4-8 Minimum and Maximum Daily Influent Flow, mgd

	2003	2004	2005	2006	2007	2008	2009	2010
AAF	0.578	0.705	0.784	0.841	0.941	0.928	0.967	1.007
Min Day	0.429	0.427	0.626	0.68	0.79	0.425	0.824	0.864
PF	0.74	0.61	0.80	0.81	0.84	0.46	0.85	0.86
Max Day	0.906	0.860	1.086	0.998	1.148	1.103	1.139	1.395
PF	1.57	1.22	1.38	1.19	1.22	1.19	1.18	1.39

Flow reported by client for May 10, 2003 was 1.61 mgd. This flow is three times the previous daily flow and is likely a typographical error. Data point was deleted from data set.

Selection of Peak Hour Factor

Influent flows vary diurnally with peak hour flows typically occurring between about eleven o'clock in the morning and three in the afternoon. As the service area increases in size, the peak hour flow tends to

occur later in the day and to become less pronounced. The time delay is a result of travel time through an expanded collection system while flattening of the diurnal curve is a result of both attenuation of flows and greater diversity of flow sources. Peak hour flows are used to size collection system pipes, lift stations, pumps, headworks equipment, and disinfection equipment. For design, a peak hour peaking factor is typically determined by taking a minimum of three years of influent flow data and calculating a peak hour factor for each day. The peak hour factor is the average daily flow divided by the peak hour flow for the day.

Wastewater enters the SWRF through a gravity sewer and influent pump station. Influent flow measurements are taken in the pump station discharge pipeline. The wet well fills at an unknown rate before being pumped periodically into the SWRF. For this reason, peak flows measured at this location are not meaningful. In the absence of historic data, a peaking factor can be estimated using typical factors based on population or by prescribed mathematical formulas.

Table 4-9 Ranges of Hourly Peaking Factors

Population, 1,000s	Hourly Peaking Factor	
	Minimum	Maximum
1	3.9	5.5
5	3.3	4.5
10	2.9	4.0
50	2.3	2.9
100	2.0	2.6
200	1.8	2.3
500	1.5	1.9

Source: *Gravity Sanitary Sewer Design and Construction*, MOP FD-5, WEF, 1982.

Representative peaking factor ranges are listed in Table 4-9 (*Gravity Sanitary Sewer Design and Construction* manual, MOP FD-5, WEF, 1982). The Town of Erie's hourly peaking factor is likely between 2.9 and 4.0 based on the 2010 population of 18,135 people. As indicated in Table 4-9, hourly peaking factors tend to decrease as the population and service area increase. Peaking factors selected for modeling and design should reflect current conditions with a minimum hourly peaking factor of 2.9. As population increases over time, the peak hour peaking factor will decrease. For a buildout population of 68,820 residents, a peak hour peaking factor in the range of 2.0 to 2.9 would be typical. However, future flows will be split between the SWRF and NWRF which effectively reduces the collection system area for each facility and increases the peak hour flows.

Previous planning efforts assumed 1.2 MGD would be treated at the SWRF and 3.6 MGD would be treated at the NWRF for a combined treatment capacity of 4.8 MGD (Burns and McDonnell June 2008). This represents the 15-year planning horizon and a population of 40,680 residents by the year 2025. Additional capacity will be needed to meet the ultimate buildout condition of 68,820 residents. The SWRF was rerated for 1.6 MGD in 2006 and will treat a maximum month flow equal to the rerated capacity in the future.

The DRCOG Wastewater Utility Plan Guidance Manual and NFRWQPA recommend the following equation for calculating peak hour flows:

$$Peaking\ Factor = \frac{3.65}{Q_{average\ total}{}^{0.167}}$$

Table 4-10 Estimated Future Wastewater Generation Rates for Overall System

Year	Population	Influent Flow, MGD		Peak Hour PF (DRCOG)	Peak Hour Flow, mgd
		AAF	MM		
2010	18,135	1.26	1.45	3.51	4.42
2015	26,525	1.86	2.12	3.29	6.12
2020	33,525	2.35	2.68	3.16	7.43
2025	40,680	2.85	3.25	3.06	8.72
2030	49,625	3.47	3.96	2.97	10.31
Buildout	68,820	4.82	5.49	2.81	13.54

AAF = Annual Average Flow, MM = Maximum Month Average, PF = Peaking Factor
Estimated future flows are based on a per capita generation rate of 70 gpcd.

Table 4-11 Estimated Peak Hour Peaking Factors and Peak Hour Flows by Facility

SWRF			NWRF		
MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD	MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD
1	3.73	3.73	1.5	3.49	5.23
1.2	3.62	4.34	3.6	3.01	10.84
1.6	3.45	5.52	3.9	2.90	11.34

Note: MMF is the average daily flow during the maximum month. Peak hour peaking factors are calculated using the annual average flow. The maximum month peaking factor is 1.14

The DRCOG equation was developed using data for Colorado communities and is more representative than the ranges referenced in the Gravity Sewer Design and Construction Manual. The DRCOG equation was applied to the estimated future wastewater generation rates for Erie. Future flows and load are discussed in detail towards the end of this section on page 4-19. Peak hour peaking factors are presented in Tables 4-10 and 4-11 along with estimated future wastewater flows. Peak hour peaking factors range between 2.9 at buildout to 3.73 under current conditions. The calculated peaking factors agree with the recommended ranges in MOP FD-5. Peak hour peaking factors are higher at the SWRF because a smaller portion of the service area will be served by this facility.

CHARACTER OF INFLUENT

The Erie SWRF collects influent and effluent samples for permit compliance once per week. Analyses include biochemical oxygen demand (BOD), total suspended solids (TSS), and ammonia for both influent and effluent. Operations staff also record daily effluent temperature and pH. Data collected between 2003 and 2009 are presented in this section. Wastewater characteristics for the Town have been developed based on a eight-year record of flows and waste strength from 2003 through 2010.

Temperature

Influent water temperature is not measured however, effluent water temperature is measured daily by operations staff and is recorded on the daily operations bench sheet. Effluent water temperatures are likely to be slightly higher than influent water temperatures by one or two degrees Celsius. Average daily effluent wastewater temperature data is presented in Table 4-12. Water temperatures range between 14.8 and 21.2 degrees Celsius (58 and 70 degrees Fahrenheit) with warmer water temperatures measured in the summer months.

Table 4-12 Monthly Average Effluent Temperature, Degrees Celsius

	2003	2004	2005	2006	2007	2008	2009	2010	Average
January	16.5	15.8	15.9	15.4	15.1	15.8	16.6	16.1	15.9
February	16.0	15.2	15.1	14.9	14.8	15.6	16.2	15.7	15.4
March	15.9	15.7	14.9	15.5	15.8	15.7	16.3	15.9	15.7
April	17.2	16.7	15.9	16.7	16.2	16.2	16.7	16.5	16.5
May	17.8	18.9	17.3	17.6	17.5	17.3	17.9	17.3	17.8
June	19.6	20.1	18.6	19.4	18.6	18.7	19.0	18.8	19.1
July	20.3	20.0	19.6	20.6	19.9	20.0	20.1	20.2	20.1
August	20.5	20.3	20.3	21.2	21.1	21.0	20.8	21.1	20.7
September	19.8	19.8	20.8	20.5	20.9	21.1	20.9	21.0	20.5
October	18.7	18.6	19.4	19.5	20.2	20.2	19.8	20.8	19.5
November	17.5	17.2	18.0	17.9	18.4	19.0	No Data	19.1	18.0
December	16.7	16.0	16.5	16.6	16.7	17.5	No Data	17.9	16.6
Average	18.0	17.9	17.7	18	17.9	18.2	18.4	18.4	

Water temperature affects treatment plant design and operation. For every 10 degrees Celsius increase in water temperature, biological activity doubles. At colder temperatures, the microorganisms in the aeration basin grow and metabolize organic matter slower. Certain populations of microorganisms like the nitrifying bacteria are especially sensitive to low temperatures. Nitrification essentially stops when water temperatures reach 5 degrees Celsius and colder. Because of this, minimum water temperatures will control aeration basin size and digester efficiency and may affect other aspects of facility design and operation.

Biological Oxygen Demand

Biochemical Oxygen Demand (BOD) is a bulk measurement of the organic compounds present in wastewater. Wastewater contains proteins, carbohydrates, sugars, fats, and complex organic compounds. There are too many different compounds to measure each one separately even if were possible to identify all of them. Instead, we measure the amount of oxygen used when bacteria consume the biologically available organics. The BOD test is run by aliquoting a known volume of wastewater into a special sample bottle. The remaining volume is filled with dilution water that contains salts and nutrients for the bacteria. The dissolved oxygen concentration is measured in the bottle. The bottle is then placed into an incubator at 20 degrees Celsius for five days. At the end of the testing period, the bottle is removed and the dissolved oxygen concentration is measured a second time. The difference between the initial and final dissolved oxygen concentrations is the biological oxygen demand of the sample.

Influent BOD concentration data for the SWRF influent is presented in Table 4-13. Annual average BOD concentrations have averaged between 265 and 310 mg/L. Monthly average BOD concentrations have been as low as 210 mg/L and as high as 402 mg/L. Domestic wastewater is typically in the range of 110 to 350 mg/L of BOD (Metcalf and Eddy, 2003). Average BOD concentrations have increased over the last seven years. The gradual increase is a result of newer construction, low flow appliances, and reduced inflow and infiltration.

Table 4-13 Monthly Average Influent BOD, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010
January	277	314	269	331	316	373	303	264
February	296	351	247	315	258	402	340	247
March	271	291	282	327	262	317	344	265
April	282	278	273	325	237	322	337	270
May	307	300	270	268	247	301	288	272
June	285	260	269	315	264	265	283	227
July	265	241	295	306	289	269	284	227
August	210	228	298	306	277	262	298	206
September	245	222	288	284	262	272	285	247
October	238	223	296	266	291	278	332	329
November	240	223	339	287	308	330	297	274
December	296	253	319	282	384	331	296	377
Average	268	265	287	301	283	310	307	265

On an annual average basis, BOD_5 ranged from a minimum of 1,280 ppd (in 2003) to a maximum of 2,141 ppd (in 2009). As shown in Table 4-14, maximum monthly loads ranged from 1,543 ppd (in 2003) to 2,960 ppd (in 2010). The maximum month BOD load of ,2960 ppd recorded in October 2010 represents 76.5 percent of the permitted organic capacity of 3,870 ppd for the SWRF.

Table 4-14 Monthly Average Influent BOD, ppd

	2003	2004	2005	2006	2007	2008	2009	2010
January	1,076	1,417	1,725	2,142	2,476	2,708	2,210	2,029
February	1,207	1,589	1,597	2,037	2,110	2,849	2,427	1,880
March	1,246	1,531	1,838	2,100	2,120	2,286	2,474	2,215
April	1,282	1,470	1,880	2,091	2,044	2,344	2,537	2,138
May	1,543	1,682	1,758	1,817	2,005	2,325	2,257	2,256
June	1,387	1,612	1,736	2,381	2,021	2,057	2,390	1,981
July	1,330	1,561	1,852	2,301	2,245	2,150	2,386	1,921
August	1,127	1,529	1,954	2,209	2,027	2,028	2,400	1,743
September	1,329	1,490	1,860	2,054	2,005	2,041	2,273	1,963
October	1,239	1,454	2,014	1,967	2,005	2,008	2,658	2,565
November	1,225	1,413	2,178	2,107	2,135	2,394	2,272	2,158
December	1,367	1,619	2,127	2,052	2,817	2,492	2,378	2,960
Average	1,280	1,530	1,877	2,105	2,168	2,307	2,389	2,141
MM	1,543	1,682	2,178	2,381	2,817	2,849	2,658	2,960
PF	1.21	1.10	1.16	1.13	1.30	1.24	1.11	1.38

Total Suspended Solids

Total Suspended Solids (TSS) can include a wide variety of material, such as silt, decaying plant and animal matter, grease, fecal material, fibers, and more. The TSS test does not include large debris such as rags. The total suspended solids are determined by filtering a known volume of sample through a weighted glass-fiber filter disc in an appropriate filtering apparatus. The filter and the trapped solids are oven dried at 103° -105°C, cooled in a desiccator, and subsequently weighed. The increase in filter

weight represents the suspended solids. TSS is a significant parameter in terms of estimating the organic strength of the wastewater, evaluating solids loading and solids capture, calculating various process control parameters and estimating the effects of the effluent discharge on the receiving stream.

Influent suspended solids concentrations and loads for the Erie SWRF influent are presented in Tables 4-15 and 4-16, respectively. Because composite samples are collected after screening and grit removal, TSS results should not include contributions from grit. For 2009, influent TSS concentrations averaged 405 mg/L and ranged between 374 and 453 mg/L. Domestic wastewater is typically in the range of 120 and 400 mg/L (Metcalf and Eddy, 2003). Higher concentrations at the Erie SWRF are a result of low per capita water use and a lack of inflow and infiltration. Between 2003 and 2010, annual average TSS loading has increased from 1,271 to 3,195 ppd. Loading increases track population increases over the same time period.

Table 4-15 Monthly Average Influent TSS, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010
January	287	349	313	357	417	448	403	395
February	296	390	351	344	334	461	392	395
March	288	329	369	387	357	361	453	388
April	311	327	311	395	362	491	386	430
May	274	352	370	388	391	401	374	443
June	284	349	361	403	425	382	396	434
July	244	318	350	417	392	423	416	461
August	206	313	372	347	355	361	452	356
September	206	317	372	345	326	363	388	397
October	239	323	375	344	384	350	394	375
November	251	331	380	356	410	409	357	367
December	305	298	375	369	484	391	423	396
Average	266	333	358	371	386	403	402	402

Table 4-16 Monthly Average Influent TSS, ppd

	2003	2004	2005	2006	2007	2008	2009	2010
January	1,145	1,574	2,008	2,316	3,269	3,255	2,903	3,043
February	1,206	1,768	2,275	2,220	2,736	3,268	2,802	3,014
March	1,353	1,727	2,403	2,492	2,913	2,605	3,253	3,185
April	1,418	1,731	2,136	2,547	3,141	3,577	2,903	3,411
May	1,379	1,956	2,418	2,672	3,180	3,097	2,927	3,689
June	1,384	2,158	2,331	3,040	3,261	2,987	3,350	3,794
July	1,223	2,060	2,197	3,139	3,044	3,354	3,499	3,893
August	1,096	2,099	2,443	2,504	2,608	2,806	3,637	3,015
September	1,122	2,126	2,401	2,498	2,499	2,731	3,091	3,151
October	1,243	2,107	2,555	2,550	2,655	2,522	3,158	2,923
November	1,281	2,094	2,436	2,614	2,836	2,972	2,727	2,890
December	1,407	1,908	2,502	2,689	3,545	2,943	3,410	2,483
Average	1,271	1,942	2,342	2,607	2,974	3,010	3,138	3,195
MM	1,418	2,158	2,555	3,139	3,545	3,577	3,637	3,893
PF	1.12	1.11	1.09	1.20	1.19	1.19	1.16	1.22

Ammonia Nitrogen

Nitrogen is an essential nutrient for plants and animals. Approximately 80 percent of the earth's atmosphere is composed of nitrogen and it is a key element of proteins and cells. The major contributors to nitrogen in wastewater are food preparation, showering, and waste excretion. Per capita generation rates range from 0.014 to 0.025 pounds per capita day (Metcalf and Eddy 2003).

For wastewater facilities that are required to nitrify completely, the secondary process oxygen demand from nitrification can easily reach 20 to 30 percent of the total oxygen demand. Ammonia oxidation requires 4.5 pounds of oxygen for every pound of ammonia oxidized. BOD removal requires 1.0 to 1.2 pounds of oxygen per pound of BOD removed. If the influent BOD concentration is 300 mg/L and the influent ammonia concentration is 25 mg/L, the oxygen needed to support full nitrification will be twenty-five percent of the total aeration requirement. Some of the oxygen demand may be offset through denitrification. Accurately predicting future ammonia concentrations and loads impacts blower sizing and aeration basin sizing.

Influent ammonia concentrations and loads are presented in Tables 4-17 and 4-18, respectively. Influent ammonia concentrations typically range between 20 and 80 mg/L as N for domestic wastewater (Metcalf and Eddy 2003). Higher concentrations are often measured in resort communities and areas with very low water use per capita. For Erie, ammonia concentrations have averaged between 31.0 and 42.1 mg/L on an annual average basis. Monthly average concentrations have been as high as 47.1 mg/L which reflects both the composition of Erie's service area which is predominantly residential and low per capita water use.

Table 4-17 Monthly Average Influent Ammonia, mg/L as N

	2003	2004	2005	2006	2007	2008	2009	2010
January	30.9	38.8	27.0	30.5	37.5	41.4	42.2	40.2
February	32.7	38.5	28.8	30.5	30.5	30.8	42.1	42.7
March	34.5	33.9	30.2	37.1	33.1	39.5	43.6	39.6
April	30.7	31.6	29.3	36.2	35.0	37.7	39.3	38.2
May	30.2	33.8	26.2	29.7	32.2	35.2	46.0	45.4
June	29.6	26.9	27.5	30.4	32.2	36.8	38.2	47.1
July	28.6	29.5	28.9	33.6	36.5	33.7	37.9	40.8
August	31.2	34.6	31.0	30.1	34.1	37.9	40.7	39.8
September	28.9	26.9	35.3	32.9	32.0	40.6	36.5	44.3
October	27.6	26.2	32.6	33.5	36.9	41.6	38.2	42.8
November	28.8	28.4	35.6	35.4	34.0	44.2	39.6	42.3
December	38.4	28.7	33.2	34.5	35.1	40.5	44.6	41.5
Average	31.0	31.5	30.5	32.9	34.1	38.3	40.7	42.1

The ammonia load has increased from an annual average load of 149 pounds per day in 2003 up to 334 pounds per day in 2010. The ammonia load has doubled in eight years and corresponds to population growth in the service area. Maximum month peaking factors range between 1.13 and 1.23 which is lower than the typical maximum month peaking factor of 1.25 published by Water Environment Federation (Water Environment Federation 2005).

Table 4-18 Monthly Average Influent Ammonia, ppd

	2003	2004	2005	2006	2007	2008	2009	2010
January	123	175	173	198	294	300	305	311
February	133	175	187	197	250	218	301	324
March	159	178	196	238	268	285	313	321
April	139	167	202	233	303	273	296	303
May	151	189	171	201	262	272	359	376
June	144	167	177	228	247	286	322	412
July	143	192	181	253	285	268	319	345
August	164	232	203	218	250	292	328	338
September	157	180	228	238	245	304	290	352
October	143	171	222	246	253	300	307	334
November	148	179	229	261	235	321	303	334
December	178	184	222	251	255	303	358	260
Average	149	182	199	230	262	285	317	334
MM	178	232	229	261	303	321	359	412
PF	1.20	1.27	1.15	1.13	1.16	1.13	1.13	1.23

Total Kjeldahl Nitrogen

In addition to ammonia nitrogen, the influent wastewater contains significant quantities of nitrogen in organic compounds. Most of the organic nitrogen is contained in proteins or their degradation products. In the treatment process, approximately two-thirds of the organic nitrogen is hydrolyzed to ammonia. The ammonia released by oxidation of organic compounds must be added to the influent ammonia to determine total oxygen demands for nitrification.

The Total Kjeldahl Nitrogen (TKN) is a measure of the total reduced nitrogen in the form of ammonia and organic nitrogen. The Erie SWRF has no historical TKN data. TKN to ammonia ratios tend to be fairly constant for domestic wastewater from one facility to another. For process evaluation purposes, TKN loads were assumed to be 1.67 times the projected influent ammonia loads. The corresponding NH_3-N:TKN ratio is 0.60 pounds per pound.

Fats, Oils, and Greases

The term oil and grease, as commonly used, includes the fats, oils, waxes and other related constituents found in wastewater. Oil and grease may be present in wastewater as free oil, dispersed oil, emulsified oil, or dissolved oil (Rhee, Martyn, and Kremer 2006). Oils and greases are contributed to domestic wastewater in butter, lard, margarine, and vegetable fats and oils. Fats are also commonly found in meats, in the germinal area of cereals, in seeds, nuts, and in certain fruits. Kerosene, lubricating, and road oils are derived from petroleum and coal tar. These oils sometimes reach the sewers in considerable volumes from shops, garages, and streets. For the most part, they float on the wastewater, although a portion is carried into the sludge on settling solids. Oil and grease will cause a visible sheen on water at a concentration of about 10 mg/L (United States Environmental Protection Agency 2010).

Oils and greases are problematic in collection systems and wastewater treatment plants for a variety of reasons. In collection systems, oils and greases coat the insides of pipes, manholes, and pumps thereby reducing their capacity. Petroleum-based oils and greases are more problematic in this regard than vegetable and animal fats. As much as forty to fifty percent of sanitary sewer blockages are related in some way to oil and grease discharges into the collection system (Newton 2003).

Once oils and greases enter a treatment plant, they can stick to the sides of channels and wet wells, interfering with level sensing equipment (Bowen and Poretta, 2001). Some facilities have experienced clogging of pores on fine screens and rotary screens. The low solubility of oils and greases reduces their rate of microbial degradation and can cause them to accumulate in biological treatment processes like activated sludge. It has been suggested that oils and greases may be contributing factors to the promotion of Nocardia growth and the foaming in the aeration basins (WEF MOP 11, 1998) and to digester foaming problems. The SWRF periodically experiences severe foaming episodes that may be related in part to oil and grease.

The best way to minimize the impact of oil and grease on both the collection system and the treatment plant is to control the source. The Town of Erie has completed a draft ordinance for the pretreatment of grease that will require all commercial and industrial customers that have a high probability of discharging significant quantities of grease to install grease interceptors. The draft ordinance has provisions for grease trap design and installation, but does not establish an actual limit on the quantity of oil and grease that may be discharged to the Town sewer system nor does it mandate maintenance procedures, frequency of cleaning, or record keeping requirements. The Denver Metro Reclamation District and the Littleton Englewood WWTP have set local limits on oil and grease for all of their commercial and industrial dischargers. The L/E WWTP limit is 75 mg/L (Gardener, 2005). Setting a local limit gives the Town legal recourse when a trap is not functioning properly even when it is maintained and cleaned at regular intervals. At present, the Town performs routine inspections of grease traps or maintenance records. A review of the most recent published version of the Town Municipal Code does not reference grease traps (Town of Erie 2009). Thus, the draft ordinance has not been formally adopted.

Alternative technologies to clean interceptors and lift stations are bioremediation and bioaugmentation using extracted enzymes and/or bacteria cultures. The use of enzymes, either extracted or in bacterial cultures, emulsifies the free, floating oils and greases into a liquid or semi-solid state. This is accomplished by breaking the glycerol-fatty acid bonds. The end products in this reaction are fatty acids and glycerol (Metcalf and Eddy, 2003). However, the oils and greases may not remain emulsified when enzymes are used alone. Southgate Sanitation Town has acknowledged some problems downstream in the sewer lines from sources that use enzymes in lieu of pumping out the grease interceptor on a regular interval. Kent County, Delaware only allows the use of grease enzymes in instances where an existing operation cannot install grease interception or traps (Newton, 2003).

Two companies market a product that adds bacteria as an additional treatment step following enzyme addition. They are BioBlock or Bio-Charge 200 (East Coast Environmental, Inc.) and Lipase© (Sigma

Chemical Corp.). The City of Edmonton, Canada used biochemical treatment of various forms (liquid spray, mixed spray, solid block) in multiple lift stations with limited success (Bowen and Porretta, 2001). A study by Lei et al (2003) investigated the impact of adding Lipase©, BOD Balance, or nothing to rendering wastewater prior to digestion by aerobic and anaerobic means. In each case, biochemical treatment improved overall digestion performance. Soluble COD removal increased by 203.7% under aerobic conditions and 180.1% under anaerobic conditions.

Wastewater Parameter Ratio Analysis

Domestic wastewater has a similar chemical composition from one municipality to another. Large industrial users within a service area can skew the chemical composition; however, even the effects of significant industrial users tend to be averaged out over a large service area because their contributions are small relative to the total flow received by the WWTP. Wastewater modeling and treatment plant design both depend on being able to make reasonable assumptions about the composition of the influent wastewater when actual data are not available. For instance, individual residents tend to generate between 70 and 120 gallons per day per person and about 0.2 pounds of BOD per person. Domestic wastewater also tends to adhere to a range of ratios for some constituents like BOD, TSS, and ammonia.

For domestic wastewater, influent BOD and TSS tend to be roughly equal to one another, ranging between 80 and 120 percent. This is true because most of the solids present in the wastewater are organic in nature. The BOD test is simply a bulk measure of edible organics which means that most of the solids entering the plant can be measured as BOD. The inorganic solids such as grit, eggshells, and hair do not contribute to the BOD load. Not all BOD is particulate. Some BOD is soluble and is not measured in the TSS test. The fraction of soluble BOD depends on the businesses present in the service area and the length of the collection system. Breweries, for example, discharge high quantities of soluble BOD. A sprawling collection system can enhance the breakdown of particulate BOD to soluble BOD during its journey to the treatment plant. Generally, the amount of soluble BOD tends to be small relative to the amount of particulate BOD. Because of these variations in particulate versus soluble, the ratio of BOD to TSS is not absolute and varies with one constituent or the other sometimes being present in greater quantities. Still, the expected ratio of BOD to TSS or TSS to BOD is between 0.8 and 1.2 pounds per pound.

Organic matter – plant material, human waste, etc. – contains between six and twelve percent nitrogen by weight and between one and two percent nitrogen. For every hundred pounds of BOD in the influent, there should also be between six and twelve pounds of nitrogen and one to two pounds of phosphorus.

Wastewater characteristic ratios for the SWRF influent are presented in Table 4-19. To create the ratio results presented, only paired data sets were used. Paired data sets are days where a BOD, TSS, and ammonia result were available for the same composite sample. Ratios cannot be calculated for data collected from different samples – especially samples collected on different days – because of natural variations in the wastewater received from moment to moment and because of potential errors introduced by sampling. Ratios were calculated for each individual sample and then statistics were calculated for each parameter. On average, the influent BOD to TSS ratio is 0.84 or 84 percent. This is consistent with

domestic wastewater influent ratios published for hundreds of other facilities. The influent ammonia to BOD ratio averages 0.12 and ranges between 0.06 and 0.21 or six to twenty-one percent.

Table 4-19 Wastewater Parameter Ratios for Town of Erie

	Influent BOD/TSS	Influent NH_3/BOD	Effluent BOD/TSS
Minimum	0.43	0.06	0.11
25th Percentile	0.73	0.10	0.43
Average	0.84	0.12	0.53
75th Percentile	0.91	0.13	0.60
Maximum	1.48	0.21	1.87

Influent and Effluent BOD/TSS Ratios are typically between 0.8 and 1.2 for domestic wastewater.
Influent NH_3/BOD Ratio is typically between 0.067 and 0.134 for domestic wastewater.

The wastewater parameter ratios for the Erie SWRF fall within expected ranges with minimal variation. This is indicative of good, representative sampling and consistent laboratory analysis technique. The data is internally consistent which means that results for different parameters agree well with one another. Internally consistent data collected over a long period of time means that past and future wastewater characteristics can be determined/predicted with a high degree of confidence.

Per Capita Loading

Average wastewater flows per person, shown in Table 4-20, were calculated by dividing the influent flows and loads for each month by the estimated population for that year. Calculation of per capita loading for BOD, TSS, and ammonia were done similarly.

Table 4-20 Per Capita Generation Rates for Town of Erie, 2001-2008

		Influent Flow, gpcd		BOD, ppcd		TSS, ppcd		NH3-N, ppcd	
Year	Population[1]	AAF	MM	AA	MM	AA	MM	AA	MM
2001[4]	8,281	59.7	68.1	0.14	0.16	0.15	0.19	0.012	0.014
2002[4]	9,028	54.9	58.3	0.13	0.15	0.13	0.17	0.014	0.016
2003	9,109	63.5	71.7	0.14	0.17	0.14	0.16	0.016	0.020
2004	10,264	68.7	78.5	0.15	0.16	0.19	0.21	0.018	0.023
2005	11,872	66.1	69.6	0.16	0.18	0.20	0.22	0.017	0.019
2006	13,478	62.4	67.2	0.16	0.18	0.19	0.23	0.017	0.019
2007	14,189	66.3	73.1	0.15	0.20	0.21	0.25	0.018	0.021
2008	14,958	62.0	65.3	0.15	0.19	0.20	0.24	0.019	0.021
2009	16,408	59.7	64.4	0.15	0.16	0.19	0.22	0.019	0.022
2010	18,135	55.5	59.5	0.16	0.23	0.21	0.26	0.022	0.028
Average		61.9	67.6	0.15	0.18	0.18	0.22	0.017	0.020
Normal Range[2]		40-130		0.13 - 0.33		0.11 - 0.26		0.011-0.026	

[1] (Colorado) Population data from Colorado State Demographer's Office: Accessed 12/07/2009
[2] (Metcalf and Eddy, 2003) Table 3-12
[3] AAF = Annual Average Flow, MM = Maximum Month Average, AA = Annual Average
[4] Data for 2001 and 2002 were taken from the 2008 Wastewater Utility Plan Update

Between 2001 and 2010, the annual average per capita flows for the Town averaged between 56 and 69 gpcd and maximum month generation rates varied between 60 and 73 gpcd. Per capita wastewater generation rates have been very consistent. Generation rates for Erie fall in the middle of the published

range of typical wastewater generation rates (Metcalf and Eddy 2003). Per capita generation rates have been decreasing in many Colorado communities as a direct result of consumer awareness and water conservation efforts, installation of low water use appliances and fixtures, and low inflow and infiltration. Per capita flow generation rates published in utility plans for other municipalities include: Meeker Sanitation District at 87 gpcd, The Town of Carbondale at 87 gpcd, Saint Vrain Sanitation District at 85 gpcd, and the City of Greeley at 81 gpcd. Erie's generation rates are slightly lower at 67 to 73 gpcd and may reflect a greater percentage of newer construction and low flow fixtures than these other communities. Additionally, the Town has been aggressive in eliminating inflow and infiltration which further decreases total wastewater flows.

Overall, the per capita loading for the Town falls within the expected ranges for domestic wastewater. This suggests that the estimated population numbers from the State and historic flow, BOD, TSS, and ammonia data are reasonably accurate. Influent BOD loading averaged 0.15 ppcd over the last seven years while influent TSS loading averaged 0.19 ppcd. Influent ammonia loading averaged 0.018 ppcd.

PEAKING FACTORS

Wastewater flows vary daily and seasonally which affect hydraulic as well as biological processes in wastewater treatment. To accommodate the changes in flow, peaking factors are applied. Peaking factors provide a factor of safety that facilitates hydraulic and biological treatment demands in times of extreme flow or loading conditions. Calculated peaking factors for the Erie SWRF are presented in Table 4-21.

Historical peaking factors for influent flow, BOD_5, TSS, and ammonia were determined by dividing the monthly average by the corresponding annual average for each year from 2003 through 2010. The maximum day and maximum month peaking factors were selected based on a statistical analysis of the 95th and 99th percentile frequency of the monthly peaking factors over the four-year historical record. Maximum day peaking factors were calculated by dividing the highest flow for a single day in each year by the annual average flow. Peak day peaking factors ranged between 1.18 and 1.38 between 2004 and 2010. Peak hour peaking factors were determined by using the DRCOG peaking factor equation as previously discussed. Peak hour peaking factors are presented in Table 4-11.

Table 4-21 Historical Influent Peaking Factors for the Town of Erie SWRF, 2004 to 2009

	Peaking Factors			
	Maximum Day		Maximum Month	
Parameter	95th Percentile	99th Percentile	95th Percentile	99th Percentile
Flow	1.34	1.38	1.14	1.14
BOD_5			1.28	1.30
TSS			1.20	1.20
Ammonia			1.25	1.27

Note: Maximum day peaking factors for flow do not include data from 2003.
Note: Flow reported by client for May 10, 2003 was 1.61 mgd. This flow is three times the previous daily flow and is likely a typographical error. Data point was deleted from data set.

WASTEWATER FLOW AND LOAD PROJECTIONS

For planning purposes, future flows and loads were estimated using both historic generation rates and higher per capita flow generation rates that agree with planning criteria recommended by DRCOG and the State of Colorado. For planning purposes, it is highly recommended that the Town reevaluate its per capita generation rates once every five years. This will enable the Town to determine whether the lower or higher end of predicted flows and loads presented in this section are more appropriate for continued planning. If current trends continue, flows and loads will be on the lower end. As industrial and commercial users move into the service area, per capita generation rates may increase and push flows and loads toward the higher end.

Flow and Load Projections from Historic Generation Rates. More than eight years of consistent, historic data are available for the SWRF. For each year, annual average per capita flow generation rates have been below 70 gpcd and maximum month generation rates have been below 79 gpcd. The length of the data record as well as its consistency from year to year provides justification for using calculated historic generation rates. At the same time, using a low per capita generation rate for planning purposes will not account for flows generated by future commercial and industrial developments that the Town hopes to attract to its service area. Using a higher per capita generation rate may better reflect future conditions and should result in appropriately sized infrastructure.

Flow projections for the Town, shown in Table 4-22, were estimated by multiplying the population projections from Table 4-5 by the highest historic annual average per capita flow contribution of 70 gallons of wastewater per capita day. Per capita generation rates are presented in Table 4-20. Maximum Month flows were calculated by multiplying the projected average annual flows by the 99th percentile maximum month peaking factor of 1.14 given in Table 4-21. This results in a maximum month per capita generation rate of 80 gpcd. Peak day flows were estimated by multiplying the projected annual average flows by the peak day peaking factor of 1.38 given in Table 4-21. Peak hour flow rates were determined by multiplying the projected annual average flows by a peaking factor calculated using the DRCOG formula. Peak hour peaking factors decrease with increasing flows and differ between the SWRF and NWRF. Peak hour peaking factors are presented in Table 4-23.

Table 4-22 Estimated Future Wastewater Flows from Historic Generation Rates

Year	Population	Influent Flow, MGD		
		AAF	MM	Peak Day
2010	18,135	1.27	1.45	1.74
2015	26,525	1.86	2.12	2.57
2020	33,525	2.35	2.68	3.24
2025	40,680	2.85	3.25	3.93
2030	49,625	3.47	3.96	4.79
Buildout	68,820	4.82	5.49	6.65

AAF = Annual Average Flow, MM = Maximum Month Average, PF = Peaking Factor
Estimated future flows are based on a per capita generation rate of 70 gpcd.

Table 4-23 Estimated Peak Hour Peaking Factors and Peak Hour Flows by Facility

SWRF			NWRF		
MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD	MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD
1.0	3.73	3.73	1.5	3.49	5.23
1.2	3.62	4.34	3.6	3.01	10.84
1.6	3.45	5.52	3.9	2.90	11.34

Note: MMF is the average daily flow during the maximum month. Peak hour peaking factors are calculated using the annual average flow. The maximum month peaking factor is 1.14

Future annual average loads for BOD, TSS, and ammonia were estimated by multiplying the projected population for each year by the per capita generation rates presented in Table 4-20. The per capita generation rates used for BOD, TSS, and ammonia are 0.16 ppd, 0.19 ppd, and 0.02 ppd, respectively. These generation rates are the highest annual average generation rates observed over the last eight years. Future maximum month loads were estimated by multiplying the calculated annual average loads by the maximum month peaking factors presented in Table 4-21.

Table 4-24 Future Loads for Town of Erie from Historic Generation Rates

		BOD, ppcd		TSS, ppcd		NH_3-N, ppcd	
Year	Population[1]	AA	MM	AA	MM	AA	MM
2010	18,135	2,902	3,773	3,446	4,135	326	414
2015	26,525	4,244	5,517	5,040	6,048	477	606
2020	33,525	5,364	6,973	6,370	7,644	603	766
2025	40,680	6,509	8,462	7,729	9,275	732	930
2030	49,625	7,940	10,322	9,429	11,315	893	1,134
Buildout	68,820	11,011	14,314	13,076	15,691	1,239	1,574

AAF = Annual Average Flow, MM = Maximum Month Average, AA = Annual Average
AA per capita generation rates utilized are 0.16 ppd BOD, 0.19 ppd TSS, and 0.018 ppd NH_3N
Maximum Month Peaking Factors Applied are BOD = 1.30, TSS = 1.20, Ammonia = 1.27

Flow and Load Projections using Recommended State Design Criteria. A previous planning study utilized an annual average per capita wastewater generation rate of 90 gpcd and a maximum month wastewater generation rate of 118 gpcd (Burns and McDonnell June 2008). The higher flow rates were generated by assuming additional flows for commercial activities and I&I which were then added to the historic generation rate. This assumption was made to satisfy design criteria set forth in State Policy 96-1 and to ensure that Erie will have adequate capacity to treat future flows and loads as more commercial properties move into the service area. The resulting flow rate of 90 gpcd is conservative and provides a margin of safety for future planning. The highest annual average per capita generation rate measured over the last eight years was 68 gpcd. Using a generation rate of 90 gpcd to estimate future flows may overestimate future flows unless the character of the service area changes to include more commercial and industrial activities.

Per capita wastewater generation rates are lower in Erie than in similarly sized municipalities throughout Colorado. Reasons for lower water use include water saving fixtures in newer construction, a predominantly plastic pipe collection system, and the makeup of the service area. The Town of Erie is predominantly a bedroom community with no major industries and few commercial users. A large

commercial enterprise such as a shopping mall would attract shoppers from outside the service area. These external users would contribute to wastewater flows and loads thereby driving up the calculated per capita generation rates. Industrial users, especially manufacturers, tend to be use large amounts of water compared to residential users.

Table 4-25 compares the projected flows and loads of this WUP update to the previous planning study. Differences between future flows and loads projected by this update versus earlier planning efforts are due primarily to the inclusion of more recent influent data. The 2008 Update included influent data collected between 2001 and 2005 whereas this update included influent data from 2003 through 2009. The newer data resulted in slightly different per capita generation rates for flow, BOD, TSS, and ammonia. Peaking factors for peak day flows decreased significantly from 1.82 to 1.38. This is partly due to exclusion of a single bad flow measurement from the historic data set, but also reflects attenuation of flows by increasing population.

Table 4-25 Comparison of Recommended Year 2025 Flow and Load Projections

Parameter	Average Day	Max Month	Peak Day	Source
Flow, mgd	3.7	4.8	6.3	Burns and McDonnell
	2.85	3.25	3.93	Indigo / Farnsworth
BOD_5, ppd	7,811	10,170	15,947	Burns and McDonnell
	6,509	8,462	NA	Indigo / Farnsworth
TSS, ppd	6,061	8,299	13,831	Burns and McDonnell
	7,729	9,275	NA	Indigo / Farnsworth
Ammonia, ppd	570	732	1,220	Burns and McDonnell
	732	930	NA	Indigo / Farnsworth

*** Population for 2025 is projected to be 40,680 persons.

In the 2008 Update, Burns and McDonnell utilized a per capita BOD generation rate of 0.192 ppcd even though the generation rates observed to date ranged between 0.123 and 0.142 ppcd. A higher generation rate was selected for planning purposes and design of the NWRF because BOD loadings had been trending upward at new development areas and to ensure adequate treatment capacity to accommodate future commercial and industrial developments (Burns and McDonnell June 2008). The 0.192 ppcd of BOD value is consistent with published data for other treatment facilities and allows for a conservative design. Data collected at the SWRF between 2005 and 2010 do not reflect an upward trend in per capita generation rates. The highest average per capita BOD generation rate observed during this period was 0.16 ppcd. Planning for higher loads will help the Town to accommodate future commercial and industrial development.

Flow and Load Projections Selected for Planning Purposes. For planning purposes, slightly higher per capita generation rates were used to estimate future flows and loads than might be suggested from the historic data set. Currently, the Town is a bedroom community with few commercial accounts and no major industrial users. This may change in the future. More commercial and industrial users will change the characteristics of the influent wastewater and will result in higher observed per capita generation rates. For these reasons, the flow projections presented in Table 4-26 were selected to ensure adequate future capacity in both the collection system and the treatment facilities. Future loads are given in Table 4-24.

Table 4-26 Selected Future Flows for Planning Purposes

Parameter	2025	Buildout
Annual Average Flow Generation Rate, gpcd	90	90
Maximum Month Flow Generation Rate, gpcd	118	118
Annual Average Daily Flow, mgd	3.70	6.26
Maximum Month Flow, mgd (MM PF = 1.3)	4.80	8.12
Peak Day Flow, mgd (PD PF = 1.7)	6.30	10.66
Maximum Month Flow to SWRF, mgd	1.60	1.60
Annual Average Flow to SWRF, mgd	1.23	1.23
Peak Hour Peaking Factor for SWRF	3.52	3.52
Peak Hour Flow to SWRF, mgd	4.35	4.35
Maximum Month Flow to NWRF, mgd	3.20	6.52
Annual Average Flow to NWRF, mgd	2.46	5.02
Peak Hour Peaking Factor for NWRF	3.14	2.79
Peak Hour Flow to NWRF, mgd	7.74	14.02

MM PF = Maximum Month Peaking Factor, PD PF = Peak Day Peaking Factor

Table 4-27 lists projected future average and maximum month flows based on population projections. Average daily flows use a per capita generation rate of 90 gallons per person per day. Maximum month flows use a per capita generation rate of 118 gallons per person per day.

Table 4-27 Projected Future Average and Maximum Month Flows

Date	Population	ADF	MM
2010	18,135	1.63	2.14
2015	26,525	2.39	3.13
2020	33,525	3.02	3.96
2025	40,680	3.66	4.80
2030	49,625	4.47	5.86
Buildout	68,820	6.19	8.12

Flow Apportionment Between SWRF and NWRF. Pursuant to Colorado Law, the Town is required to initiate engineering and financial planning for expansion of the wastewater treatment facility(s) when throughput and treatment reaches eighty-percent of the 30-day average permitted capacity. When ninety-five percent of the 30-day average permitted capacity is met, the Town is required to begin construction of the necessary expansion. The eighty-percent and ninety-five percent triggers apply to maximum month conditions.

The SWRF is currently operating at 63.9 percent of its permitted hydraulic capacity and 68.7 percent of its permitted organic capacity. The SWRF was rerated in 2006 for 1.6 MGD and 3,870 ppd of BOD. The rerated capacity was approved by the State in October 2006. The new discharge permit was issued in September 2011 and became effective October 1, 2011.

The NWRF has a permitted capacity of 1.5 mgd and 3,233 ppd of BOD. It can be expanded to 3.6 MGD and 7,750 ppd of BOD with relatively minor changes including increasing the Kaldnes media fill in the aeration basins from 25% to 60%, construction of a third secondary clarifier, and installing multiple pumps and a second influent screen. Presently, the SWRF and NWRF have a combined maximum month treatment capacity of 3.1 MGD. The next expansion of the NWRF may be needed before the year 2015.

Erie is able to divert flows to the NWRF from the SWRF via the new north interceptor. The diversion structures allow the Town to control the amount of flow and load reaching each reclamation facility. Previous planning efforts recommended treating 25% of the buildout flow at the SWRF with the remaining 75% being treated at the NWRF (Burns and McDonnell June 2008).

INDUSTRIAL PRETREATMENT PROGRAM

Wastewater treatment plants that are larger than 5 mgd or that have categorical dischargers within their service areas are required to develop and maintain an industrial pretreatment program. Erie's service area contains residential and light commercial users. The City is not required to have a formal pretreatment program. The City does enforce a local limit and grease trap ordinance at EPA's request.

INFLOW AND INFILTRATION ANALYSIS

Inflow is water that enters the collection system through direct connections such as cross-connected storm drains and pipes, roof leaders plumbed to the sanitary sewer, open cleanouts, and holes in manhole lids. Infiltration is water that enters the collection system through cracks in pipes, poorly fitting pipe joints, offset joints, manholes in poor condition, and other sources. With infiltration, the connection is not direct. Together, inflow and infiltration (I&I) can contribute a significant amount of flow to the collection system. I&I should be eliminated to the greatest extent possible because it uses up valuable capacity both in the collection system and at the wastewater treatment plant.

When the groundwater table rises above infiltration points in the collection system – offset joints, cracked pipes, and poorly maintained manholes – groundwater is able to flow into the collection system. It carries sand, silt, and grit with it. When the groundwater table is low, wastewater is able to exfiltrate or leak out of the collection system into the ground. Eliminating I&I also reduces exfiltration.

Two good indicators of I&I are high per capita wastewater generation rates and seasonal variations in wastewater flows that correspond with wet weather events. The State of Colorado asks systems with wastewater generation rates higher than 120 gpcd to investigate their collection systems and implement repairs to decrease groundwater infiltration. Erie's per capita wastewater generation rate is only 70 gpcd which is on the lower end of typical generation rates of 40 to 120 gpcd (Metcalf and Eddy 2003). Flows have been increasing with increasing population, but flows do not follow a strong seasonal pattern or vary significantly in response to storm events. Taken together, low per capita generation rates and consistent flows year round suggest that I&I, if present at all, is exceedingly low.

For most municipalities and water/sanitation districts, it is possible to estimate I&I by comparing winter potable water use to winter wastewater generated. Since residents do not water their lawns during the winter months, almost all potable water eventually finds its way into the sewer. Table 4-28 compares winter water used/billed by the Town to the wastewater flows measured at the SWRF. Water and wastewater flows generally agree. Differences may be attributed to a number of things including measurement uncertainties in residential water meters, billing records rounding up or down to the nearest 1,000 gallons, measurement uncertainties in the SWRF influent flow meter, filling and flushing water tanks and lines, and cleaning of collection system lines. All flow measurement devices are accurate

within specified tolerances; however, some variability exists between measurements even within those tolerances.

Table 4-28 Compare Winter Water Use Billed to Wastewater Flows Received at SWRF

Water Billed (measured at customer meters)							
	2003	2004	2005	2006	2007	2008	2009
January	0.533	0.542	0.643	0.808	0.789	1.508	1.023
February	0.519	0.582	0.686	0.799	No Data	1.180	0.949
March	0.448	0.604	0.643	0.762	0.770	0.848	0.788
November	0.542	0.731	1.114	0.947	1.032	0.909	No Data
December	0.566	0.598	1.050	0.839	0.562	0.848	No Data
Wastewater Received (measured at SWRF)							
January	0.477	0.541	0.769	0.777	0.938	0.909	0.912
February	0.489	0.545	0.776	0.775	0.982	0.910	0.920
March	0.561	0.630	0.780	0.771	0.972	0.892	0.927
November	0.614	0.758	0.769	0.881	0.875	0.911	No Data
December	0.555	0.766	0.800	0.873	0.913	0.930	No Data
Percent Difference in Wastewater Received versus Water Billed							
January	89.5%	99.9%	119.5%	96.2%	118.9%	60.3%	89.2%
February	94.1%	93.6%	113.0%	97.0%	No Data	77.1%	97.0%
March	125.3%	104.3%	121.4%	101.2%	126.2%	105.2%	117.6%
November	113.4%	103.7%	69.0%	93.0%	84.8%	100.2%	No Data
December	98.1%	128.0%	76.2%	104.1%	162.5%	109.6%	No Data

Section 6 of this utility plan discusses hydraulic modeling of the collection system.

SECTION 5
EVALUATION OF WASTEWATER TREATMENT FACILITIES

The Town of Erie operates two wastewater treatment facilities. The South Water Reclamation Facility (SWRF) is located near the center of the service area just north of the intersection of Briggs Street and Evans Street. Briggs Street becomes County Road 1 ½ as it passes the SWRF. The North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The NWRF is located north of Highway 52 and east of County Line Road along Boulder Creek.

NWRF PROCESS SYSTEM

The NWRF process diagram and hydraulic profile are shown in Figures 5-1 and 5-2, respectively. The NWRF has a rated capacity of 1.5 mgd and 3,233 ppd of BOD. A new interceptor along with bypass structures allows the Town to distribute flow between the SWRF and NWRF. A previous planning study recommended that the SWRF treat a maximum month flow of 1.2 mgd with all other flows diverted to the NWRF (Burns and McDonnell, 2008).

The NWRF headworks consists of a mechanical screen, manual bar screen in a bypass channel, lift station, and grit basin. The secondary process is a three-stage biological nutrient removal integrated fixed film activated sludge (IFAS) process with anaerobic zones, anoxic zones, and aerated zones followed by secondary clarification. The secondary process is designed for biological phosphorus removal, nitrification, and denitrification. Secondary clarifier effluent flows to the disinfection room where a splitter structure diverts flow to either a tertiary cloth media filter or directly to the UV disinfection units. The tertiary filter may treat up to 3.6 MGD of flow to produce reuse quality water which will be stored in an on-site reservoir. Residual solids from the activated sludge process will be stabilized with lime prior to dewatering with a screw press. The NWRF will produce Class A biosolids using the time/temperature criteria.

NWRF INFRASTRUCTURE SIZING AND STAGING

The NWRF plans and specifications were approved by the State of Colorado Water Quality Control Division on December 6, 2009 (Camrud 2009). The approved hydraulic and organic loading rates are 1.50 MGD and 3,223 ppd of BOD. Design criteria are presented in Table 5-1. The NWRF will ultimately be expanded to treat 3.6 MGD.

Table 5-1 Design Criteria for NWRF

Parameter	MGD	Parameter	Lb/day	mg/L
Average Daily Flow	1.2	BOD	3,228	258
Maximum Month Flow	1.5	TSS	2,640	211
Peak Day Flow	2.1	TKN	313	25
Peak Hour Flow	4.2			

Note: Loading are based on the 30-day average value for the maximum month.

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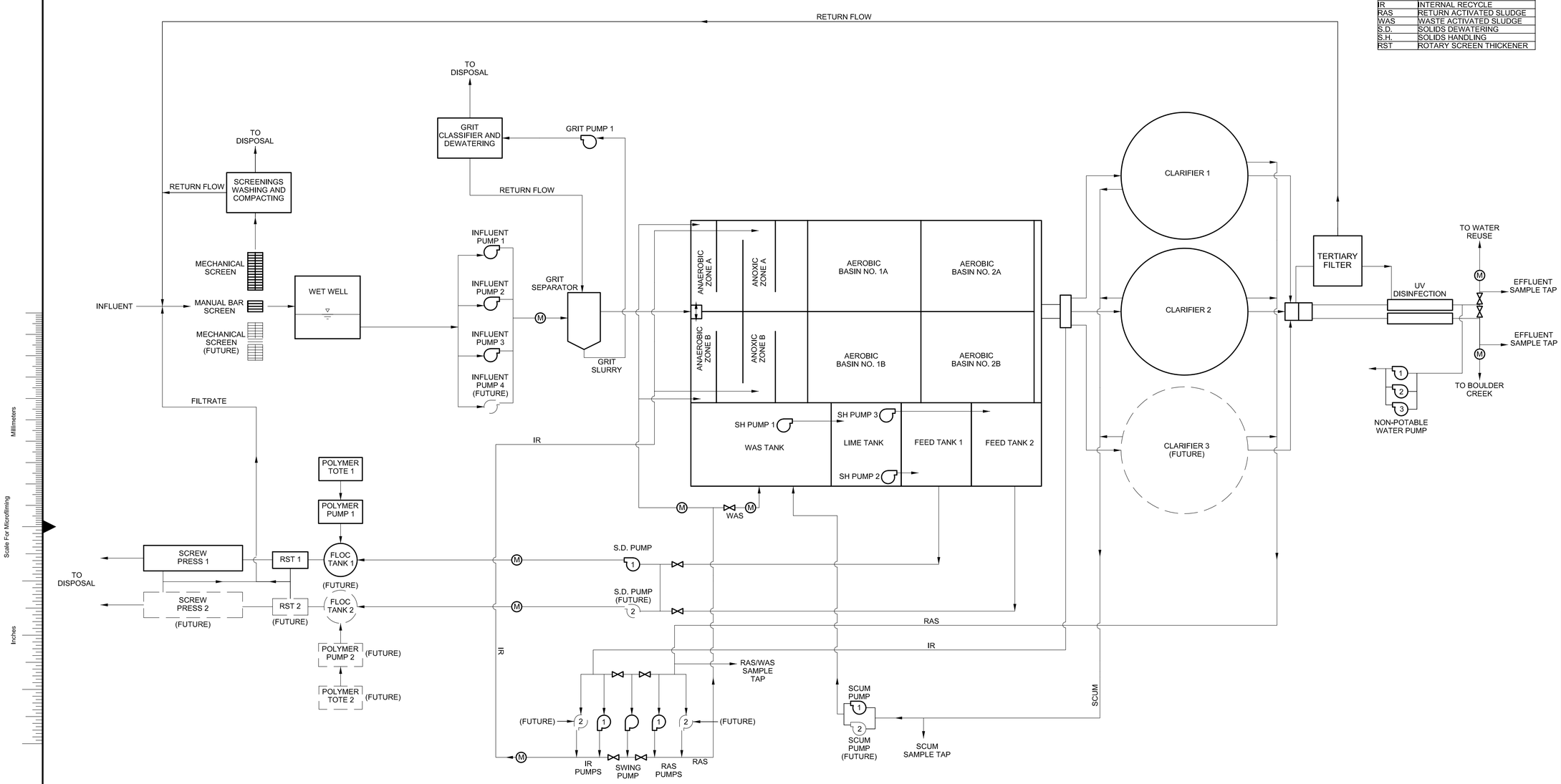
LEGEND	
IR	INTERNAL RECYCLE
RAS	RETURN ACTIVATED SLUDGE
WAS	WASTE ACTIVATED SLUDGE
S.D.	SOLIDS DEWATERING
S.H.	SOLIDS HANDLING
RST	ROTARY SCREEN THICKENER



PLANT DESIGN LOADING PARAMETERS		
HYDRAULIC FLOW	MGD	
MAXIMUM MONTH	1.5	
PEAK DAY	2.1	
PEAK HOUR	4.2	
BOD	LB/DAY	MG/L
MAXIMUM MONTH	3228	258
TSS	LB/DAY	MG/L
MAXIMUM MONTH	2640	211
TKN	LB/DAY	MG/L
MAXIMUM MONTH	313	25

IFAS TREATMENT TRAIN DESIGN PARAMETERS	
CELL 1: ANAEROBIC	
HYDRAULIC PARAMETERS	MGD
MAXIMUM MONTH FLOW	1.5
PEAK DAY FLOW	2.1
PEAK HOUR FLOW	4.2
RAS FLOW (1.5*MAX MONTH)	2.25
PHYSICAL PARAMETERS	
NUMBER OF PASSES	2
NUMBER OF BASINS	2
BASIN VOLUME (EACH IN GAL)	76,826
TOTAL VOLUME (GAL)	153,652
WATER DEPTH (FT)	18.50

IFAS TREATMENT TRAIN DESIGN PARAMETERS	
CELL 1: ANOXIC	
HYDRAULIC PARAMETERS	MGD
MAXIMUM MONTH FLOW	1.5
PEAK DAY FLOW	2.1
PEAK HOUR FLOW	4.2
RAS FLOW (1.5*MAX MONTH)	2.25
IR FLOW (1.75*MAX MONTH)	2.63
PHYSICAL PARAMETERS	
NUMBER OF PASSES	2
NUMBER OF BASINS	2
BASIN VOLUME (EACH IN GAL)	110,962
TOTAL VOLUME (GAL)	221,924
WATER DEPTH (FT)	18.50

IFAS TREATMENT TRAIN DESIGN PARAMETERS (CONT.)	
CELL 2: AEROBIC	
HYDRAULIC PARAMETERS	MGD
MAXIMUM MONTH	1.5
PEAK DAY	2.1
PEAK HOUR	4.2
RAS FLOW (1.5*MAX MONTH)	2.25
IR FLOW (1.75*MAX MONTH)	2.63
PHYSICAL PARAMETERS	
NUMBER OF BASINS	2
VOLUME (EACH IN GAL)	174,491
TOTAL VOLUME (GAL)	348,982
WATER DEPTH (FT)	17.50

IFAS TREATMENT TRAIN DESIGN PARAMETERS (CONT.)	
CELL 3: AEROBIC	
HYDRAULIC PARAMETERS	MGD
MAXIMUM MONTH	1.5
PEAK DAY	2.1
PEAK HOUR	4.2
RAS FLOW (1.5*MAX MONTH)	2.25
IR FLOW (1.75*MAX MONTH)	2.63
PHYSICAL PARAMETERS	
NUMBER OF BASINS	2
VOLUME (EACH IN GAL)	174,476
TOTAL VOLUME (GAL)	348,952
WATER DEPTH (FT)	17.50

CLARIFIER DESIGN PARAMETERS		
HYDRAULIC PARAMETERS	MGD	LB/DAY
MAXIMUM MONTH	1.5	
PEAK DAY	2.1	
PEAK HOUR	4.2	
RAS FLOW (1.5*MAX MONTH)	2.25	
WAS		3,632
PHYSICAL PARAMETERS		
NUMBER OF BASINS	2	
VOLUME (EACH IN GAL)	420,761	
TOTAL VOLUME (GAL)	841,522	
WATER DEPTH (FT)	13.70	

Burns & McDonnell SINCE 1898

date SEPTEMBER, 2009
detailed R. FULK
designed A. WADDOUPS
checked S. YONKER

TOWN OF ERIE 1874

TOWN OF ERIE
NORTH WATER RECLAMATION FACILITY
PROCESS FLOW SCHEMATIC AND
PROCESS DESIGN CRITERIA

project 39119
contract

drawing D002 – rev. 0

Figure 5-1 North Water Reclamation Facility Process Diagram

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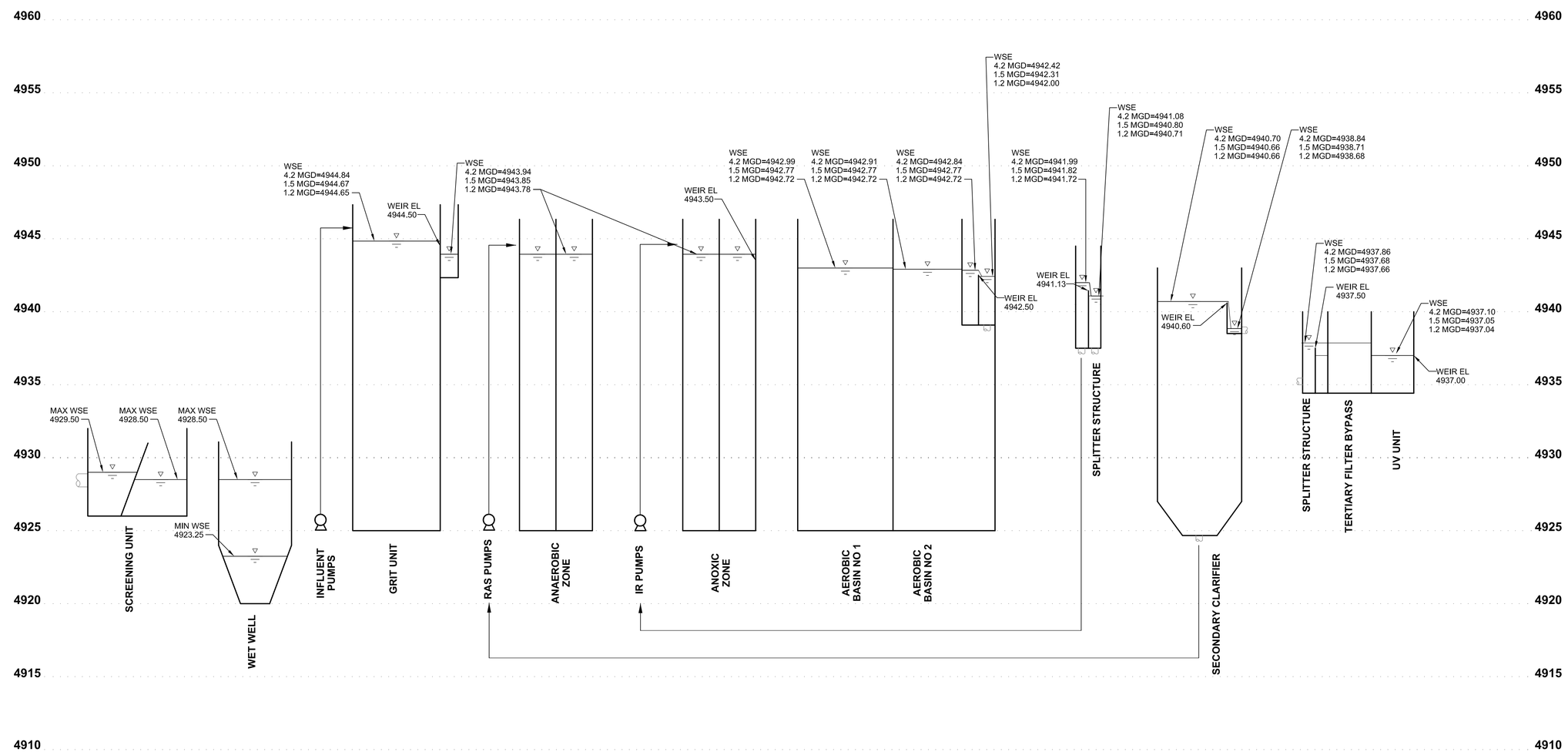
Scale For Microfilming: Millimeters / Inches



NOTES:

1. WSE ELEVATIONS DISPLAYED REPRESENT PEAK HOURLY FLOW.
2. BOTH IFAS TRAINS ARE IN SERVICE.
3. TWO SECONDARY CLARIFIERS ARE IN SERVICE.
4. ONE UV UNIT IN SERVICE.

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date	detailed
SEPTEMBER, 2009	R. FULK
designed	checked
A. WADDOUPS	S. YONKER

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NORTH WATER RECLAMATION FACILITY

HYDRAULIC PROFILE

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Figure 5-2 North Water Reclamation Facility Hydraulic Profile

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NWRF Headworks

Headworks for the NWRF consists of screening, influent pumping, and grit removal. Screening and grit removal processes remove coarse solids and inorganic particles that might damage equipment or interfere with operation of subsequent treatment units. Rags and fibrous materials can accumulate on pump impellers and shafts, causing vibration that may result in premature failure of pump bearings and seals. Placing the screening equipment upstream of the influent pumps will help to protect the pumps from clogging and damage. Grit and sand in the wastewater also increase wear on pump impellers, shafts, casings, and seals. Solid debris may also accumulate in process tanks and digesters.

Headworks equipment is typically sized to accommodate peak hour flows with one unit out of service. It is critical that the headworks be able to handle the hydraulics to keep the influent from overflowing the headworks structure. It is not critical that the headworks achieve the same level of treatment for brief periods of time when flows are at their peaks since performance does not directly impact discharge permit compliance.

Design criteria for the NWRF headworks are summarized in Tables 5-2 and 5-3. Wastewater enters the facility through a thirty-six inch gravity sewer line. Under normal operating conditions, the wastewater enters an influent channel that contains a 6 mm screen with a washer and compactor. Screenings are collected in a dumpster and are ultimately landfilled. Rinse water from the washer/compactor is returned to the influent channel. Flow may be diverted into a bypass channel that contains a manual bar screen with 1 to 1 ¾-inch openings. The bypass channel allows operations staff to take the mechanical screen out of service for routine maintenance and repairs. The manual bar screen is not intended for continuous operation. A third channel has been provided that will ultimately house a second mechanical screen.

Table 5-2 NWRF Headworks Design Criteria

Process	Design Criteria	Flow Rate	Capacity	Notes
Influent Channel (Primary)		1 fps	5.8 mgd @ 36” Deep	3’ wide at narrowest point
(Primary)		2.5 fps	14.5 mgd @ 36” Deep	3’ wide at narrowest point
(Manual Backup)		1 fps	4.8 mgd @ 36” Deep	2.5’ wide at narrowest point
(Mechanical)		2.5 fps	12.1 mgd @ 36” Deep	2.5’ wide at narrowest point
Mechanical Screen	2.5 fps max	2.5 fps 2.5 fps 2.0 fps	4.5” deep @ 1.5 mgd 14” deep @ 4.6 mgd 5.6 mgd	Max Month Flow Peak Hour Flow Max Capacity per manufacturer
Influent Pumps	Minimum 2 pumps w/ capacity greater than influent flow	1600 gpm each of 3	2.3 mgd each, 2 duty, one backup	Will operate as lead/lag if necessary. Non-clog, sewage handling.
Grit Removal	Vortex system – 6 foot diameter	3500 gpm	5.1 mgd	Discharges to washer and decanter.

Table 5-3 Design Criteria for NWRF Influent and Grit Pumps

Pump Description	Design Point	HP	Control	Number
Influent Pumping Flooded Suction Centrifugal	900 gpm @ 25' head 1600 gpm @ 26' head	15 hp	VFD, Level	3
Grit Pump	200 gpm @ 25' head	5 hp	On/Off	1

After screening, the wastewater flows to an influent wet well which provides a flooded suction for the influent pumps. There are three, equally sized influent pumps and space for a fourth influent pump to be added in the future. The wet well is divided into two compartments with a slide gate between the compartments so that half of the wet well may be isolated for cleaning or maintenance. Each wet well is equipped with an ultrasonic level sensor/transmitter and high and low level float switches.

The influent pumps have a capacity of 900 to 1600 gpm each and are equipped with variable frequency drives. Influent flows are measured by an eighteen-inch magnetic flow meter in the influent pump station discharge line. The raw wastewater is lifted approximately 21 feet to the grit basin. The grit system uses gravity to settle out heavy particles. A grit pump constantly removes the settled grit and conveys it to an elevated vortex type grit washer and classifier. Excess water is returned to the grit chamber.

NWRF Secondary Treatment

The secondary treatment process is a three-stage, biological nutrient removal integrated fixed film activated sludge system (IFAS). It consists of two treatment trains and three secondary clarifiers. Design criteria for the secondary process are presented in Table 5-4. Each train contains an anaerobic zone, anoxic zone, and two aerobic zones. The anaerobic and anoxic zones are baffled to minimize short-circuiting. Biological phosphorus removal begins in the anaerobic zone and is completed in the aerobic zone. Nitrate is removed in the anoxic zone and ammonia is converted to nitrate in the aerobic zones. Internal recycle pumps return nitrate rich wastewater from the secondary clarifier splitter structure to the beginning of the anoxic zone for nitrate removal. Organic matter and total suspended solids are removed in all three zones. Return activated sludge (RAS) pumps bring settled sludge back from the secondary clarifiers to the beginning of the anoxic zones.

The aerobic zones contain floating plastic media to increase the amount of biomass retained in the aeration basins of an activated sludge process. The free-floating media, shown in Figure 5-3, is added directly to the aerobic zones of the aeration basins. The NWRF will have an initial media fill volume in all aerobic zones of about 25%. Additional biomass growing on the integrated fixed film activated sludge (IFAS) media increases the capacity per unit volume of aeration basin by increasing the solids inventory without increasing the MLSS concentration. IFAS systems increase BOD removal, nitrification, and denitrification capacity in activated sludge processes without increasing the volume of basins. There are multiple IFAS systems in the Rocky Mountain Region including the City of Broomfield WWTF and the Crow Creek and Dry Creek WWTPs in Cheyenne, Wyoming.

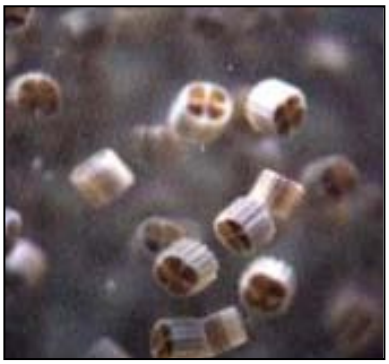

Figure 5-3 Kaldnes Media

The mixed liquor suspended solids (MLSS) concentration in a conventional activated sludge system is limited by the ability to settle the solids in the secondary clarifiers. For most systems, the MLSS concentrations are limited to 3,500 mg/L or less. Submerged fixed film media allows a larger inventory of solids (higher effective MLSS concentration) to be maintained in the aeration basins. The fixed film solids that grow on the media remain in the activated sludge basin. Since the attached solids remain in the aeration basin, they don't affect settling performance in the secondary clarifiers. The fixed film solids have a long effective sludge age (>30 days) that sustains nitrification even when the suspended biomass (MLSS) has a much shorter sludge age. Submerged fixed film media systems generally produce a sludge that settles, flocculates, and compacts well. The NWRF IFAS is designed to maintain an MLSS concentration of 2,000 mg/L at an aerobic sludge age of 3.13 days at 10 degrees Celsius.

Table 5-4 Design Criteria for NWRF Activated Sludge Basins

Zone	Anaerobic	Anoxic	Aerobic 1	Aerobic 2	Clarifiers
Number of Passes	2	2	1	1	N/A
Number of Basins	2	2	2	2	2
Basin Volume, each, gallons	78,826	110,962	174,491	174,476	420,761
Total Volume, gallons	153,652	221,294	348,982	348,982	841,522
Freeboard, inches	18	18	18	18	18
Length, feet	17	21	34	34	
Width, feet	32	39	39	39	
Water Depth, feet	18.5	18.5	17.5	17.5	13.7
Diameter, feet					70
HRT at MM Flow, hours	2.45	3.54	5.58	5.58	13.4
RAS Flow, MGD (1.5 * Max Month)			2.25	2.25	
Internal Return Flow, MGD (1.75 * Max Month)			2.63	2.63	

Flow is split evenly between each anaerobic zone by a weir structure that receives flow from the grit basin. Gates may be used to isolate either treatment train and take it out of service. An open channel between the two trains of the anaerobic cells ensures an even water surface between the two cells. The wastewater in the anaerobic cell flows through a large window between the anaerobic and anoxic cells. Both the anaerobic and anoxic zones contain submersible mixers.

The internal recycle pumps discharge MLSS into the anoxic cells immediately downstream of the anaerobic/anoxic divider wall. A weir wall at the end of the anoxic cells maintains the water surface in the anaerobic and anoxic cells. The weir provides an elevation drop between the anoxic and aeration basins to ensure that process media doesn't migrate back into the anoxic basin. At the NWRF, each of the aerobic zones contains cylindrical immersed screens consisting of wedge wire retain the IFAS media in the

Figure 5-4 IFAS Media Retention Screens

system. The immersed media retention screens are shown in Figure 5-4.

The treated wastewater from each of the treatment trains is conveyed by an effluent lauder to a discharge box at the end of the aeration basins. From here, a twenty-four inch diameter pipe transfers the mixture of treated wastewater and biological solids to a splitter structure upstream of the secondary clarifiers. The splitter structure also serves as the internal recycle pump wet well. There are two, seventy-foot diameter, center-feed secondary clarifiers. Design criteria for the secondary clarifiers are presented in Table 5-5.

Table 5-5 Design Criteria for NWRF Secondary Clarifiers

Parameter	Value
Number	2
Diameter, feet	70
Depth, feet	13.7
Surface Area, square feet, each	3,846
Volume, gallons, each	420,761
Hydraulic Retention Time at MM Flow, hours	13.5
Surface Overflow Rate at MM Flow, gpd/sf	195
Surface Overflow Rate at Peak Hour Flow, gpd/sf	546

Note: Maximum month flow is 1.5 MGD. Peak hour flow is 4.2 MGD. Calculations assume both clarifiers are on-line.

The internal recycle (IR) and return activated sludge (RAS) pumps operate as flooded suction centrifugal pumps. Pump design criteria are given in Table 5-6. The IR and RAS pumps are identical and arranged in order to maintain redundancy between the IR and RAS systems. A single swing pump is located on the common header between the IR and RAS pumps. With the use of VFD control and isolation valves, the swing pump can function as either an IR or RAS pump. The RAS pumps have a capacity for up to 1.5 times the maximum month flow rate while the IR pumps have a capacity of 1.75 times the maximum month flow rate. Both IR and RAS pumps are equipped with VFDs.

Table 5-6 Design Criteria for NWRF RAS and IR Pumps and Mixers

Pump Description	Design Point	HP	Control	Number
Internal Recycle Pumps Flooded Suction Centrifugal	1825 gpm @ 10' head 900 gpm @ 8' head	15 hp	VFD, Level	1 + Shared Pump
RAS Pumps Flooded Suction Centrifugal	1600 gpm @ 10' head 800 gpm @ 8' head	15 hp	VFD, Level	1+ Shared Pump
Anaerobic Cell Mixers, Submersible	400 N	3 hp	On/Off	4 Total
Anoxic Cell Mixers, Submersible	480 N	4 hp	On/Off	4 Total

The secondary clarifier effluent is combined in a junction structure and flows by gravity to the disinfection room. Reuse quality water may be produced by diverting up to 3.6 MGD of secondary clarifier effluent through a cloth media filter prior to disinfection. Reuse water will be stored in a reservoir on-site. The filter is a drop in item and will be installed by the end of 2012 when construction of the reuse reservoir is complete. The cloth media filter is designed to meet Category 2 reuse standards of less than 3 NTU's on a monthly basis. The disk filter has an automatic backwash cycle that cleans the cloth media based on head loss through the filter. Backwash water is returned to the plant headworks

upstream of influent monitoring. If reuse quality water is not needed, secondary clarifier effluent bypasses the filter and goes directly to the UV disinfection system.

NWRF Disinfection

The NWRF is equipped with two, parallel ultra-violet disinfection units. Each unit is housed in a dedicated channel. Each channel has the capacity to process a peak hour flow of 8.7 mgd which provides complete redundancy. The creek discharge and reuse discharge will have the same effluent standard for E. coli.

Effluent flow is monitored by a 24-inch magnetic flow meter on the discharge pipe from the clarifiers to the UV disinfection system. The plant computer control system will monitor and totalize the flow.

NWRF Solids Handling

The solids processing basins consist of a Waste Activated Sludge (WAS) tank, mix tank, and two feed tanks. Design criteria for the solids handling basins are included in Table 5-7. Excess solids from the activated sludge process and scum from the secondary clarifiers are pumped to the waste activated sludge (WAS) holding tank. WAS is drawn from the RAS line. The WAS tank is designed to provide approximately three days of solids storage when the facility influent flow rate is 1.75 MGD and the WAS concentration is 0.8% (8,000 mg/L). The WAS tank is aerated and contains mixers.

Table 5-7 Design Criteria for NWRF Solids Handling Tanks

	WAS Tank	Lime Mix Tank	Feed Tanks (2)	Lime Silo
Diameter, feet				12
Length, feet	40	22	22	
Width, feet	32	32	32	
Depth, feet	18	18	18	40
Volume, cf	23,040	12,672	12,672	4,522
Volume, gallons	172,340	94,787	94,787	33,822

The sludge is transferred from the WAS tank to the Lime/Sludge tank utilizing a batch process. Enough lime is added to the sludge to maintain a pH of 12 for 2 hours. The sludge is then transferred to the 1.5 day feed tank at a pH of 11.5 for 22 hours before being pumped to the screw press. A screw press uses a slowly rotating, hollow screw filled with steam to dewater biosolids continuously. Dewatering is achieved by reducing the longitudinal volume (*i.e.* tapered screw shaft) as the biosolids are conveyed towards the end of the screw press. As the biosolids are compressed, the water removed is collected and recycled to the front of the wastewater treatment plant. Similar to a belt filter press, biosolids are typically dewatered through the following three steps: chemical conditioning, gravity drainage, and compaction of the dewatered solids. Steam is applied within the press to meet class A biosolids criteria. Final dewatered solids content are usually 25% to 40%.

Table 5-8 Design Criteria for NWRF Solids Handling Pumps and Mixers

Pump Description	Design Point	HP	Control	Number
Scum Pump Positive Displacement	100 gpm @ 20' head	7.5 hp	On/Off, Level	1
Lime Mix Tank Flooded Suction Centrifugal	350 gpm @ 27' head x gpm @ 6' head	7.5 hp	On/Off, Level	3
Solids Dewatering Feed Positive Displacement	60 gpm @ 26' head	5 hp	On/Off, Level	2
WAS Tank Mixers, Submersible	550 N	4 hp	On/Off	2 Total
Lime Mix Tank Mixer, Submersible	1200 N	8.5 hp	On/Off	1 Total
Feed Tank 1&2 Mixers, Submersible	1200 N	8.5 hp	On/Off	2 Total

NWRF LOCATION AND SITING

The NWRF is located north of Highway 52 and east of County Line Road along Boulder Creek. The NWRF site is a Greenfield project that was previously a gravel mine. Table 5-9 gives the mailing address and legal description for the site. The North Water Reclamation Facility (NWRF) will be located approximately 2.6 miles north of the Town of Erie in Weld County at 40 degrees, 5.500 minutes latitude and 105 degrees, 3.083 minutes longitude. More generally, the NWRF will be located on the northeast corner of the intersection SR 52 and County Line Road in Weld County. The facility will discharge to Boulder Creek approximately one mile below the confluence of Coal Creek with Boulder Creek and approximately six miles above the confluence of Boulder Creek with St. Vrain Creek.

Table 5-9 Town of Erie North Water Reclamation Facility Legal Description and Capacity

Characteristic	Description
WWRF Mailing Address	Town of Erie 645 Holbrook P.O. Box 100 Erie, Colorado 80516
WWRF Location	501 State Highway 52 Erie, Colorado 80516
Site Legal Description	In the east half of Section 31, T2N, Range 68 West of the 6th PM of Weld County
Permitted Hydraulic Capacity, mgd	1.5
Permitted Organic Capacity, ppd BOD_5	3,233

NWRF BIOSOLIDS HANDLING

The NWRF will produce Class A biosolids suitable for public take away. Currently, all biosolids produced at the SWRF are hauled to a regional stabilization facility. The regional facility land applies sludge for beneficial reuse. The Town plans to continue beneficial use of its biosolids through land application.

NWRF SCHEMATIC OF TREATMENT WORKS

A site layout is shown in Figure 5-5. A plant process diagram and hydraulic profile are included as Figures 5-1 and 5-2, respectively.

NWRF ODOR CONTROL CONSIDERATIONS

Wastewater treatment facilities are required to address odor control in their site location approval application. The Water Quality Control Division has adopted Policy No. WQSA-7 to describe odor control and mitigation measures including suggested setbacks from habitual structures. Suggested setbacks for mechanical treatment facilities like the NWRF are 1,000 feet from the nearest habitable structure. There are no homes within 1,000 feet of the NWRF. Although the NWRF meets setback recommendations, processes that are likely to generate odors such as the plant headworks and solids dewatering processes are completely enclosed within buildings to minimize odors.

NWRF AIR QUALITY PERMIT

Some wastewater treatment plants have processes that are identified as stationary sources of air pollutants. Consequently, wastewater treatment plants with a design capacity of 10 million gallons per day or greater may require an air quality permit. The NWRF does not meet the threshold criteria and is not required to have an Air Quality Permit.

NWRF STORMWATER MANAGEMENT PLAN

Wastewater treatment plants with a design capacity over 1 million gallons per day and any plant with a pretreatment program are required to prepare a permanent stormwater management plan as part of the stormwater permitting process. The stormwater management plans for the NWRF and SWRF are included in Appendix G.

NWRF SITE CHARACTERIZATION REPORT

The site location approval process for new wastewater treatment works and new lift stations requires evidence of the suitability of the site. The site shall be characterized in relation to soils, geological hazards, floodplains and other natural hazards. Specifically, the WUP shall identify flood hazard issues and geological suitability issues related to the proposed site and the measures to be taken to mitigate any identified problems or risks. The site characterization information is included in Section 3 of this report under “Environmental Components Evaluation”. The NWRF lies within the natural floodplain of Boulder Creek, however the site has been built up to bring all process buildings and tanks above the 100 year floodplain elevation. The geotechnical report for the site is included in Appendix C.

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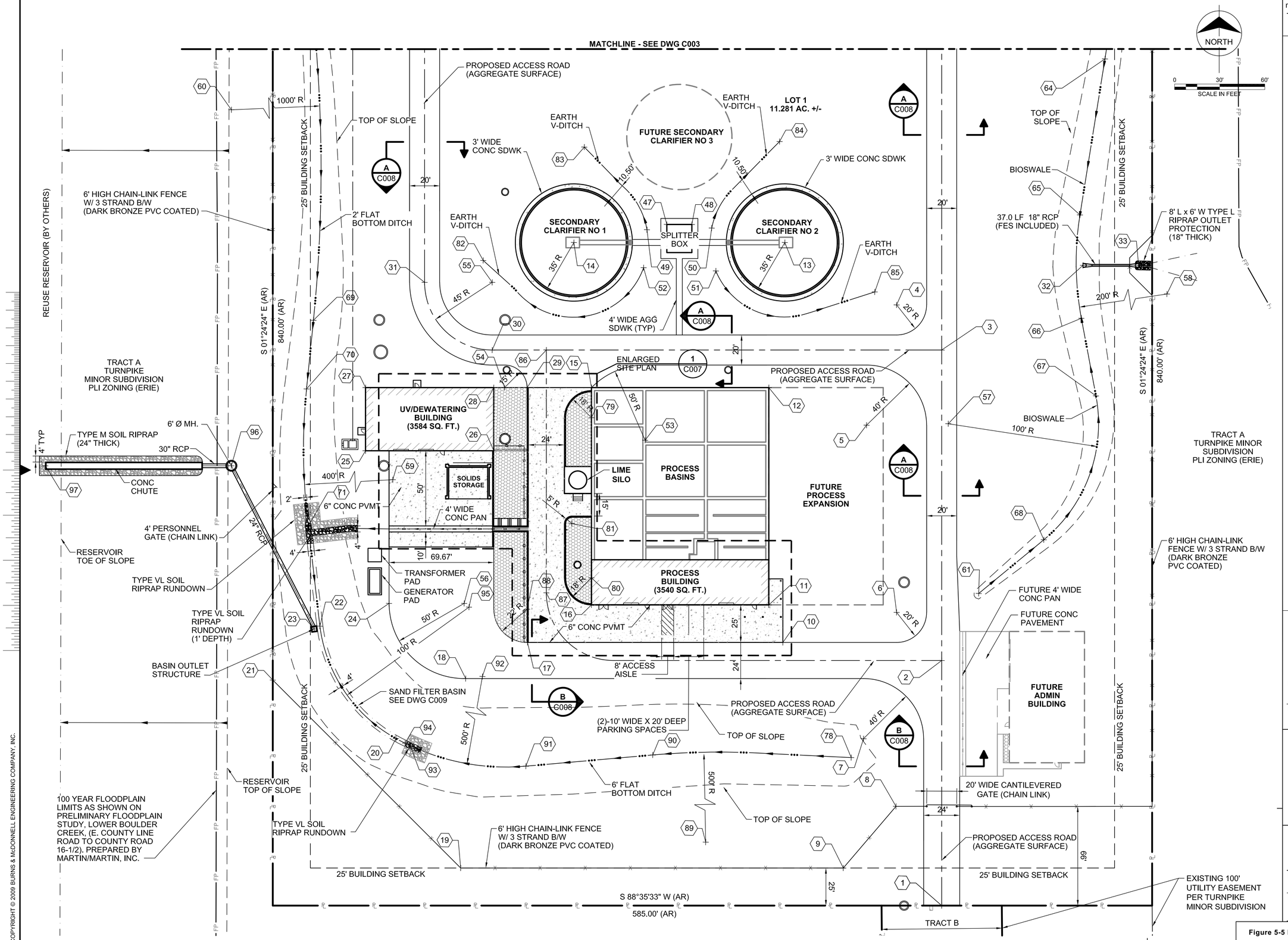

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date	detailed
SEPTEMBER, 2009	G. GARCIA
designed	checked
J. SHEPHERD	N. TESSITORE

TOWN OF ERIE 1874

TOWN OF ERIE
NORTH WATER RECLAMATION FACILITY

SITE PLAN II

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Figure 5-5 North Water Reclamation Facility Site Layout

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SWRF PROCESS SYSTEM

The SWRF was constructed in 1998 with a design capacity of 0.6 mgd to replace an aging lagoon treatment facility. Two of the lagoon cells remain, but are no longer in service. Four years later, the facility was rated at 0.8 mgd of capacity. In 2003, the SWRF was expanded for a hydraulic capacity of 1.2 mgd and an organic capacity of 2,900 ppd of BOD. The expansion included an influent pump upgrade, replacement of a mechanical screen, increased capacity for the aeration system, blower replacement, construction of a clarifier splitter structure, construction of a second secondary clarifier, modifications to the RAS and WAS pumping systems, replacement of the UV disinfection unit, piping modifications, and a decant system for the digesters (HDR Engineering, Inc., 2003). The SWRF was rerated again in 2006 at 1.6 mgd and 3,870 ppd of BOD (Burns and McDonnell, 2006). A new discharge permit which incorporates the rerated capacity was issued in September 2011 and became effective October 1, 2011. The current permit lists capacity at 1.6 mgd and 3,870 ppd of BOD.

The SWRF process diagram and hydraulic profile are shown in Figures 5-6 and 5-7, respectively. The SWRF headworks consists of an influent pump station, step screen, manual bar screen, and two grit basins operated in series. Wastewater is treated to remove organic matter and suspended solids with an activated sludge process followed by secondary clarification and ultra-violet disinfection. The treatment process was not originally designed to nitrify or denitrify; however, effluent ammonia concentrations are typically below 3 mg/L and effluent nitrate concentrations are typically below 15 mg/L. Residual solids from the activated sludge process are stabilized with aerobic digestion prior to land application. Stabilized biosolids are thickened within the digester though multiple decant cycles. The headworks and disinfection equipment are enclosed in buildings. All other processes are outdoors.

SWRF INFRASTRUCTURE SIZING AND STAGING

The SWRF rerating study was approved by the State of Colorado Water Quality Control Division on October 31, 2006 (Falco 2006). The approved hydraulic and organic loading rates are 1.6 MGD and 3,870 ppd of BOD. Design criteria are presented in Table 5-10.

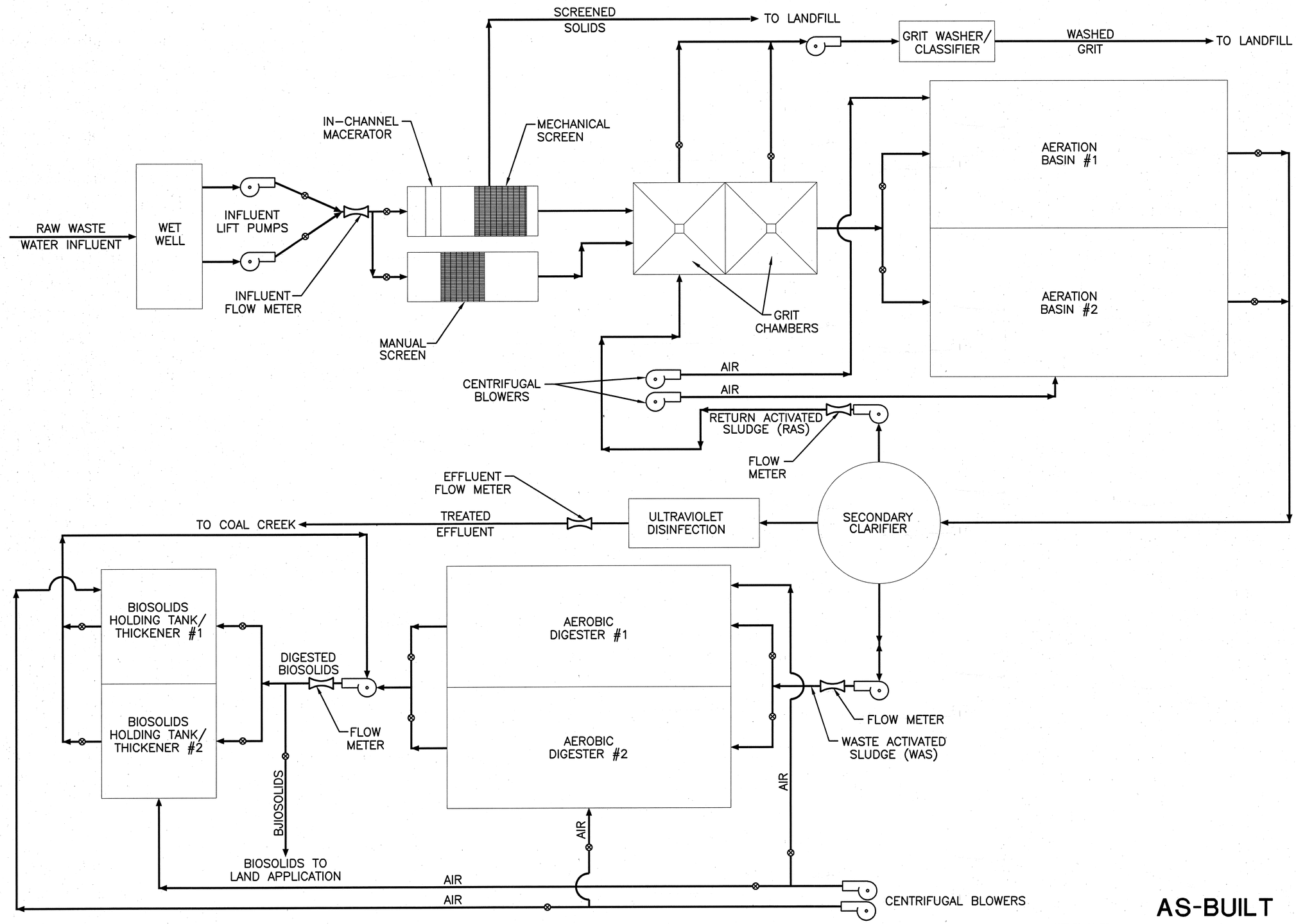
Table 5-10 Design Criteria for SWRF

Parameter	Permitted Capacity	Rerated Capacity
Average Daily Flow, MGD	0.94	1.26
Maximum Month Flow, MGD	1.20	1.60
Peak Day Flow, MGD	1.98	2.65
Peak Hour Flow, MGD	3.90	4.42
Peak Month BOD, ppd	2,900	3,870
Peak Month TSS, ppd	2,690	3,590
Peak Month Ammonia, ppd	340	454

Note: Loadings are based on the 30-day average value for the maximum month.

SWRF Headworks

Headworks facilities for the SWRF consist of influent pumping, screening, and grit removal. Raw wastewater enters the influent pump station wet well through an eighteen-inch gravity sewer. Design



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REVISIONS			
NO.		DATE	BY

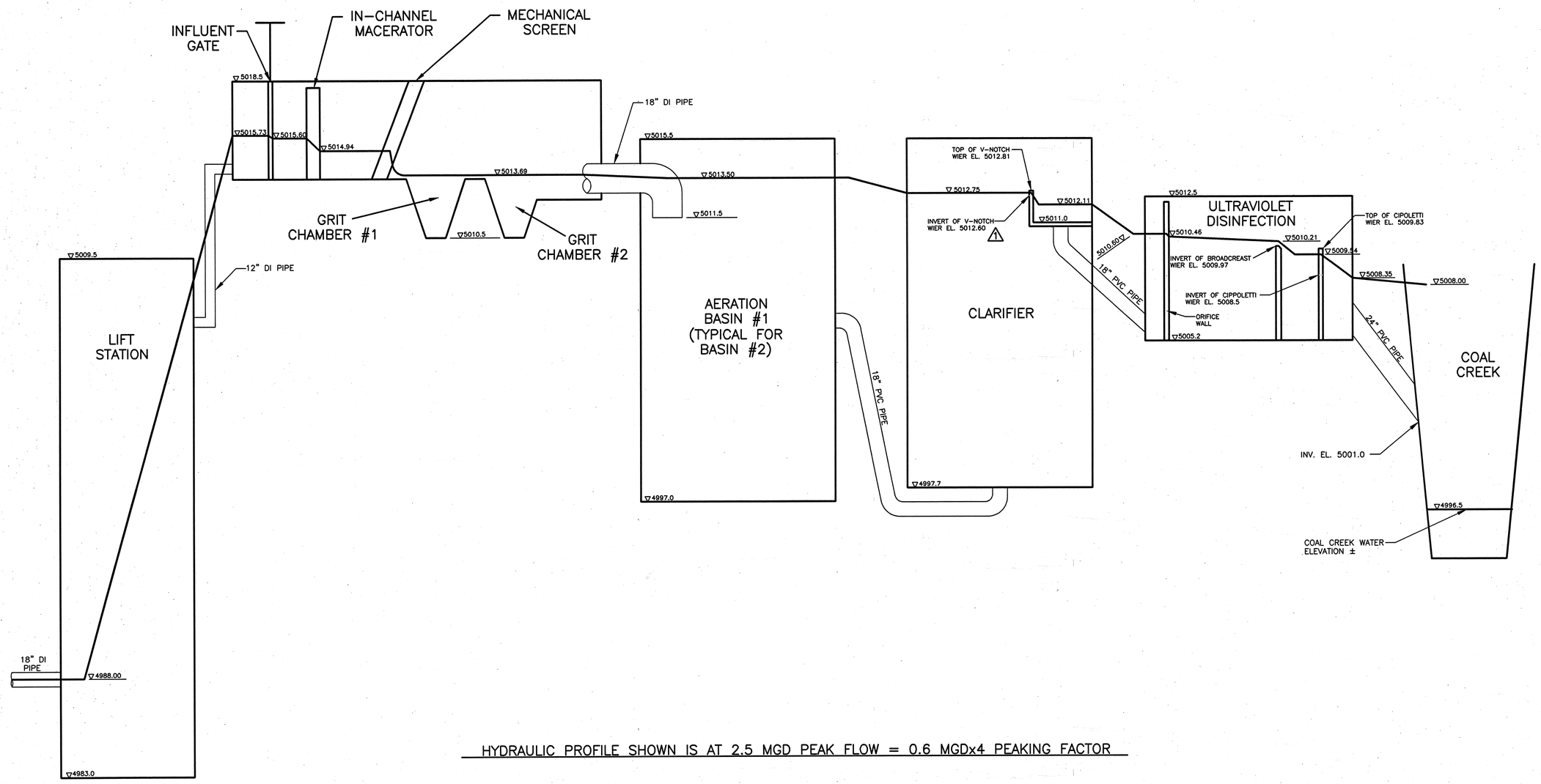
TOWN OF ERIE, COLORADO
0.6 MGD WASTEWATER TREATMENT FACILITY
PROCESS SCHEMATIC

DATE: OCT.'97
DESIGNED BY: JNY
DRAWN BY: FFF
CHECKED BY: MCK

JOB NO. 0885.011.03

SHEET 2 OF 40

Figure 5-6 South Water Reclamation Facility Process Diagram



HYDRAULIC PROFILE SHOWN IS AT 2.5 MGD PEAK FLOW = 0.6 MGDx4 PEAKING FACTOR

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NO.		DATE	BY
1	ADDENDUM #3	2/98	JNY

TOWN OF ERIE, COLORADO
0.6 MGD WASTEWATER TREATMENT FACILITY
HYDRAULIC GRADIENT PROFILE

DATE: OCT. '97
DESIGNED BY: JNY
DRAWN BY: FFF
CHECKED BY: MCK

JOB NO. 0885.011.03

SHEET 3 OF 40

Figure 5-7 South Water Reclamation Facility Hydraulic Profile

criteria for the influent wet well and influent pumps are presented in Tables 5-11 and 5-12, respectively. The influent wet well has a maximum volume of 61,052 gallons.

Table 5-11 Design Criteria for SWRF Influent Wet Well

Parameter	Value
Width, feet	7
Length, feet	44 ft 2 inches
Bottom Elevation, feet	4983.0
Top Elevation, feet	5009.5
Influent Pipe Centerline Elevation, feet	4988.0
Pump Intake Centerline Elevation, feet	4984.0
Total Volume, cf	8,162
Total Volume, gallons	61,052

Table 5-12 Design Criteria for SWRF Influent Pumps

Influent Pumps			
Type	Self-Priming Centrifugal		
Number	3		
Manufacturer	Gorman Rupp		
Model Number	T Series T8A3S-B/F		
Impeller Diameter	Standard		
Impeller Speed	650 / 1350 RPM		
Capacity, each	1,350 gpm (1.94 MGD)		
Firm Pumping Capacity	3.89 MGD		
Pump ID	1	2	3
Serial Number	1266423	1266422	1266421
Influent Pump Motors			
Type	Severe Duty TEFC		
Manufacturer	Weg		
Model Number	W21 CC029A		
Frame	326T		
Power	50 HP, 2100 RPM, 230V / 128 Amps / 3 Ph, 60 Hz		
Serial Number	10JAN07 BZ89319	08AUG06 BX67523	10JAN07 BZ89316

The influent pump station is equipped with three, self-priming centrifugal pumps. The pumps are located on the lower level of the process building in the pump room. Each pump is equipped with a variable frequency drive and has a maximum pumping capacity of 1,350 gpm (1.94 MGD). With the largest pump out of service, the lift station has a firm pumping capacity of 3.89 MGD.

Raw wastewater is lifted 27.76 feet from the influent pipe centerline to the influent channel (Rocky Mountain Consultants, Inc. 1997). Influent flow is measured by a 12-inch magnetic flow meter in the discharge line between the influent pumps and the influent channel. The magnetic flow meter has a maximum capacity of 8 mgd. The influent channel contains a 12-inch overflow pipe that directs excess wastewater flows back into the influent pump station wet well when the water level in the influent channel exceeds 3 feet. The overflow pipe prevents the influent channel from overflowing in the event of a downstream blockage.

Screening equipment at the SWRF consists of an automatic step screen within the main influent channel and a manual bar screen in the bypass channel. Two, fifteen-inch channel gates may be used to divert flow to either or both channels. Under normal operating conditions, all flow is conveyed through the main channel. The manual bar screen in the bypass channel has 1-inch openings between the bars and a rated capacity of 6.4 MGD. The influent channels are each 2 ft wide and 5 ft deep including 2 ft of freeboard. Design criteria for the automatic step screen and screening wash press are given in Table 5-13.

Table 5-13 Design Criteria for SWRF Vulcan Step Screen and Screening Wash Press

Characteristic	Step Screen	Wash Press
Number of Units	1	1
Equipment Designation	7022	7022
Manufacturer	Vulcan	Vulcan
Type/Model	Step Screen ESR 23/3576	EWP 150/600
Bar Spacing, mm	6	N/A
Rated Capacity, MGD	4.0	N/A
Power	2 HP, 1725 RPM, 230/460 V, 3 phase, 50 Hz	3 HP, 1760 RPM, 230/460 V, 3 phase, 50 Hz

The step screen and wash press were installed in 2007 as an in-kind replacement of a JWC Environmental combination "Channel Monster" and "Auger Monster". The Channel Monster is a comminuter which grinds up larger materials such as rags and sticks. The Auger Monster contains a perforated plate screw that removes some of the ground material from the channel and deposits it into a collection bin or dumpster. The Channel/Auger monster combination allowed a significant amount of screenings to pass through to downstream processes. The automatic step screen removes material from the influent wastewater on a regular basis with minimal attention from staff. The combination of removing the grinder, closer bar spacing (6 mm), and regular cleaning removes more material from the influent wastewater such as clumps of toilet paper, rags, trash, sticks, and other materials. Removing these materials increases process efficiency and decreases wear and tear on downstream equipment. Process efficiency is increased because inert materials are removed rather than being allowed to accumulate in the aeration basins and digesters.

The step screen transfers wet screenings to a wash press for further processing. The wash press sprays the screenings with rinse water to remove organic material before compacting the screenings to reduce the total volume for disposal. Screenings are collected in a small dumpster. Rinse water is returned to the influent channel. After screening, wastewater flows to two grit basins operated in series. Grit chambers remove heavier constituents of wastewater such as sand and eggshells and allow less dense organic solids to pass through.

Design criteria for the grit basins are presented in Table 5-14. Each grit basin is 7 feet wide and 7 feet long. Each basin is comprised of an upper rectangular portion and a lower trapezoidal portion that serves a grit hopper. The grit hoppers each have 1:1 side slopes that terminate in 12-inch discharge pipes. Grit collects in the bottom of the hoppers and is removed by the grit pumps. Design criteria for the grit pumps are presented in Table 5-15. The influent lift station dampens the effects of diurnal flow variations by serving as a flow equalization basin. The maximum flow rate that can be received by unit processes

downstream from the influent pump station is the maximum lift station pumping rate. For evaluation purposes, it was assumed that the maximum influent flow rate equal the firm pumping capacity.

Aerated grit basins are sized based primarily on hydraulic retention time with retention times of 2-5 minutes at peak hour flows being typical. The SWRF aerated grit basins have a flow capacity of 1.41 MGD, based on a three minute hydraulic retention time. When peak hour flows exceed this amount, grit removal efficiencies will decrease and a higher percentage of grit will pass through to downstream processes. The influent flow at the SWRF is 3.89 MGD when two influent pumps are operating at full capacity.

Table 5-14 Design Criteria for the SWRF Grit Basins

Parameter	Value
Number	2
Type	Aerated
Length, feet	7
Width, feet	7
Depth from high water level to top of trapezoidal hopper, feet	3
Depth of trapezoidal hopper, feet	3
Volume, cf, each	196
Volume, gallons, each	1,466
Volume, gallons, total	2,932
Operation	Series
HRT[1], Maximum Month, minutes	3.5
HRT[1], Two Influent Pumps On, minutes	1.1
Rated Capacity at 3 minutes HRT	977 gpm (1.41 MGD)
Rated Capacity at 1.0 fps influent velocity and a 90 second hydraulic detention time[2,3]	1,955 gpm(2.81 MGD)

[1]HRT is hydraulic retention time. Maximum month flow is 1.2 MGD. Influent flow rate with two influent pumps on-line at maximum rate is 3.89 MGD
[2]Influent velocity calculation based on cross-sectional area for upper portion of grit basins alone. Cross sectional area of hoppers was not included since flow cannot move through this area. Hydraulic profile from 1997 drawing set indicates that cross-sectional area above grit hoppers is negligible (Rocky Mountain Consultants, Inc. 1997).
[3]Recommended design criteria for Horizontal-flow grit chambers (Metcalf and Eddy 2003)

Table 5-15 Design Criteria for the SWRF Grit Pumps

Grit Pumps		
Type	Non-clog Centrifugal	
Number	2	
Manufacturer	Haywood Gordon Inc.	
Model Number	XR3-11	
Impeller Diameter		
Impeller Speed	960	
Capacity	200 gpm at 30 ft head	
Pump ID	1	2
Serial Number	276188-2	276188-1
Grit Pump Motors		
Manufacturer	Baldor Super E	Emerson
Model Number	MOO 80517019	AB17
Spec	M37F614Y568	X10E28
Frame	215T	215T
Power	10 HP, 1760 RPM, 230/460 V / 25.8/12.9 Amps / 3 Ph, 60 Hz	
NEMA Nom. Eff.	91.7	91
Serial Number	P698	H05-AB17

Screened and degritted wastewater exits the headworks building through the grit basin outlet channel and is conveyed to the secondary treatment process via an 18-inch diameter pipeline.

SWRF Secondary Treatment Process

The SWRF secondary treatment process is a conventional activated sludge system. Activated sludge is a biological treatment process designed to remove organic matter and total suspended solids. The SWRF produces high quality secondary effluent. Effluent data for BOD, TSS, ammonia, nitrate, pH, and temperature are presented in Tables 5-16 through 5-21. Effluent BOD and TSS concentrations have been below the permitted effluent concentrations of 30 mg/L. Occasionally, foaming in the activated sludge basins has elevated effluent TSS concentrations.

The SWRF is designed to oxidize ammonia, but was not designed to remove nitrate or phosphorus. Operations staff utilize a pattern of on/off aeration in the activated sludge basins to partially denitrify. Effluent ammonia concentrations have remained below 3 mg/L as N and effluent nitrate concentrations are typically below 15 mg/L.

Effluent pH averaged 6.93 between 2003 and 2010 and ranged between 6.07 to 7.34 S.U. Treatment plants like the SWRF that nitrify often have effluent pH values slightly below 7.0 SU. Effluent temperatures vary seasonally. Winter water temperatures are close to 16 degrees Celsius while summer water temperatures reach as high as 21 degrees Celsius.

Table 5-16 Monthly Average Effluent BOD, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010
January	6.77	8.08	6.55	11.52	11.68	11.00	6.75	12.8
February	6.36	6.96	6.50	6.00	6.79	14.00	10.75	17.8
March	11.77	6.30	11.90	4.84	6.84	10.50	12.25	16.8
April	11.10	5.77	6.67	5.27	8.00	8.00	9.20	15.8
May	6.39	4.48	7.16	5.52	11.29	20.50	7.00	15.3
June	4.77	5.17	12.87	6.23	9.27	9.25	10.50	8.6
July	6.90	6.19	22.65	5.26	14.71	5.80	9.60	6.3
August	4.10	6.19	11.87	4.39	7.50	5.25	13.50	5.0
September	5.03	4.93	8.60	6.07	7.00	5.50	7.60	5.5
October	6.15	4.90	4.29	10.06	10.40	11.00	10.25	8.8
November	4.55	7.33	12.03	6.77	13.40	11.25	10.0	11.0
December	5.71	9.61	9.26	8.77	8.75	10.80	11.8	14.8
Average	6.63	6.33	10.03	6.73	9.64	10.24	9.93	11.4

Table 5-17 Monthly Average Effluent TSS, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010
January	9.29	15.10	10.39	25.56	22.48	24.00	13.75	28.0
February	11.21	18.38	12.50	9.57	13.68	26.40	22.75	39.0
March	16.45	14.95	20.35	9.71	12.65	21.00	28.40	27.3
April	19.22	12.60	11.80	11.63	12.40	17.20	18.80	32.1
May	12.68	10.13	13.45	12.84	21.19	44.60	11.25	27.2
June	9.23	10.27	26.03	12.90	21.83	17.25	20.00	20.6
July	15.10	18.48	24.93	13.23	28.19	11.80	22.00	14.0
August	9.68	23.77	14.32	8.23	26.00	10.75	30.50	8.80
September	9.90	15.50	18.20	12.07	16.25	11.25	20.00	10.0
October	10.69	12.61	7.87	18.06	19.60	19.60	27.00	15.5
November	8.23	14.30	24.67	11.07	26.75	23.00	22.0	21.2
December	10.39	16.13	27.03	14.23	19.75	23.00	24.6	22.8
Average	11.84	15.19	17.63	13.26	20.06	20.82	21.75	22.9

Table 5-18 Monthly Average Effluent Ammonia, mg/L as N

	2003	2004	2005	2006	2007	2008	2009	2010
January	2.98	2.30	2.46	2.04	1.17	1.26	0.87	2.28
February	2.23	1.07	1.29	4.32	0.44	1.59	1.58	1.87
March	1.94	2.09	1.76	3.40	0.73	1.65	2.39	2.84
April	2.77	2.17	0.86	2.06	0.96	2.44	2.18	2.81
May	1.26	1.75	1.01	0.58	0.83	1.73	1.82	3.67
June	1.36	1.60	1.64	0.95	0.67	1.83	1.55	2.47
July	2.33	2.07	7.88	0.92	1.92	1.30	1.82	0.68
August	0.59	1.30	14.43	0.58	0.48	1.64	2.36	0.94
September	1.39	2.98	2.40	1.02	1.25	1.99	1.03	1.30
October	0.86	2.09	1.37	1.81	1.00	1.79	1.09	1.34
November	0.83	2.63	1.66	0.75	1.18	1.93	1.86	1.16
December	2.54	2.56	0.48	1.06	1.10	1.92	2.93	1.24
Average	1.76	2.05	3.10	1.63	0.98	1.75	1.79	1.92

Table 5-19 Monthly Average Effluent Nitrate, mg/L as N

	2003	2004	2005	2006	2007	2008	2009	2010
January	7.2	0.6	NR	NR	NR	11.6	12.4	3.85
February	14.8	NR	NR	7.2	12.0	3.2	2.2	4.23
March	NR	NR	0.7	3.7	NR	8.2	2.0	4.30
April	NR	NR	NR	8.6	16.8	7.8	3.3	3.17
May	12.0	NR	NR	NR	NR	5.6	2.6	2.42
June	NR	NR	NR	12.0	12.8	5.4	2.4	4.13
July	NR	NR	16.0	NR	19.2	6.4	2.8	6.67
August	15.8	NR	16.0	12.8	NR	6.0	2.3	6.85
September	NR	NR	5.2	25.6	16.4	6.2	4.5	6.45
October	NR	NR	16.0	14.4	16.0	14.4	4.8	6.73
November	NR	NR	12.0	NR	17.3	12.8	No Data	6.60
December	NR	NR	NR	NR	8.2	NR	4.6	6.60
Average	12.5	0.6	11.0	12.0	14.8	8.0	4.0	4.57

Table 5-20 Monthly Average Effluent pH, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010
January	7.04	6.88	6.84	6.87	6.73	6.78	6.72	6.83
February	7.00	6.97	6.86	7.04	6.66	6.90	6.92	6.85
March	7.13	6.90	7.02	7.06	6.75	6.92	6.98	6.90
April	7.32	7.05	7.01	7.08	6.81	6.93	7.01	6.95
May	7.22	7.04	7.09	6.82	6.91	6.97	7.03	7.04
June	7.06	7.08	7.06	6.80	6.94	7.07	7.07	7.11
July	7.06	7.06	7.22	6.90	6.89	7.04	7.05	7.06
August	6.07	6.98	7.34	6.84	6.74	7.00	7.07	7.01
September	7.06	7.11	7.03	6.65	6.63	7.00	6.95	6.98
October	6.93	6.93	7.16	6.67	6.66	6.95	6.95	6.90
November	6.89	6.92	6.91	6.66	6.68	6.78	6.66	6.92
December	7.05	7.00	6.90	6.64	6.73	6.76	6.69	6.94
Average	6.99	6.99	7.03	6.84	6.76	6.93	6.93	6.96

Table 5-21 Monthly Average Effluent Temperature, mg/L

	2003	2004	2005	2006	2007	2008	2009	2010	Average
January	16.5	15.8	15.9	15.4	15.1	15.8	16.6	16.1	15.9
February	16.0	15.2	15.1	14.9	14.8	15.6	16.2	15.7	15.4
March	15.9	15.7	14.9	15.5	15.8	15.7	16.3	15.9	15.7
April	17.2	16.7	15.9	16.7	16.2	16.2	16.7	16.5	16.5
May	17.8	18.9	17.3	17.6	17.5	17.3	17.9	17.3	17.7
June	19.6	20.1	18.6	19.4	18.6	18.7	19.0	18.8	19.1
July	20.3	20.0	19.6	20.6	19.9	20.0	20.1	20.2	20.1
August	20.5	20.3	20.3	21.2	21.1	21.0	20.8	21.1	20.8
September	19.8	19.8	20.8	20.5	20.9	21.1	20.9	21.0	20.6
October	18.7	18.6	19.4	19.5	20.2	20.2	19.8	20.8	19.7
November	17.5	17.2	18.0	17.9	18.4	19.0	No Data	19.1	18.2
December	16.7	16.0	16.5	16.6	16.7	17.5	No Data	17.9	16.8
Average	18.0	17.9	17.7	18.0	17.9	18.2	18.4	18.4	--

The SWRF secondary treatment process is a conventional activated sludge system. It consists of two aeration basins operated in parallel and two secondary clarifiers. Ancillary equipment includes the aeration system, dissolved oxygen probes, scum removal, RAS and WAS pumps.

Aeration Basins. Design criteria for the aeration basins are presented in Table 5-22. Each basin holds 0.32 MG for a total volume of 0.64 mg. The current rated capacity is 1.6 MGD, 3,870 ppd of BOD, and 454 ppd of ammonia nitrogen.

Table 5-22 Design Criteria for SWRF Aeration Basins

Parameter	Value
Number, each	2
Length, feet	84
Width, feet	32
Depth, feet	16.5
Volume, cf, each	44,468
Volume, gallons, each	320,000
Volume, million gallons, each	0.32
Volume, million gallons, total	0.64
MLSS, mg/L	4,000
SRT, days	10
Space Loading, lb BOD/1000 cf*day	43
Hydraulic Retention Time, hours	9.6

Aeration System. The aeration system consists of three blowers and associated air piping, valves, and diffusers. Design information for the blowers is provided in Table 5-23. Three additional blowers provide air to the aerobic digesters. The aeration system and digester blowers share a common header.

Table 5-23 Design Criteria for SWRF Activated Sludge Blowers

Aeration Basin Blowers			
Type	Multi-Stage Centrifugal		
Number	3		
Manufacturer	Gardner Denver Lamson		
Model Number	859-0-0-2-7-0-0-0-AD01		
Material of Construction	Cast Iron		
Drive	Direct – Constant Speed		
Horsepower	125		
Design Air Flow, scfm, each	900 – 1,500		
Design Air Flow, scfm, firm	900 – 3,000		
Blower ID	1	2	3
Serial Number	P004491	P004490	P004492
Motors for Aeration Basin Blowers			
Type	3-Phase Induction Motor, Continuous Duty		
Manufacturer	Toshiba Epact High Efficiency		
Model Number	2B4125L1C6757		
Frame	404TS		
Power	125 HP, 3560 RPM, 460 V / 147 Amps / 3 Ph, 60 Hz		
NEMA Des / NEMA Code	B / F		
NEMA Nom. Eff.	93.6		
Blower ID	1	2	3
Serial Number	020806531	020806533	020806532

Dissolved Oxygen Probe. The aeration basins are equipped with Endress Hauser dissolved oxygen probes. The probes measure dissolved oxygen concentrations in the aeration basins. The probes send a continuous signal to a PLC. Airflow may be modulated based on basin DO concentration through automatic valves on each inlet. Staff uses a dissolved oxygen setpoint of 2.0 mg/L with low and high level alarms set at 0.7 mg/L and 2.6 mg/L, respectively. The SCADA system records low and high dissolved oxygen alarm conditions as events during non-business hours.

Blower on/off cycles are controlled with a timer on the PLC. As of June 2010, the blowers were programmed to complete a cycle of five hours on followed by one hour minutes off. The blowers complete four cycles each day.

Diffusers. The aeration basins at the SWRF have tubular membrane fine bubble diffusers installed along the bottoms of the aeration basin to distribute the air from the blowers. Each diffuser is 6.4 cm in diameter and 2 meters long. The diffusers serve to facilitate the transfer of oxygen from air to water so the microorganisms can "breathe" and to mix the wastewater.

Each aeration basin contains four banks of diffusers. Each bank of diffusers contains four rows of diffusers that are fed by a single air supply drop pipe. The diffusers are mounted on 6-inch headers. The air supply to each each bank of diffusers can be controlled via butterfly valves. Anoxic zones can be created by completely closing one or more valves. The 6-inch air headers connect to a 12-inch air pipe that runs along the top of the aeration basin building back to the Lamson blowers.

In addition to providing oxygen for aerobic biological processes, air also provides mixing energy to keep the microorganisms in suspension and to bring food and nutrients in contact with the microorganisms. Both the USEPA Design Manual for Fine Pore Aeration Systems and MOP 8 (Water Environment Federation 2010) state that fine bubble diffusion systems provide sufficient mixing in an activated sludge process at an air flow rate of 0.12 scfm per ft^2 of reactor. For the SWRF, a minimum of 645 scfm is needed for mixing.

The SWRF has 104 diffusers in each aeration basins for a total of 208 diffusers. At the manufacturer's recommended air flow of 10.8 scfm per diffuser, the air flow rate in each aeration basin will be 1,123 scfm. At the maximum air flow rate of 14 scfm, the air flow rate in each aeration basin will be 1,456 scfm.

Secondary Clarifiers. The SWRF has two secondary clarifiers. Both clarifiers are center feed, peripheral take-off with suction type collectors. Design criteria are presented in Table 5-24. Clarifier #1, also located outside of the main plant building, was constructed as part of the 2003 expansion. Clarifier #1 has a diameter of 50 feet and is 15 feet deep.Clarifier #2 is located outside of the main plant building and was constructed in 1997. It has a diameter of 50 feet and is 15 feet deep.

Table 5-24 Design Criteria for SWRF Secondary Clarifiers

Parameter	1.2 MGD Expansion	1.6 MGD Rerating
Number	2	
Diameter, feet	50	
Depth, feet	15	
Volume, each, cf	29,543	
Volume, each, MG	0.22	
Surface Area, each, sf	1,960	
Clarifier Mechanism Type	Rapid sludge withdrawal, suction draw header	
Clarifier Power Requirements	0.5 HP, 480 V, 3-Phase, 60 Hz	
SOR[a], maximum month, gpd/sf	306[b]	408[c]
SOR[a], peak hour, gpd/sf	992[b]	992[c]
Weir Loading Rate, max month, gpd/ft	3,821	5,094
Solids Loading Rate, lbs/day/sf[d]	17.3	23.1

[a]SOR is Surface Overflow Rate. Assumes both clarifiers are in service.
[b]Maximum month flow is 1.2 MGD. Influent flow rate with two influent pumps on-line at maximum rate is 3.89 MGD
[c]Maximum month flow is 1.6 MGD. Influent flow rate with two influent pumps on-line at maximum rate is 3.89 MGD
[d]Assumes MLSS concentration of 4,000 mg/L and RAS rate of 70% influent flow.

The secondary clarifiers separate the activated sludge from the treated wastewater. MLSS enters the clarifier in the center where it is briefly contained by the clarifier floc well. The floc well is the metal skirt in the center of each clarifier. Its purpose is to slow down the MLSS and allow for some gentle mixing and reflocculation before directing the MLSS down towards the clarifier floor. Openings along the outside of the floc well dissipate energy and allow scum to escape into the main portion of the clarifier where it can be removed by the skimmer arm. Suspended solids in the secondary clarifier effluent should be less than 10 mg/L if the secondary system is being properly operated and less than 30 mg/L to satisfy permit limits.

Splitter Box The clarifier splitter box can be used to direct flow to either or both clarifiers. Design criteria for the clarifier splitter box are shown in Table 5-25. The splitter box is constructed to distribute flow proportionally to both clarifiers when both clarifiers are on line. Proportional flow splitting ensures that both clarifiers operate under the same solids and hydraulic loading conditions.

Table 5-25 Design Criteria for SWRF Secondary Clarifier Splitter Box

Parameter	Value
Number of Basins	1
Length and Width	10 ft by 15 ft
Number of Weirs/Gates	2
Weir Length, each	4 ft 6 inches

Collection Mechanisms Both of the secondary clarifiers are equipped with skimmer arms and scum collection boxes for removing floatables from the surface of the clarifiers.

Return and Waste Activated Sludge Pumps. Return Activated Sludge (RAS) is withdrawn from the bottom of each of the secondary clarifiers via rapid removal suction headers. RAS pump design criteria are given in Table 5-26. The RAS pumps are essentially vacuuming the bottom of the secondary clarifiers. Suction headers are thought to reduce the total amount of time that solids remain in the clarifier compared to more traditional scraper type collection mechanisms. The RAS lines from each clarifier are

manifolded together inside the lower level pump room. The manifold arrangement allows either of the two RAS pumps to be used with either clarifier or for one RAS pump to pull from both clarifiers. RAS is returned to the aeration basin through the grit basin effluent channel. Decant from the sludge storage basins is also returned to this point. Waste activated sludge is drawn directly from the RAS line by the RAS pumps. Sludge is wasted four times per hour.

Table 5-26 Design Criteria for SWRF RAS and WAS Pumps

Return and Waste Activated Sludge Pumps		
Type	Non-Clog Centrifugal	
Number	2	
Manufacturer	Cornell Pump Company	
Model Number	6NHTA-F16	
Capacity, gpm, each	800	
Pump ID	1	2
Serial Number	106861 13.63 Deg 0 TB02907	106862 13.63 Deg 0 TB02907
Return and Waste Activated Sludge Pump Motors		
Type	Continuous Duty	
Manufacturer	Baldor Industrial Motor	
Model Number	A20454B-55	
Spec	40H028W18361	
Frame	286T	
Power	15 HP, 870 RPM, 460 V / 22 Amps / 3 Ph, 60 Hz	
NEMA Nom. Eff.	85.5	
Motor ID	1	2
Serial Number	09808	09808

SWRF Disinfection

Disinfection at the SWRF is accomplished with a Sunlight Systems UV Disinfection system. UV light inactivates bacteria and viruses by damaging to genetic material (DNA and RNA) that regulates cell function and reproduction in all living organisms. UV altered DNA or RNA inhibits cell replication or induces lethal mutations in daughter cells. Unable to reproduce, the damaged cells cannot cause infection and die out within a short time. UV systems are sized based upon the permitted coliform limits and the peak hourly flow rate. The UV system at the SWRF was designed to treat a peak hour flow of 6.5 MGD.

Design criteria for the UV disinfection units are presented in Table 5-27. The system contains three banks of low-pressure, high-intensity lamps. Under normal operating conditions, two units are always in service with the unit in standby. Two units provide enough firm capacity to treat peak hour flow. Any one of the banks may be taken off-line for cleaning or maintenance while leaving the other two units in service.

Table 5-27 Design Criteria for SWRF UV Disinfection System

Parameter	Value
Type	Vertical, open channel low pressure, high intensity bulbs
Manufacturer	Sunlight Systems
Model Number	WTV-LAVA-25-AM-300-3
Design Treatment Level, fecal coliforms / 100 mL	200
Number of Modules	3
Number of Lamps per Module	25
Total Number of Lamps	75
Design Dosage, mJ/cm^2	30
Design Wastewater Transmittance, Percent	65
Design Lamp Efficiency, Percent	80
Design Sleeve Transmittance, Percent	92
Design Firm Capacity, MGD	4.6
Design Maximum Capacity, MGD	6.5
Depth of Channel at Influent, feet	5.26

SWRF Solids Handling

Waste solids from the activated sludge process (WAS) at the SWRF are stabilized using aerobic digestion. The SWRF has two aerobic digesters and two aerated sludge holding tanks. Design criteria are presented in Table 5-28. The aerobic digesters have a total volume of 370,000 gallons. The aerated sludge holding tanks have a combined volume of 184,000 gallons.

Table 5-28 Design Criteria for SWRF Aerobic Digesters and Biosolids Holding Tanks

Parameter	Aerobic Digesters	Biosolids Holding Tanks
Number of Digesters	2	2
Length, feet	48	24
Width, feet	32	32
Depth, feet	17	17.5
Volume, cf, each	24,730	12,300
Volume, gallons, each	184,980	92,000
Volume, gallons, total	370,000	184,000
Volatile Solids Loading, lb VS/CF	0.035	N/A
Average Digester Thickened Solids, Percent	1.5	N/A
Expected Volatile Solids Reduction, Percent	>45	N/A
SRT at 1.6 mgd, Days	77	N/A

There are two solids transfer pumps that are located in the pump room on the lower level of the main plant building. Pump design criteria are listed in Table 5-29. Sludge transfer pump number 1 may be used to transfer sludge from the RAS line to either Digester 1 or Digester 2. Sludge transfer pump number 2 may be used to transfer digested biosolids from either of the aerobic digesters to either of the biosolids holding tanks or directly to the sludge transfer station. The sludge transfer station may be used to load liquid biosolids into a truck for disposal. Sludge transfer pump 2 may also be used to pull digested biosolids out of the biosolids holding tanks and send them to the sludge transfer station. The sludge transfer pumps are manifolded together in the pump room. This allows pump 1 and pump 2 to

swap duties as needed. An 8-inch magnetic flow meter in the sludge line to the aerobic digesters tracks the total volume of sludge wasted. A second, 8-inch flow meter is located in the crossover line that connects the holding tank line to the digester line.

Table 5-29 Design Criteria for Sludge Transfer Pumps

Sludge Transfer Pumps		
Type	Non-clog Centrifugal	
Number	2	
Manufacturer	Cornell Pump Co.	
Model Number	6NHTA-F16	
Pump ID	1	2
Serial Number	106864 13.25 Deg0 TB02910	106863 13.25 Deg0 TB02910
Solids Transfer Pumps Motors		
Type	SuperE	
Manufacturer	Baldor Electric Co.	
Frame	324T	
Power	25 HP, 1180 RPM, 460V / 31 Amps / 3 Ph, 60 Hz	
NEMA Nom. Eff.	92.4	
Motor ID	1	2
Serial Number	0980805	0980805

The digester tanks are operated in series. Waste activated sludge is directed to the north digester; digester number 1. After multiple cycles of settling and decanting, thickened sludge is transferred from the north digester to the south holding tank. The south holding tank gradually fills until it overflows into the south digester. The south digester is cycled through multiple steps of settling and decanting to get the digested solids as thick as possible. Thickened solids from the south digester are then transferred to the north holding tank. Solids may go through one additional thickening cycle in the north holding tank before being pumped to the sludge transfer station. Historically, solids concentrations up to three percent have been possible. For the last several years, odor control concerns have been more important to the Town than obtaining additional thickening. In 2007, 2008, and 2009, the digested biosolids leaving the facility have been between one and two percent solids.

Air is supplied to the aerobic digesters and sludge holding tanks by three, dedicated 75 horsepower blowers. Design criteria for the blowers are presented in Table 5-30. Each basin is equipped with coarse bubble stainless steel diffusers. There are 24 diffusers in each digester and 16 in each holding tank. Each diffuser is 24 inches long and has a capacity of 10 to 20 scfm. The diffusers serve to facilitate the transfer of oxygen from air to water so the microorganisms can “breathe” and to mix the wastewater.

Liquid Waste Management, a contract hauler, collects the digested solids and takes them to a regional biosolids facility. There, the solids are mixed with biosolids from other facilities prior to land application. In 2009, the SWRF removed roughly twelve, 6,000 gallon trucks each week of 1.2% solid biosolids. Pickup and disposal fees currently total $186.50 per truck or about $116,000 a year. Biosolids generation data for years 2005 through 2009 are presented in Table 5-31. The amount of biosolids generated each year is slowly increasing due to population growth within the service area.

Table 5-30 Design Criteria for SWRF Digester Blowers

Digester Blowers			
Type	Multi-Stage Centrifugal		
Number	3		
Manufacturer	Gardner Denver Lamson		
Model Number	819-0-0-9-0-0-0-0-AD01		
Material of Construction	Cast Iron		
Drive	Direct – Constant Speed		
Horsepower	75		
Design Air Flow, scfm, each	1,000		
Design Air Flow, scfm, firm	2,000		
Blower ID	1	2	3
Serial Number	P004493	P004494	P004393
Motors for Digester Blowers			
Type	3-Phase Induction Motor, Continuous Duty		
Manufacturer	Reliance Electric		Toshiba Epact High Efficiency
Model Number	Duty Master AC-Motor		B0752VLG3USW
Frame	364TS		364TS
Power	75 HP, 3560 RPM, 460 V / 81.9 Amps 3 Ph, 60 Hz		75 HP, 3540 RPM, 230/460 V / 176/88 Amps / 3 Ph, 60 Hz
NEMA Des / NEMA Code	B / P		B / E
NEMA Nom. Eff.	95.0		93.6
Motor ID	1	2	3
Serial Number	01MAN60745G001HA	01MAN60745G003HA	010603194

Table 5-31 Biosolids Generated at SWRF from 2005 through 2009

Parameter	Units	2005	2006	2007	2008	2009
Influent Flow	MGD	0.784	0.841	0.941	0.928	0.967
Influent BOD	mg/L	287	301	283	310	309
Influent BOD	mtpy	311	349	368	397	413
Influent TSS	mg/L	358	371	386	403	405
Land Application Total	DMT	134.16	182.87	210.73	ND	ND
Average Percent Solids	%	1.76	1.66	1.71	ND	ND
Average Yield	lb Biosolids/lb BOD	0.47	0.61	0.75	ND	ND

ND = No data available.

SWRF Biosolids Quality Liquid Waste Management produces an annual report of biosolids quality for the Town for all stabilized waste sludge applied to agricultural land. Biosolids quality data for stabilized waste sludge applied in years 2005 through 2007 are presented in Tables 5-32. The concentrations of regulated trace elements in the Town's biosolids are all below both the maximum allowable concentrations.

Table 5-32 SWRF Biosolids Quality Data for 2005 through 2007

Constituent	Units	4/7/2005	5/25/2005	10/31/2006	5/18/2007	9/14/2007	11/13/2007
Arsenic	mg/kg	1.5	2.2	1.8	3.1		3.9
Cadmium	mg/kg	1.2	2.91	1.91	0.87		0.75
Chromium	mg/kg	9	15.5	5.1	12.5		15.6
Copper	mg/kg	188.9	385.7	510.8	460.5		547.7
Lead	mg/kg	5.7	14.9	14.6	12.3		12.3
Mercury	mg/kg				0.38		0.33
Molybdenum	mg/kg	0.12	0.55	8.3	5.7		5.7
Nickel	mg/kg	1.8	9.9	3.2	6.1		16.9
Selenium	mg/kg	6.2	9.7	11.9	10.4		13
Zinc	mg/kg	1.2	16.1	405.6	364.5		425.6
Salmonella	#/g	221.9	335.1				
Nitrite	%	0.0001	0.0001	0.0567	0.0302	0.0014	0.0022
Nitrate	%	0.0942	0.0001	0.3691	0.6399	0.0021	0.7387
TKN	%	2.307	4.385	3.017	4.294	6.154	4.56
Ammonia	%	0.119	0.02	0.001	0.001	0.097	0.001
Organic Nitrogen	%	2.094	4.364	2.647	3.653	6.054	3.821
Solids	%	4.655	1.506	1.561	1.603	12.926	1.811
Solids Volatile	% of TS			77.74		66.9	
pH	S.U.			5.38		6.71	
Phosphorus	%	0.86	1.909	8.163	1.83	2.059	1.393
Potassium	%	0.228	0.585	0.467	0.591	0.276	0.491
Fecal Coliforms	MPN/g	11622	174590	150227	10678	12064	

SWRF Location and Siting

The South Water Reclamation Facility (SWRF) is located near the center of the service area just north of the intersection of Briggs Street and Evans Street. Briggs Street becomes County Road 1 ½ as it passes the SWRF. The SWRF is an extended aeration activated sludge plant and has a rated capacity of 1.6 mgd and 3,870 ppd of Biochemical Oxygen Demand (BOD). The North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The site legal description is given in Tables 5-33.

Table 5-33 Town of Erie South Water Reclamation Facility Legal Description and Capacity

Characteristic	Description
WWRF Mailing Address	Town of Erie 645 Holbrook P.O. Box 100 Erie, Colorado 80516
WWRF Location	1000 Briggs Street Erie, Colorado 80516
Site Legal Description	In the NE ¼ of the NW ¼ of Section 18, T1N, R68W
Permitted Hydraulic Capacity, mgd	1.6
Permitted Organic Capacity, ppd BOD_5	3,870

SWRF BIOSOLIDS HANDLING

The SWRF produces Class B biosolids. Currently, all biosolids produced at the SWRF are hauled to a regional stabilization facility. The regional facility land applies sludge for beneficial reuse. The Town plans to continue beneficial use of its biosolids through land application.

SWRF SCHEMATIC OF TREATMENT WORKS

A site layout is shown in Figure 5-8. A plant process diagram and hydraulic profile are included as Figures 5-1 and 5-2, respectively.

SWRF ODOR CONTROL CONSIDERATIONS

Wastewater treatment facilities are required to address odor control in their site location approval application. The Water Quality Control Division has adopted Policy No. WQSA-7 to describe odor control and mitigation measures including suggested setbacks from habitual structures. Suggested setbacks for mechanical treatment facilities like the SWRF are 1,000 feet from the nearest habitable structure. The nearest habitable structure is approximately 600 feet south of the SWRF site. Processes that are likely to generate odors such as the plant headworks are completely enclosed within buildings to minimize odors. The headworks and disinfection building is equipped with odor control air handling blowers.

SWRF AIR QUALITY PERMIT

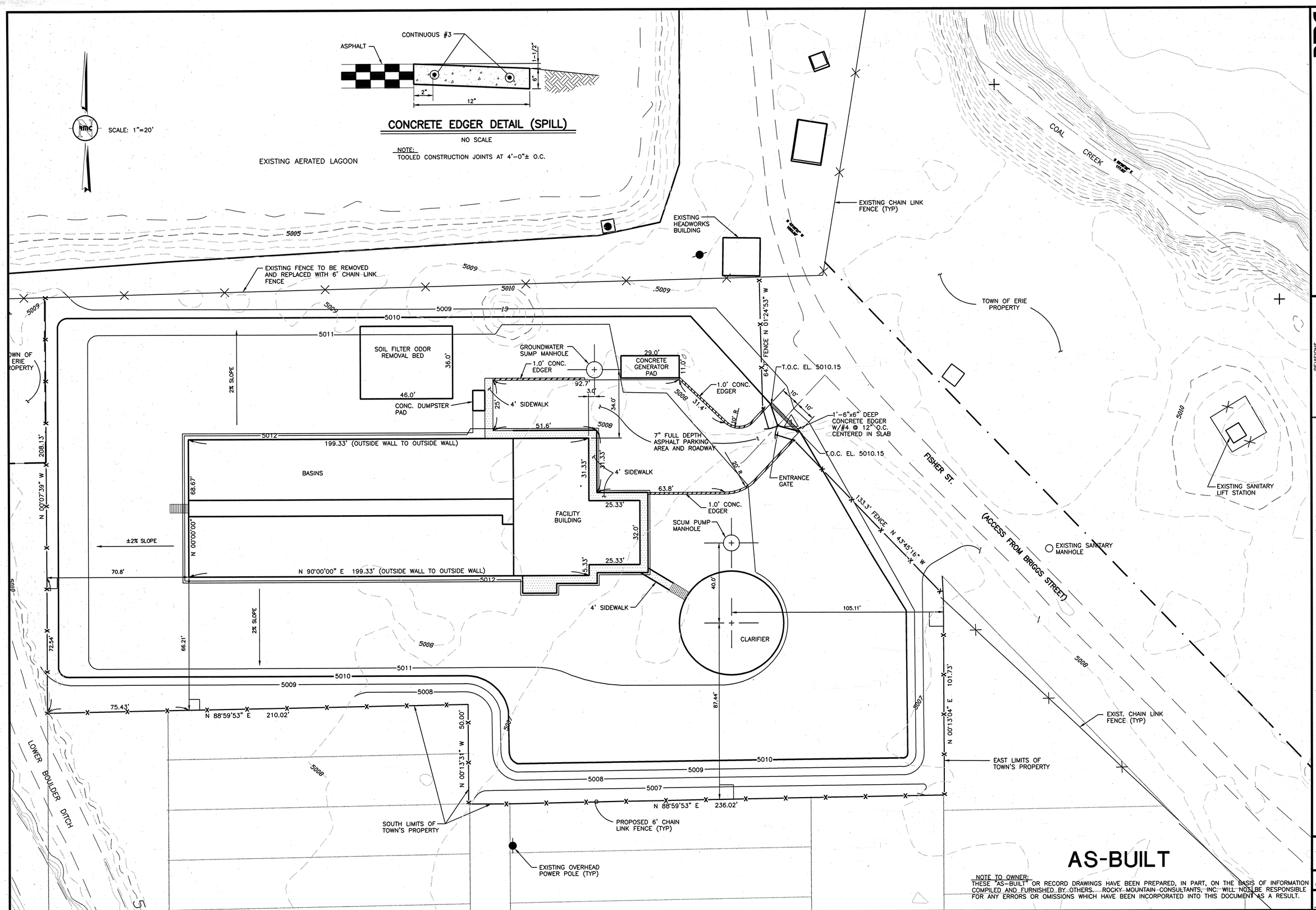
Some wastewater treatment plants have processes that are identified as stationary sources of air pollutants. Consequently, wastewater treatment plants with a design capacity of 10 million gallons per day or greater may require an air quality permit. The SWRF does not meet the threshold criteria and is not required to have an Air Quality Permit.

SWRF STORMWATER MANAGEMENT PLAN

Wastewater treatment plants with a design capacity over 1 million gallons per day and any plant with a pretreatment program are required to prepare a permanent stormwater management plan as part of the stormwater permitting process. The SWRF has an approved stormwater management plan which is included in Appendix G.

SWRF SITE CHARACTERIZATION REPORT SUMMARY

The site location approval process for new wastewater treatment works and new lift stations requires evidence of the suitability of the site. The site characterization information is included in Section 3 of this report under “Environmental Components Evaluation”. The SWRF lies within the natural floodplain of Coal Creek, however the site has been built up to bring all process buildings and tanks above the 100 year floodplain elevation. The geotechnical report for the site is included in Appendix C.



RMC
ROCKY MOUNTAIN CONSULTANTS, INC.
CIVIL AND ENVIRONMENTAL ENGINEERING • PLANNING
8301 E. Prentice Ave.
Suite 101
Englewood, CO 80111
(303) 741-6000
FAX (303) 741-6106

REVISIONS

NO.		DATE	BY
1	ADDENDUM #3	2/98	JNY

TOWN OF ERIE, COLORADO
0.6 MGDWASTEWATER TREATMENT FACILITY
SITE GRADING/GEOMETRIC CONTROL PLAN

DATE: SEPT. '97
DESIGNED BY: MCK
DRAWN BY: FV
CHECKED BY: MCK

JOB NO. 0885.011.03

SHEET 4 OF 40

Figure 5-8 South Water Reclamation Facility Site Layout

SECTION 6
COLLECTION SYSTEM

Within the Town of Erie's study area, most of the existing wastewater collection system flows by gravity either from south to north, or west to east, to the existing Water Reclamation Facility (WRF). There is one lift station that serves Kenosha Farms. Land elevations in the service area vary from 5,250 feet on the south end, to 4,950 at the north border; a drop of 300 feet. The collection system contains 107,109 linear feet or 20.29 miles of pipes with 10-inch diameters and larger, and 336,040 linear feet or 63.64 miles of pipes with 8-inch diameters and smaller. The collection system contains 2,199 known manholes. The existing collection system is illustrated in Figure 6-1.

The collection systems was previously modeled with H2OMap Sewer (MWHSoft, GIS Professional Suite 8.0, Update #4) in 2007 (Black and Veatch April 2007). An addendum to the 2007 Water and Wastewater Masterplan was issued in 2008. The addendum discussed an alternative alignment for portions of the Highway 52 and Interstate interceptors that would eliminate the need to construct lift stations. The primary reason for reevaluating the collection system was to update the hydraulic model by adding pipes, manholes, and interceptors that were installed after 2007. A secondary goal was to review the assumptions made during the first modeling effort and to confirm previous recommendations for upgrading and expanding the collection system.

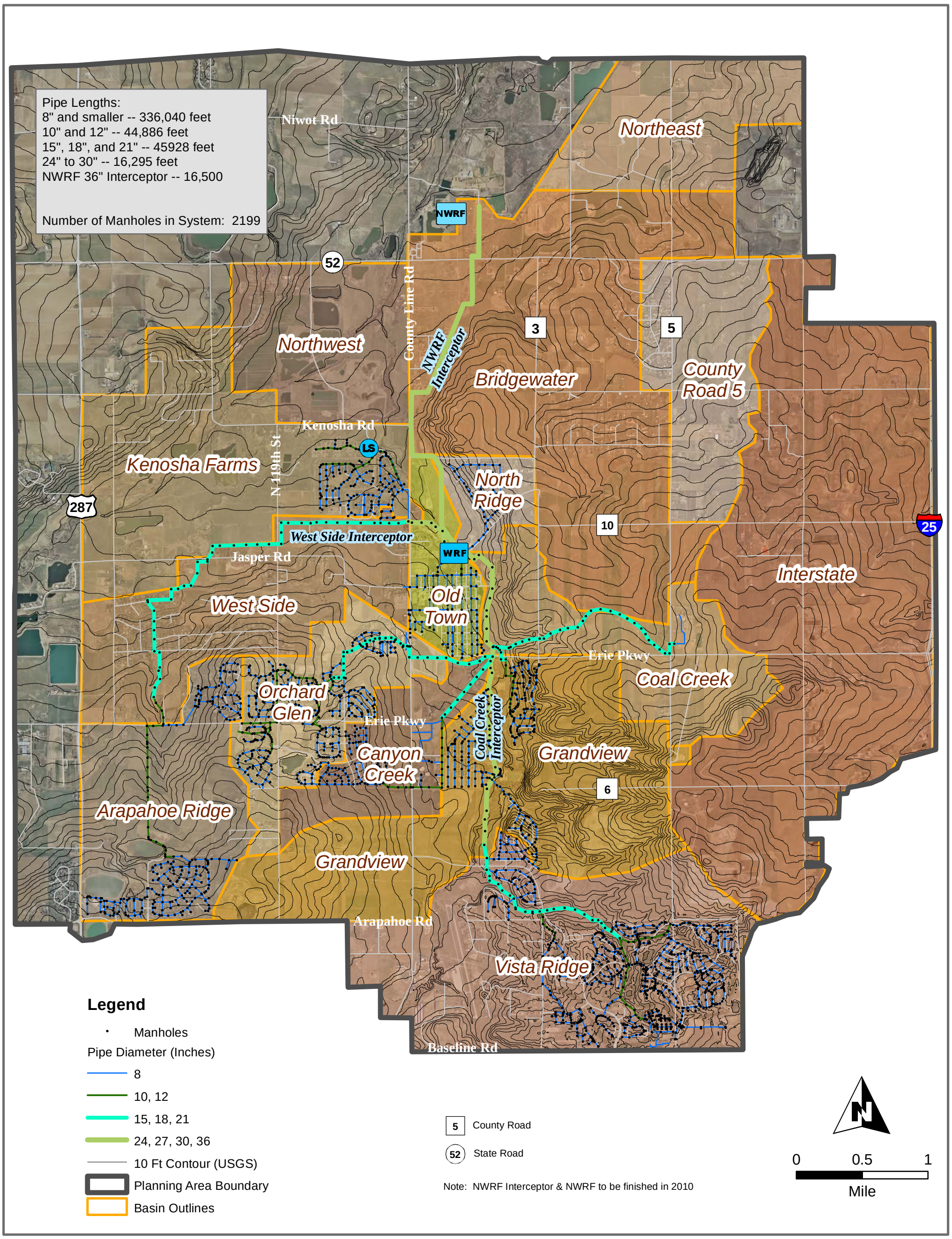
DESCRIPTION OF EXISTING COLLECTION SYSTEM

Drainage Basins

The wastewater collection system evaluation began by identifying the flow tributary areas or drainage basins. The drainage basins previously defined by Black and Veatch in 2006 were verified using a digital elevation model (DEM) provided by the Town. Fifteen (15) drainage basins were identified and are illustrated in Figure 6-1.

The existing sewer collection system serves ten (10) of the fifteen (15) drainage basins, which are:

- Old Town
- Canyon Creek
- Arapahoe Ridge
- West Side
- Coal Creek
- Vista Ridge
- Grandview
- North Ridge
- Orchard Glen
- Kenosha Farms



Existing Collection System with Contours and Drainage Basins
Figure 6-1

The remaining five drainage basins are located in the northern part of the service area. These drainage basins are not developed and therefore do not currently generate wastewater flow. The five undeveloped basins are:

- Bridgewater
- Northeast
- Northwest
- County Road 5
- Interstate

Figure 6-2 is a simplified graphic showing the relationships between different drainage basins and interceptors.

Manhole Naming Convention

The collection system contains 2,199 known manholes. The existing collection system is illustrated in Figure 6-1. Manhole ID numbers are alpha-numeric with the letters indicating the subdivision where the manhole is located. Manholes are then numbered by the collections department by the order that they are cleaned and maintained. For this reason, manhole numbers are not always sequential down a particular street. For new subdivisions and developments, the manhole IDs from the record drawings that were assigned by the developer or engineer are used until the collections department performs maintenance on those pipes and manholes for the first time. At that time, the collections department renames the manholes in accordance with the naming system described above.

Major Interceptors

The existing wastewater collection system serves the majority of the homes within the Town of Erie planning area, with the exception of a few low-density development areas served by septic systems. These areas are Baxter Farm and Erie Air Park. Baxter Farm and Erie Air Park are expected to connect to the sanitary sewer when it becomes both technically and economically feasible. The Brownsville Water and Sanitation District is not part of Erie's service area.

There are three major interceptors within the existing sewer collection system; West Side, Coal Creek, and the NWRF interceptor. The West Side Interceptor consists of gravity pipelines from 10 to 24 inches in diameter, running from the southwestern corner of the collection system or Arapahoe Ridge, north to Weld County Road 10, then running from west to east, ending at the SWRF. The Town of Erie constructed the West Side Interceptor to serve growth in Boulder County.

The Coal Creek Interceptor is located along Coal Creek, extending from the SWRF south towards the Grandview and Vista Ridge basins. The interceptor has three major extensions; one leading west into the Orchard Glenn basin, one leading southwest into the Canyon Creek basin, and one leading east into the Coal Creek basin. All three extensions connect to the primary interceptor just southeast of the Old Town basin. The Coal Creek interceptor and its extensions are visible on Figure 6-1. The pipelines are gravity lines ranging in size from 10 to 30 inches in diameter.

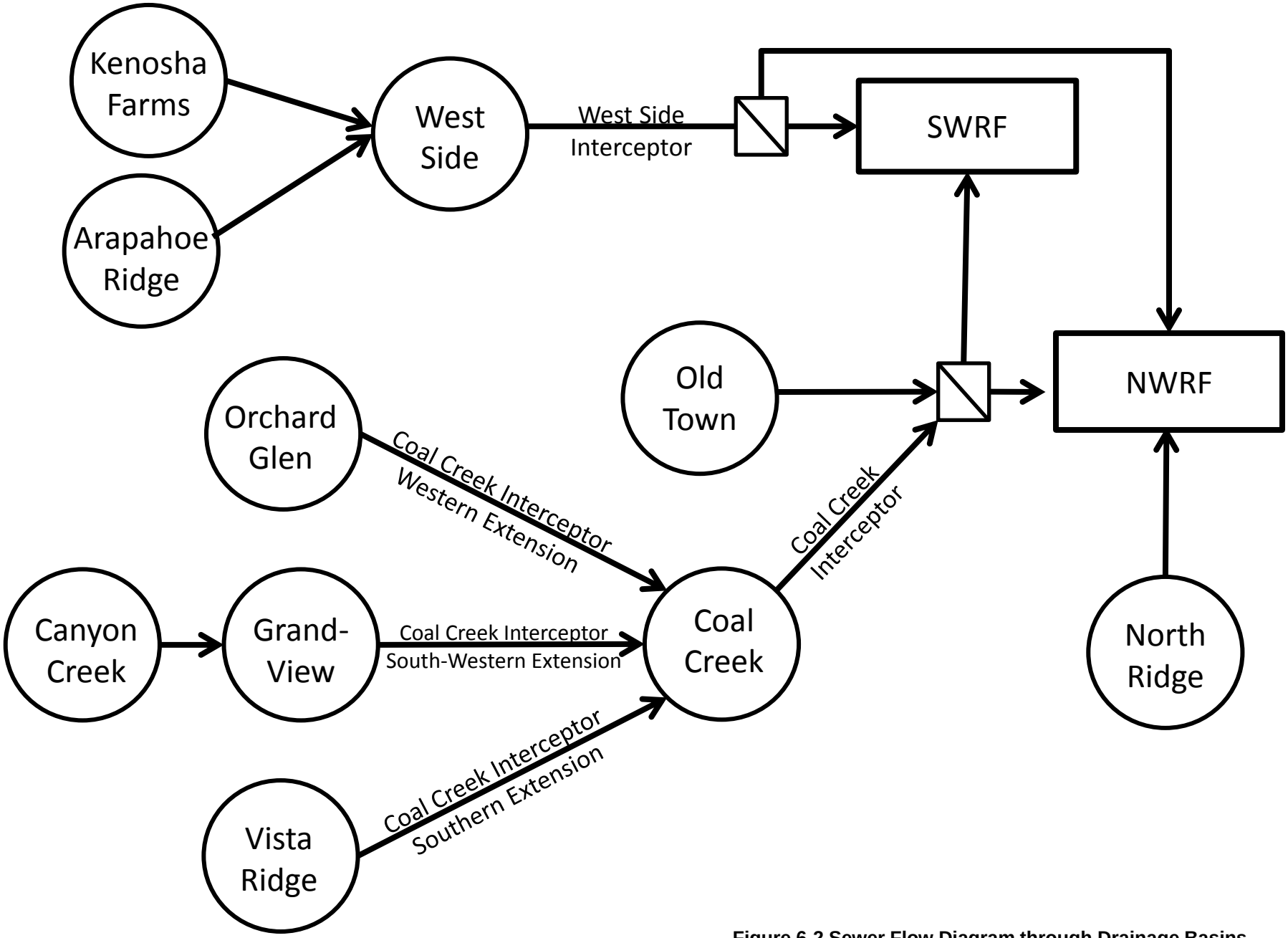


Figure 6-2 Sewer Flow Diagram through Drainage Basins

The NWRF interceptor begins at the SWRF and heads north to the new NWRF. The NWRF interceptor is 36-inches in diameter. Construction of this interceptor was completed in 2010. The NWRF interceptor includes two diversion structures located near the SWRF. The diversion structures allow plant staff to control the volume of flow received at the SWRF. Excess flows are automatically diverted to the NWRF for treatment.

Diversion Structures

The collection system contains two diversion structures located near the SWRF. The diversion structures allow plant staff to control the amount of flow received at the SWRF. Excess flows are automatically diverted to the NWRF. The location of each diversion structure is shown in Figure 6-3. The West Side interceptor diversion structure is located at the top of Figure 6-3 and roughly north of the SWRF. The Coal Creek and Old Town diversion structure is located at the bottom of Figure 6-3 and roughly east of the SWRF. Both diversion structures were constructed in 2010 as part of the NWRF interceptor project. In Figure 6-3, previously existing pipelines are shown in dark blue and the new pipelines constructed along with the diversion structures are shown in turquoise.

The West Side interceptor diversion structure is shown in Figure 6-4. This figure was taken from the construction drawing set issued by Burns and McDonnell in September 2009 as part of the NWRF interceptor construction project. The diversion structure was built within the previously existing 24-inch West Side interceptor line and includes a "Y" structure that splits the influent line into two, 24-inch pipelines. It can be used to direct the flow from the Arapahoe Ridge, West Side, and Kenosha Farms basins to either the SWRF, the NWRF, or a combination of both facilities. Two, manually operated canal gates allow plant staff to open one or both of the 24-inch gates located in each effluent pipeline. With the NWRF gate closed, all flow continues to flow to the SWRF.

The Coal Creek and Old Town diversion structure is shown in Figure 6-5. This figure was taken from the construction drawing set issued by Burns and McDonnell in September 2009 as part of the NWRF interceptor construction project. The interceptor is located within the existing Coal Creek interceptor immediately before it enters the SWRF. The diversion structure contains two influent lines in addition to the Coal Creek interceptor: an 18-inch sanitary waste loading line and the 24-inch Old Town interceptor. The sanitary waste loading line conveys sanitary wastewater from the SWRF to the diversion structure. An existing 8-inch biosolids line at the SWRF could be connected to the 24-inch Old Town interceptor immediately upstream of the diversion structure. In Figure 6-5, this influent line is identified as "proposed 24-inch biosolids and Old Town interceptor".

The Coal Creek and Old Town diversion structure contains a manually operated slide gate that divides the structure diagonally through its center. When the diagonal gate is in the fully closed position, all of the flow from the 30-inch Coal Creek interceptor will be diverted to the NWRF. At the same time, flows received from the Old Town interceptor as well as from the sanitary waste loading and biosolids lines will be conveyed to the SWRF. When the diagonal gate is in the open or partially open position, flows can be sent to either the SWRF and NWRF or be split between them. Each of the diversion structure effluent lines is equipped with a manually operated slide gate. These gates may be used to divert one-hundred

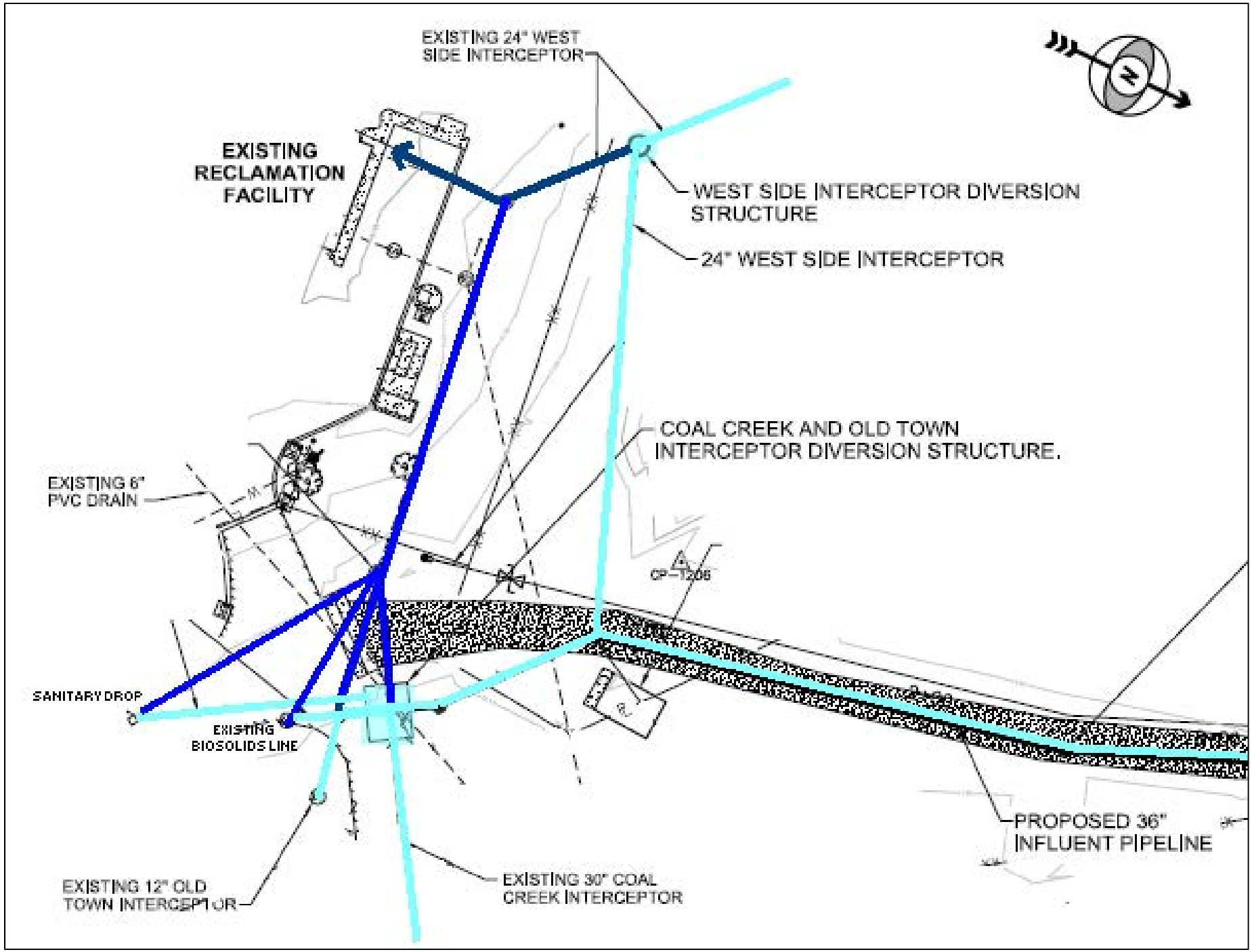
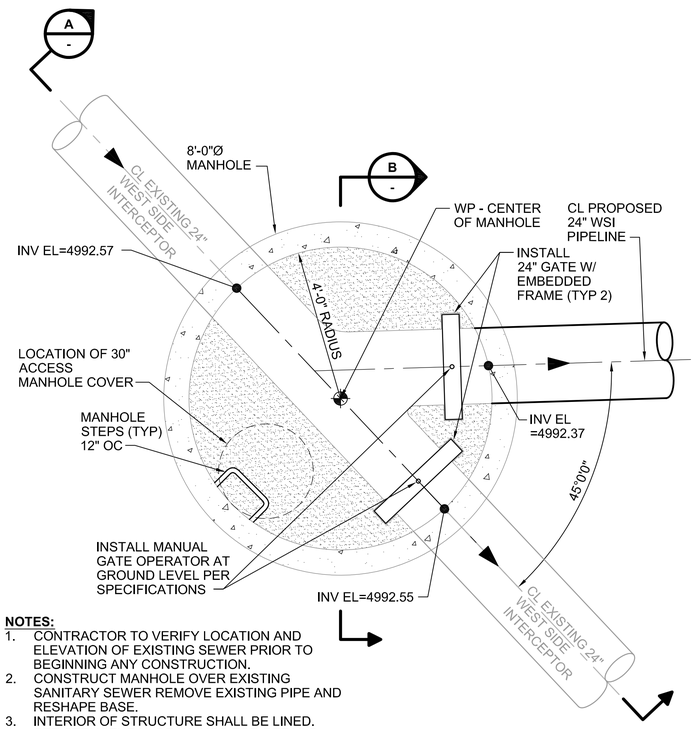


Figure 6-3 Diversion Structure Locations

no.	date	by	ckd	description
0	05/08/09	KR	DDK	ISSUED FOR BID
	09/03/09	KR		ISSUED FOR CONSTRUCTION



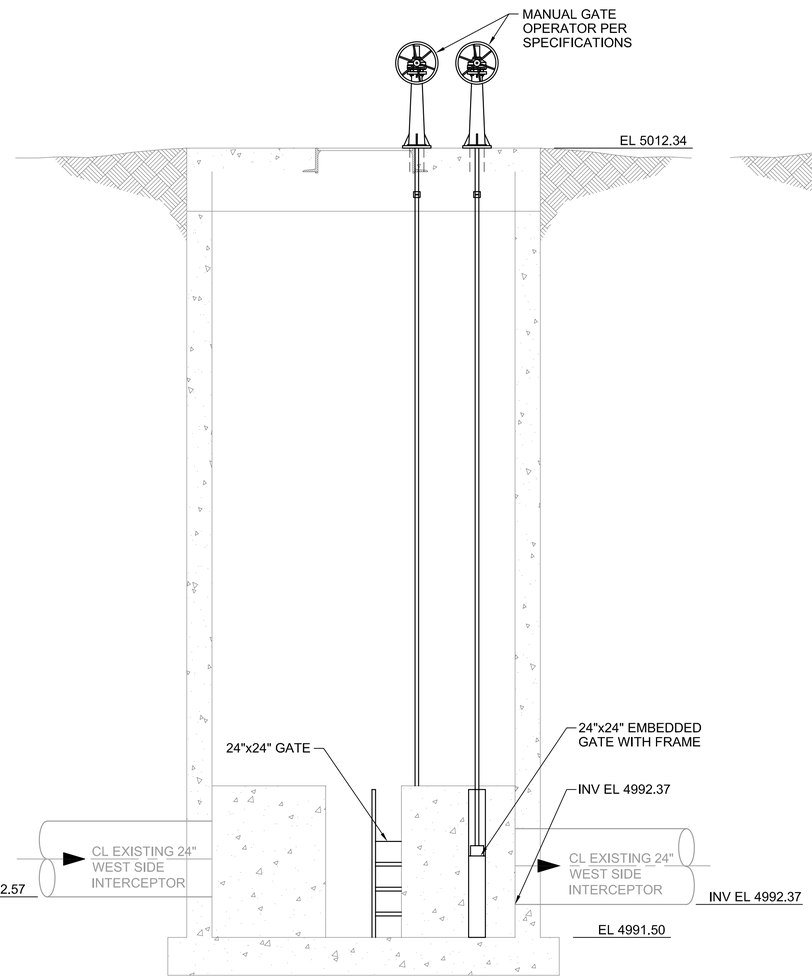
NOTES:

1. CONTRACTOR TO VERIFY LOCATION AND ELEVATION OF EXISTING SEWER PRIOR TO BEGINNING ANY CONSTRUCTION.
2. CONSTRUCT MANHOLE OVER EXISTING SANITARY SEWER REMOVE EXISTING PIPE AND RESHAPE BASE.
3. INTERIOR OF STRUCTURE SHALL BE LINED.
4. SEE STRUCTURAL DRAWINGS FOR DETAILS.

WEST SIDE INTERCEPTOR DIVERSION STRUCTURE

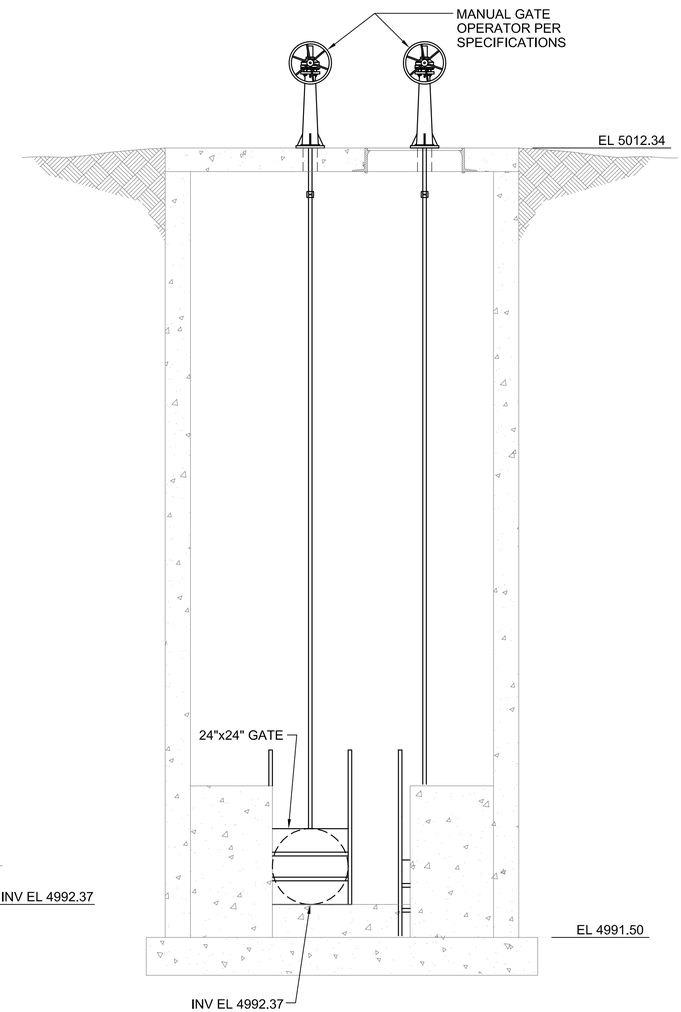
0 1' 2' 4'
SCALE IN FEET

NORTH



SECTION A -

0 1' 2' 4'
SCALE IN FEET



SECTION B -

0 1' 2' 4'
SCALE IN FEET

Burns & McDonnell
SINCE 1898

date	detailed
SEPTEMBER, 2009	P. HUNTZINGER
designed	checked
K. REICHSTEIN	D. KORINEK

TOWN OF ERIE
1874

NORTH WATER RECLAMATION FACILITY
INFLUENT PIPELINE AND REUSE WATERLINE
WEST SIDE INTERCEPTOR
EQUIPMENT PLAN

project	contract
40860	

drawing **PP050** – rev. **0**

Figure 6-4 West Side Interceptor Diversion Structure

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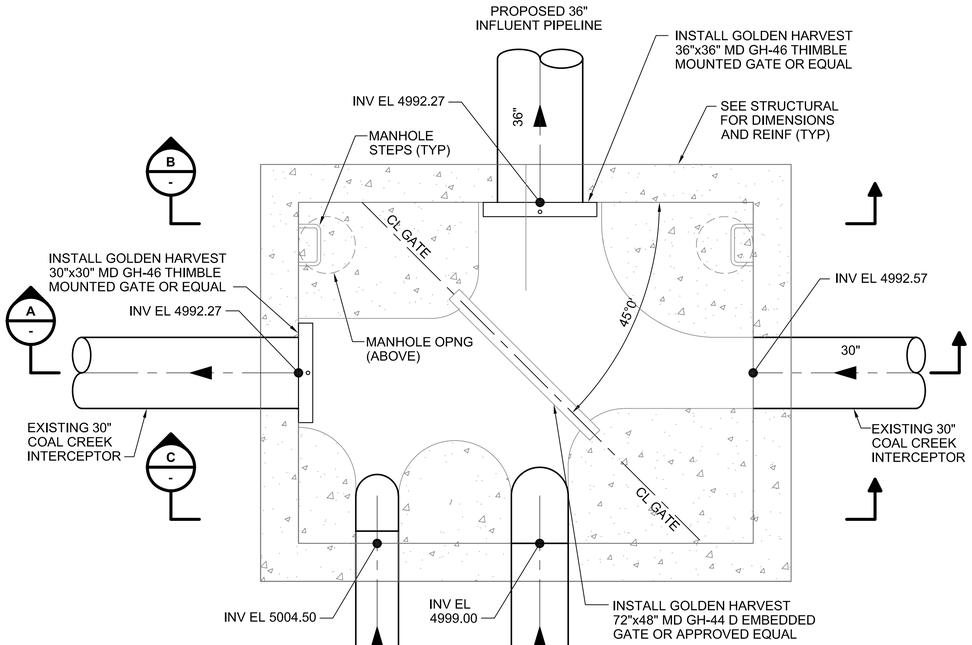


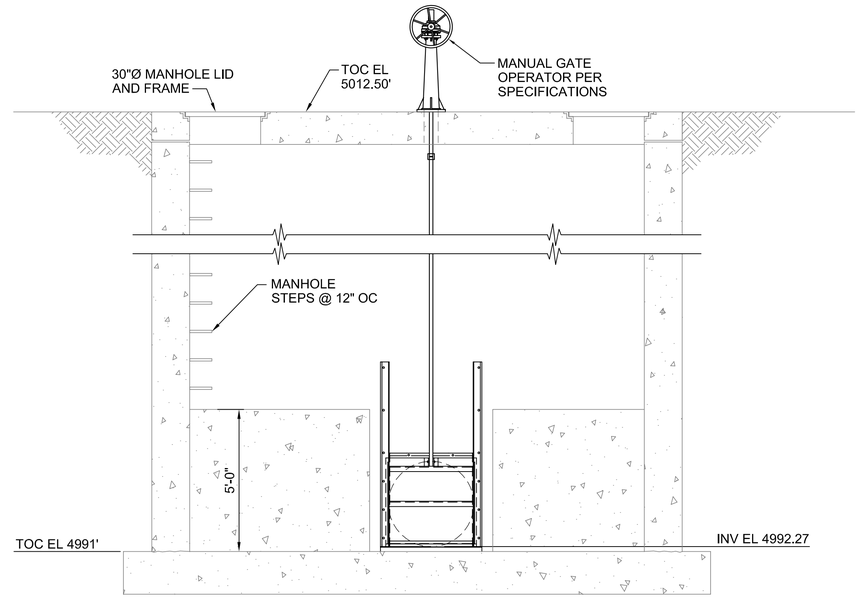
NOTES:

1. CONSTRUCT OVER THE COAL CREEK INTERCEPTOR. CONFIRM DEPTH AND LOCATION PRIOR TO BEGINNING CONSTRUCTION.
2. INTERIOR OF STRUCTURE SHALL BE LINED.
3. EXISTING COAL CREEK INTERCEPTOR PIPE SHALL BE REMOVED FROM THE INSIDE OF THE STRUCTURE.

FOUNDATION PLAN

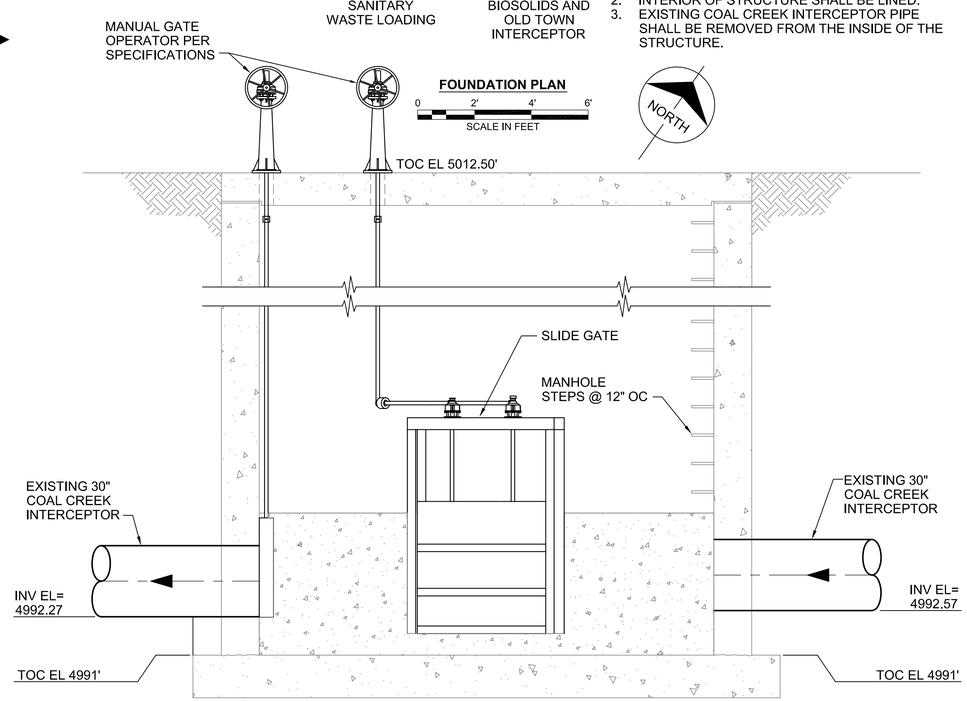
0 2' 4' 6'
SCALE IN FEET

NORTH



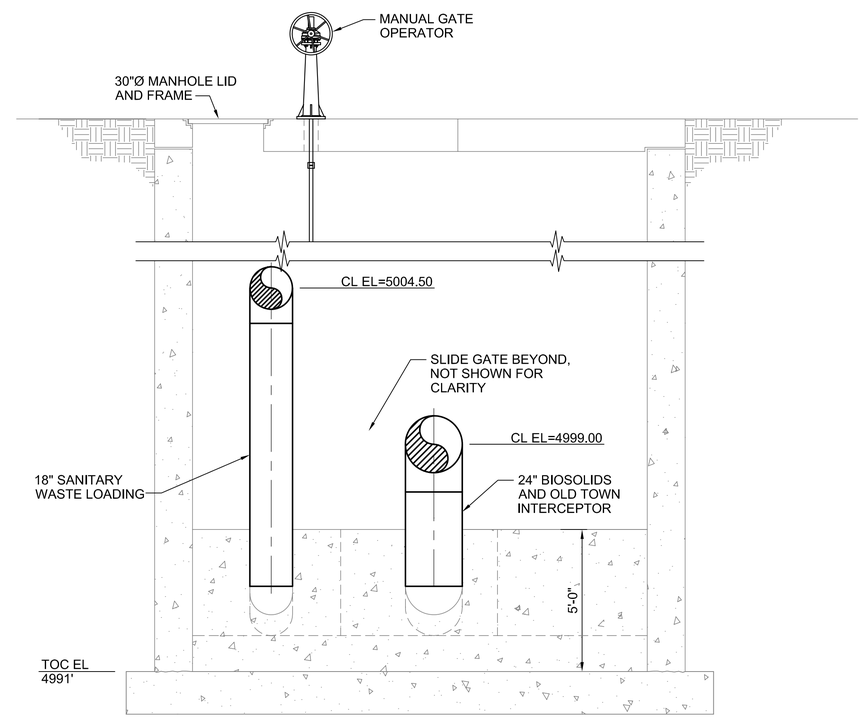
SECTION B

0 2' 4' 6'
SCALE IN FEET



SECTION A

0 2' 4' 6'
SCALE IN FEET



SECTION C

0 2' 4' 6'
SCALE IN FEET

no.	date	by	ckd	description
0	05/08/09	KR	DDK	ISSUED FOR BID
	09/03/09	KR		ISSUED FOR CONSTRUCTION

Burns & McDonnell
SINCE 1898

date	detailed
SEPTEMBER, 2009	J. EVANS
designed	checked
K. REICHSTEIN	D. KORINEK

TOWN OF ERIE
1874

NORTH WATER RECLAMATION FACILITY
INFLUENT PIPELINE AND REUSE WATERLINE
COAL CREEK AND OLD TOWN DIVERSION STRUCTURE PLANS AND SECTIONS

project	contract
40860	

drawing **PP049** – rev. **0**

Figure 6-5 Coal Creek and Old Town Interceptors Diversion Structure

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Scale For Microfilming
Millimeters
Inches

percent of the flows entering the diversion structure to either the SWRF or NWRF when the diagonal slide gate is in the full open position.

Neither diversion structure contains any flow measurement devices.

MODEL CONSTRUCTION

Geometric Network

The Town's existing collection system model includes approximately 99,000 linear feet of sewer line 8-inch-diameter or greater. It was initially developed as part of the 2006 Water and Wastewater Master Plan (Black and Veatch April 2007). Portions of the Plan are included in Appendix H along with an Addenda issued in 2008.

The existing model was updated by importing it into InfoSWMM (MWHSoft, GIS Professional Suite 10.0, Update #1) and adding pipes and manholes constructed after the model was completed in 2006. The collection system was previously modeled with H2OMap Sewer (MWHSoft, GIS Professional Suite 8.0, Update #4). Both modeling software packages are made by the same company and are compatible with one another. InfoSWMM includes additional modeling capabilities such as stormwater modeling. The final model was converted back into the H2OMap Sewer format at the Town’s request.

The collection system was evaluated using the InfoSWMM computer model. The model contains pipes 10-inch and greater as well as their associated manholes. Some 8-inch lines were also included due to their importance. Modeling was used to determine the ability of the collection system to convey current and future wastewater flows under average day, maximum month, peak day, and peak hour conditions. The modeling effort consisted of verifying manhole rim elevations and pipe inverts based on GIS data, previous modeling data, record drawings and other data sources, allocation of wastewater flows to individual manholes, application of appropriate peaking factors, and development of evaluation criteria.

For modeling results to be meaningful, model inputs must be representative of actual field conditions. The model was constructed using the Town’s GIS data to update the 2006 model created by Black and Veatch. After importing both GIS and the 2006 model, data discrepancies were found. The Town’s GIS sewer manhole layer was missing the elevation attributes for manholes, therefore the 2006 model data for starting and ending nodes were used to match to the starting and ending nodal IDs for the pipes in the model update. Further field verification is recommended to confirm manhole rim and invert data.

The 2006 model was built as a simplified hydraulic network including pipes 8-inch diameter and larger; while the Town’s GIS data layer covers all the existing wastewater collection system components. The Town should consider including all of the collection system components in future models. The Town is in the process of collecting GIS location, rim elevation, and invert elevation data for each manhole in the collection system. When this work is completed, an all-pipe model based on the GIS dataset can be constructed. The advantage of including all pipes in the model is the ability to create a one-to-one, linked relationship between the hydraulic model and the GIS dataset. As future sewer lines are constructed, the model can then be easily updated by importing the GIS data.

Several capital improvement projects (CIPs) identified in the 2006 Water and Wastewater Master Plan are under construction. The model was updated using drawings issued for construction including:

- Inclusion of the diversion structures.
- Addition of the NWRF interceptor.
- Addition of the gravity sewer bypass from the Kenosha Farms Lift Station to the NWRF interceptor.
- Setting the Kenosha Farms Lift Station (KFLS) offline.
- Updating the outfall elevation at the NWRF.
- Adding additional pipe from the GIS dataset.

Wastewater Flow Allocation

Influent flows and loads are discussed in detail in Section 4. For convenience, the flows selected for planning purposes are summarized in Tables 6-1 and 6-2 below. For planning purposes, the years 2025 and buildout were selected.

Table 6-2 lists projected future average and maximum month flows based on population projections. Average daily flows use a per capita generation rate of 90 gallons per person per day. Maximum month flows use a per capita generation rate of 118 gallons per person per day.

Table 6-1 Selected Future Flows for Planning Purposes

Parameter	2025	Buildout
Annual Average Flow Generation Rate, gpcd	90	90
Maximum Month Flow Generation Rate, gpcd	118	118
Annual Average Daily Flow, mgd	3.70	6.26
Maximum Month Flow, mgd (MM PF = 1.3)	4.80	8.12
Peak Day Flow, mgd (PD PF = 1.7)	6.30	10.66
Maximum Month Flow to SWRF, mgd	1.60	1.60
Annual Average Flow to SWRF, mgd	1.23	1.23
Peak Hour Peaking Factor for SWRF	3.52	3.52
Peak Hour Flow to SWRF, mgd	4.35	4.35
Maximum Month Flow to NWRF, mgd	3.20	6.52
Annual Average Flow to NWRF, mgd	2.46	5.02
Peak Hour Peaking Factor for NWRF	3.14	2.79
Peak Hour Flow to NWRF, mgd	7.74	14.02

MM PF = Maximum Month Peaking Factor, PD PF = Peak Day Peaking Factor

Table 6-2 Projected Future Average and Maximum Month Flows

Date	Population	ADF	MM
2010	18,135	1.63	2.14
2015	26,525	2.39	3.13
2020	33,525	3.02	3.96
2025	40,680	3.66	4.80
2030	49,625	4.47	5.86
Buildout	68,820	6.19	8.12

Criteria employed in the future sewer loading were determined based on the historical flow data. Historically, the Town of Erie has been a "bedroom" community with most residents commuting to surrounding cities such as Denver, Longmont, and Boulder for work. Based on the water usage data, approximately 85% of the wastewater flow is created by residential customers, while the remainder is contributed by the non-residential users such as commercial, business, schools, libraries, and public institutions. The most current version of the Town's land use map was used to assign wastewater loads to each parcel depending on land use type. Wastewater flows were allocated based on estimates of population, population distribution, and expected rates of wastewater generation for each land use. Typical wastewater generation rates were used to estimate wastewater flows for each land use within the District. The criteria used for the wastewater flow projections were:

- Business — 10 People per acre
- Community commercial — 10 people per acre
- Downtown district — 12 people per acre
- Industrial — 13 people per acre
- Mixed use for commercial — 8 people per acre
- Neighborhood commercial — 10 people per acre
- Public — 5 people per acre
- Regional commercial — 10 people per acre

For a detailed explanation of how population and flows were allocated to individual land parcels, refer to the section under *Future Population Growth* starting on page 4-6 of this utility plan. Flow allocation was done in five year increments from 2010 to 2025 and for the buildout condition. After flows had been assigned to individual land parcels, the flows for those parcels were assigned to individual manholes within the model. Manhole assignments were made based on geographic proximity and topography.

Flows from individual land use parcels were assigned to specific manholes based on topography and location of existing collection pipes. Allocating flows to individual manholes that are most likely to receive this flow provides a realistic prediction of capacities and the potential for surcharging gravity lines. All sewer loadings, such as AAFs, MM and peak hour flows were allocated to the hydraulic model using a feature in InfoSWMM called GIS-Gateway. Flow monitoring was not done as part of this study, so no information was available for model calibration. Estimated MM flows, which include infiltration, were used as the base flow and peak flow was calculated using the DRCOG and NFRWQPA peaking factor. The model revises the peaking factor based on flow at each segment of the collection system, so pipes further out in the collection system experience a much higher peak than those near the SWRF or NWRF.

Peak Hour Flows

Peak hour peaking factors are discussed in detail in Section 4. For convenience, the peak hour peaking factors utilized in the collection system model are summarized in Table 6-3 below. Peaking factors differ between the SWRF and the NWRF based on the annual average flow received by each facility. The SWRF will operate as a base-load facility with excess flows being diverted to the NWRF. Peak hour peaking factors range between 3.45 and 3.73 for the SWRF and between 2.90 and 3.49 for the NWRF. These peak hour peaking factors were estimated using the DRCOG formula. Actual peak hour peaking factors may be much lower, especially at the NWRF.

Table 6-3 Estimated Peak Hour Peaking Factors and Peak Hour Flows by Facility

SWRF			NWRF		
MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD	MMF, MGD	Peak Hour PF (DRCOG)	Peak Hour Flow, MGD
1	3.73	3.73	1.5	3.49	5.23
1.2	3.62	4.34	3.6	3.01	10.84
1.6	3.45	5.52	3.9	2.90	11.34

Note: MMF is the average daily flow during the maximum month. PF is peaking factor. Peak hour peaking factors are calculated using the annual average flow. The maximum month peaking factor is 1.14

Peaking factors vary according the total flow received. Smaller flows generally correlate to smaller service areas while larger flows correlate to larger service areas. As the service area gets bigger and the collection system gets longer, more attenuation occurs which results in lower peak hour peaking factors. In essence, the peaks and valleys are smoothed as flows are allowed to spread out through the collection system. This is partly due to the tendency of larger service areas to incorporate shift workers and commercial accounts. Businesses and shift workers will generate wastewater at different times of day than a typical single-family residence or apartment building.

Peak hour peaking factors were modeled for each treatment facility independently. Two different methods were utilized to assign different peak hour peaking factors to each WRF. In the first method, each drainage basin was assigned to either the SWRF or the NWRF. Drainage basins assigned to the SWRF included Orchard Glen, Canyon Creek, Grandview, Vista Ridge, Coal Creek, North Ridge, Old Town, and Arapahoe Ridge. The remaining basins were assigned to the NWRF. Then, the peak hour peaking factors from Table 6-3 were assigned to each drainage basin. This is done in the model by creating domains or groups of pipes and manholes. Each domain can then be assigned its own peak hour peaking factor and/or diurnal curve. The model was run under current conditions, 2025, and buildout.

Because the diversion structures will send flows combined from multiple basins to the NWRF, it is difficult to estimate which basin's flows will actually end up at each facility. As a secondary check, the model was also run using the same peak hour peaking factor for the entire service area. This was done multiple times using first the SWRF peak hour peaking factors and then using the NWRF peak hour peaking factors.

Diversion Structure Modeling

The diversion structures were modeled as overflow manholes. The amount of flow diverted to either the SWRF or NWRF was then adjusted by raising or lowering the invert or weir elevation of the overflow pipe within the model.

Diurnal Curve for EPS Modeling and Peak Hour Factors

Peak hour factors are critical for analysis of the collection system, lift stations, and some portions of the treatment plant. If peak hour flows cannot be processed efficiently, wastewater may back up in the treatment plant and collection system, potentially resulting in overflows. In order to better simulate a sewer collection's backup effects from a downstream boundary or from hydraulic structures, InfoSWMM's Extended Period Simulation (EPS) feature is used. EPS is a dynamic wastewater model which applies a series of peak hour peaking factors, taking into account the variability of flow throughout the day. As part of the simulation, a diurnal demand curve was developed, which is a series of time variations in sanitary flow that better matches actual flow conditions.

When flow data for a town or city is available, a diurnal curve can be created using the actual data. Because the SWRF has an influent lift station prior to flow monitoring, diurnal flow data was not available. Two days of diurnal data was available from a previous planning study (Black and Veatch April 2007). This diurnal data was not considered to be representative because the location where the data was collected was not identified

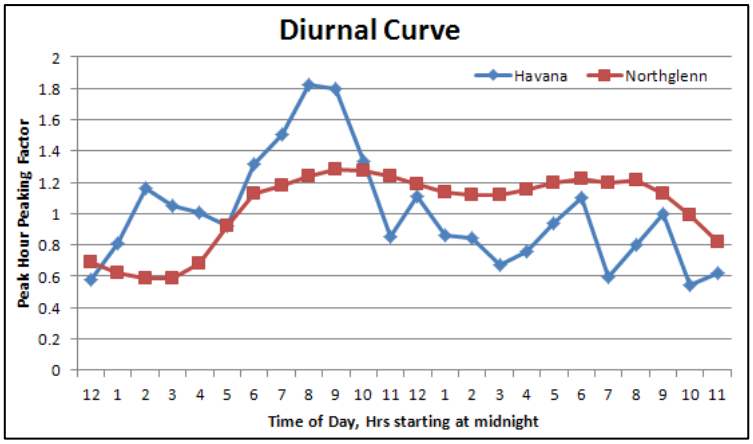


Figure 6-6 Diurnal Curve

in the report and because the data was collected on a week-end. Diurnal data from the Town of Havana, Illinois was selected as a surrogate. Like Erie, Havana is a bedroom community with no industry and few commercial sewer accounts. The diurnal curve used for the model is displayed in Figure 6-6. For comparison, a diurnal curve for the City of Northglenn, population 33,250 is also shown. Northglenn's service area contains a variety of both commercial and industrial accounts.

MODEL EVALUATION RESULTS

The wastewater system was evaluated to identify possible capacity issues such as surcharged sewers and manholes during the annual average flow (AAF), maximum month flow (MM), and peak hour flow conditions. Recommendations were made regarding the size and potential alignment of future interceptors.

Evaluation Criteria

Using the evaluation criteria presented in Table 6-4, the existing and future pipelines were evaluated under extended period simulation (EPS) conditions for AAF, MM, and peak hour flows starting with 2010 through Build-out. Two main parameters were evaluated: flow velocity and depth to diameter (d/D) ratios at AAF, MM and peak hour. Gravity sewers need maximum flow velocities greater than two feet per second (2 fps) at peak hour flow to prevent solids from settling out and accumulating within the pipes. Solids that settle during low flow periods of the day may be resuspended during peak hour flows. Flow velocities greater than 10 fps are undesirable as they may scour, abrade, and erode pipes. The depth to diameter (d/D) ratio, which is the ratio of water depth in the pipe divided by the pipe diameter, represents the percentage of capacity in use. Generally, pipes that are 15-inches in diameter and larger should have a d/D below 0.8 at the maximum monthly flow to ensure peak hour flows can be accommodated. Pipes smaller than 15-inches in diameter should have a d/D ration below 0.5 at the maximum monthly flow.

Table 6-4 Evaluation Criteria for Wastewater Hydraulic Modeling

Parameters	Evaluation Standard	Parameters	Evaluation Standard
Gravity Sewer		Force Mains	
Hydraulic Capacity	Based on Manning's equation	Minimum velocity, fps	2.0
Manning's n friction coefficient	0.013 for all pipe	Maximum velocity, fps at MM	6.5
Minimum velocity, fps	2.0	Pump Stations	Based on peak hour wastewater flow with largest pump out of service (firm capacity)
Maximum velocity, fps	10.0		
Maximum Depth of flow ratio, d/D at MM	0.8 for existing pipes and for proposed pipes greater than 15-inches in diameter. 0.5 for pipes less than 15-inches in diameter		
Depth of flow ratio at AAF	0.4		

A pipe with a d/D of 0.8 is flagged for potential capacity improvements such as increasing pipe slope, proposed paralleling, or replacing with larger diameter pipe. In addition, d/D ratios greater than 0.8

indicate places where surcharging is likely and/or where the carrying capacity of the pipe has been exceeded.

Surcharging occurs when sewers exceed 100% of their capacity. Surcharging causes water to backup into the pipe and manholes. The hydraulic gradeline may be several feet above the top of the pipe. As a result, surcharged pipes are subjected to hydrostatic pressure. Surcharging is caused by excessive peak flows or when pipe slopes are too flat or positive. Surcharged sewers raise the upstream hydraulic grade line and may cause sewage to backup into resident's basements or to overflow manholes onto the ground surface. High hydrostatic pressure may also cause joints to leak and can damage pipes.

Existing System Evaluation Results

The existing wastewater collection system (2010) was modeled at annual average flow (AAF), maximum month (MM), and peak hour conditions. No surcharging was identified. The results of the existing system modeling are presented in Table 6-5.

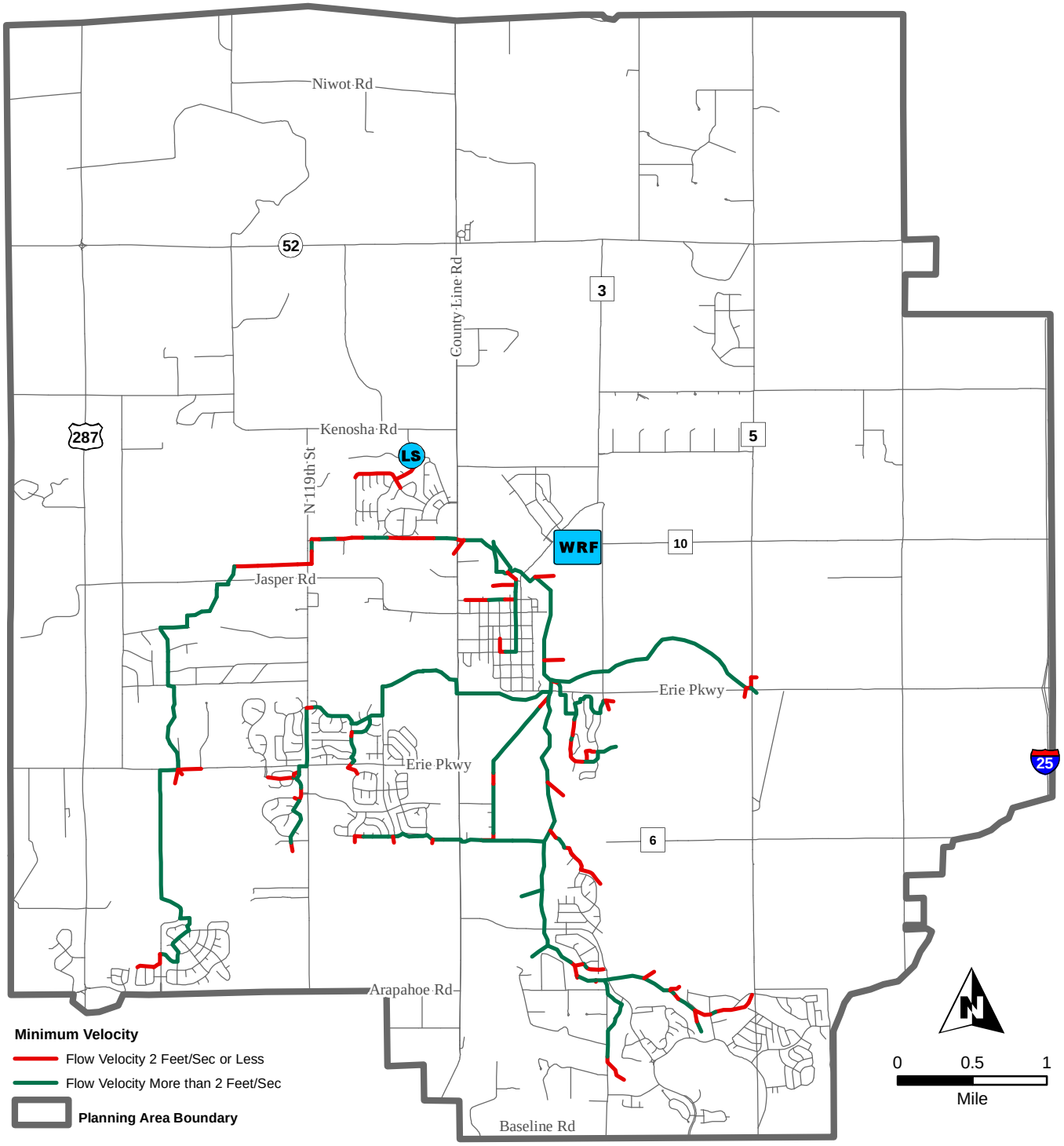
Table 6-5 Existing / 2010 Wastewater Hydraulic Modeling Results

Parameter	Minimum	Maximum	Standard
Flow Velocity at 2010 AAF, fps	0.007	5.229	>2.0
Flow Velocity at 2010 MM, fps	0.007	5.421	<10.0
Flow Velocity at 2010 Peak Hour Flow, fps	0.023	6.914	<10.0
Depth of Flow Ratio at 2010 AAF, d/D	0.001	0.639	<0.4
Depth of Flow Ratio at 2010 MM, d/D	0.001	0.645	<0.8 (for >15"pipe) <0.5 (for <15" pipe)
Depth of Flow Ratio at 2010 Peak Hour Flow, d/D	0.005	0.692	No surcharging

Modeling indicates that under current conditions, the existing collection system is under 70% capacity. Flow velocities at peak hour range between 0.023 and 6.914 fps. Figure 6-7 shows the minimum flow velocities during peak hour. Pipes shown in red have maximum flow velocities below 2 fps which may result in solids accumulation. These areas include portions of the West Side Interceptor as well as the extremities of most of the major interceptors. Low flow velocities are common in residential areas because 8-inch pipe must be used to prevent blockages. Routine jetting of pipelines in residential areas will help maintain these low velocity pipelines. The Town currently cleans 25 percent of the collection system every year. Low flow velocities are also common when interceptors and laterals are upsized to accommodate future flow that may not manifest for many years. The West Side interceptor will eventually serve many more single-family residences and businesses than it does in 2010. As the Arapahoe Ridge, West Side, and Kenosha Farms drainage basins continue to develop, flow velocities will increase in the West Side interceptor. Until then, the interceptor may require more frequent cleaning.

Future Interceptors, Extensions, and Lift Stations

The existing collection system will need to be expanded to accommodate future flows generated within the existing service area and to extend service into currently undeveloped drainage basins. Recommendations for placement and sizing of future interceptors are based on topography, expected future land uses, population density, and per capita generation rates for the Town. This section summarizes the recommendations for new major interceptors and extensions that were outlined in the



Peak Hour Flow for 2010 - Minimum Velocity
Figure 6-7

2006 Water and Wastewater Master Plan and were confirmed by the current modeling effort. Small diameter interceptors and laterals can be constructed by developers; however, the Town may need to take an active role in the development, construction, and funding of major interceptors.

The 2006 Water and Wastewater Master Plan recommended four new interceptors, multiple extensions and laterals for existing and future interceptors and four lift stations. Those recommendations are summarized in Tables 6-6 through 6-9. A copy of the figure from the 2006 Master Plan that shows the locations of the recommended improvements is included as Figure 6-8 for reference. An addenda issued in 2008 rerouted the Highway 52 interceptor and Interstate interceptor. The second alternative eliminated the Highway 52 and Interstate lift stations, but substantially increased the bury depth of both interceptors; over 30 feet in some locations.

Table 6-6 Future Wastewater Collection System Interceptors from 2006 Master Plan

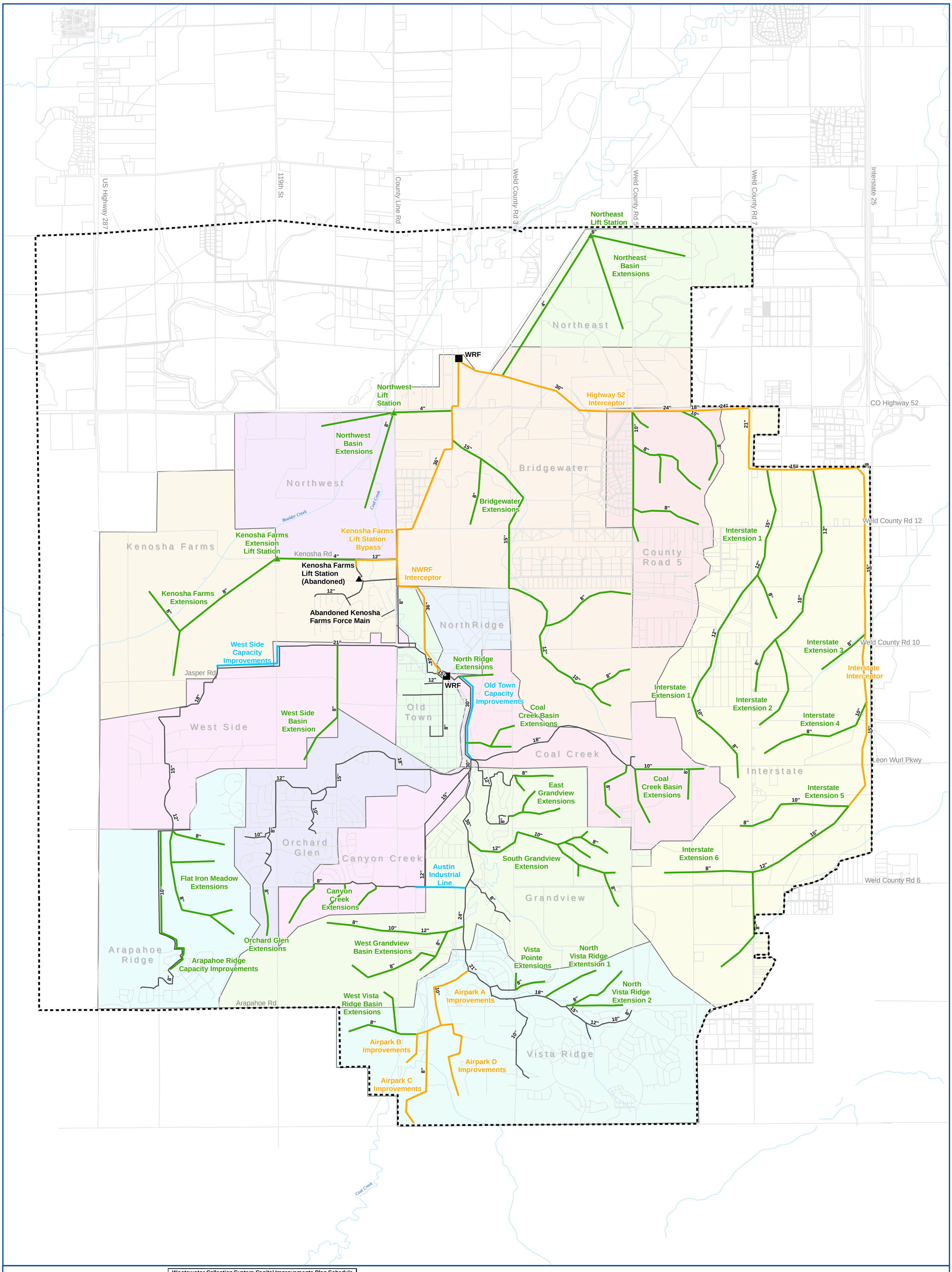
Name	Length, ft	Diameter Range, inches
Highway 52 Interceptor	13,400	24 to 30
Interstate Interceptor	22,800	15 to 24
Kenosha Farms Lift Station Bypass[1]	2,800	12
NWRF Interceptor[1]	16,500	36

[1]Construction completed in 2010.
[2]Source: (Black and Veatch April 2007), Lengths updated to reflect deep sewer alternative (Black and Veatch July 2008).

Table 6-7 Future Wastewater Collection System Extensions from 2006 Master Plan

Name	Length, ft	Diameter Range, inches
Bridgewater Extensions	25,500	8 to 15
Canyon Creek Extensions	4,800	8
Coal Creek Basin East Extensions	13,800	8 to 10
Count Road 5 Extensions	19,300	8 to 10
East Grandview Extensions	6,100	8
Interstate Extension 1	17,600	8 to 15
Interstate Extension 2	15,200	8 to 12
Interstate Extension 3	2,500	8
Interstate Extension 4	5,500	8 to 10
Interstate Extension 5	5,200	8 to 10
Interstate Extension 6	14,000	8 to 15
Kenosha Farms Extensions	9,600	8
Flat Iron Meadow Extensions	11,600	8 to 10
North Interstate Extension	5,058	8
North Vista Ridge Extension 1	2,600	8
North Vista Ridge Extension 2	5,400	8
Northeast Basin Extensions	8,700	8
North Ridge Extensions	5,200	8
Northwest Basin Extensions	7,700	8
Orchard Glen Extension	2,300	8
South Grandview Extension	14,200	8 to 12
Vista Pointe Extensions	2,400	8
West Grandview Basin Extensions	12,500	8 to 12
West Side Basin Extension	5,700	8
West Vista Ridge Basin Extensions	5,200	8

Source: (Black and Veatch April 2007). Lengths updated to reflect deep sewer alternative (Black and Veatch July 2008).



LEGEND

Lift Station

- Buildout Lift Station
- CIP Lift Station
- Existing Lift Station
- Treatment Plant

Model Sewer Lines

- CIP
- Capacity Improvements
- Extensions
- Existing
- Study Area Boundary

Wastewater Collection System Capital Improvements Plan Schedule

Name	Opinion of Capital Cost ($)	Project in Service	Trigger for Project Initiation
Short-Term (5-Year) CIP 2010 Projects			
Highway 52 Interceptor	4,420,000	2010	Development in northeast along Highway 52
NWRF Interceptor	8,300,000	2009	In progress
Kenosha Farms Lift Station Bypass	470,000	2009	In conjunction with NWRF Interceptor
Long-Term (Buildout) CIP Projects			
Interstate Interceptor	4,440,000	Buildout	Development along I-25
Arapahoe Ridge Capacity Improvements	1,200,000	Buildout	Additional development in basin
West Side Capacity Improvements	880,000	Buildout	Flat Iron Meadow Development
Old Town Capacity Improvements	1,386,000	Buildout	Exceed 10,000 units in drainage basin
Austin Industrial Line	302,000	Buildout	Canyon Creek Development

N
W E
S

0 2,000 4,000 Feet

1 inch equals 2,000 feet

BLACK & VEATCH
Building a world of difference.

Wastewater Collection System Recommended Improvements

2006 Master Plan Update

Figure 6-8

Revised: April 17, 2008

Table 6-8 Future Lift Stations from 2006 Master Plan

Name	Lift Station		Force Main	
	Wet Well Dimensions diameter/depth, ft	Firm Capacity GPM	Diameter inches	Length Feet
Highway 52 L.S.[1]	Two at 15 / 8 each	8,000	24	1,800
Interstate L.S.[1]	5 / 4	200	4	3,100
Northeast L.S.	4 / 2	10	4	7,400
Northwest L.S.	4 / 4	130	4	2,600

Source: (Black and Veatch April 2007). [1]The Highway 52 and Interstate lift stations are optional. Alternate alignments for the interceptor have been proposed that utilize deep sewers, in places more than 30 feet deep, to eliminate the need for lift stations.

The buildout scenario model was initially constructed with all of the new interceptors, extensions, and lift stations listed in Tables 6-6 through 6-8 in place. The placement of each interceptor and extension was visually inspected to verify that each one followed a natural drainage. When interceptors are placed to follow the lowest elevations of a drainage basin, the collection area is maximized and the potential need for lift stations is minimized. All of the proposed interceptors and extensions follow industry accepted practices for placement.

The model was then updated to reflect the alignments of the Highway 52 and Interstate interceptors proposed in the 2008 Addenda (Black and Veatch July 2008). The new alignments were reviewed to verify that they complied with industry accepted practices. Connecting laterals and interceptor inverts were then adjusted to match the updated interceptors.

Two small lift stations could not be eliminated: the Northeast lift station which will serve the Northeast drainage basin and the Northwest lift station which will serve the Northwest drainage basin. These lift stations are relatively small and will serve defined areas. Under the Town's policy of holding developers responsible for providing the necessary infrastructure to connect new developments to the collection system, both of these lift stations will likely be constructed and paid for by developers.

Wastewater flows were allocated to individual parcels and interceptors as previously described in Section 4 and on pages 6-5 through 6-7 of this Section.

Future System Evaluation Results

Future wastewater collection system modeling was performed based on the future sewer loadings starting from 2015 through 2025 and buildout under annual average flow (AAF), maximum month flow (MMF), and peak hour flow conditions. The 2006 Water and Wastewater Master Plan identified four areas at more than 100 percent of capacity during peak flows at buildout. These areas include:

- Sections in the Canyon Creek Drainage Basin
- Sections in the Arapahoe Ridge Drainage Basin
- Sections of the West Side Interceptor
- Sections of the Old Town Interceptor

Table 6-9 summarizes the improvements recommended in 2006 to increase capacity in these areas. The model was run with and without the recommended capacity improvements in place to verify the need and/or adequacy of each. Each of these improvements is discussed in greater detail under the Recommendations heading of this Section.

Table 6-9 Capacity Improvements Recommended in 2006 Master Plan

Name	Length, feet	Parallel Diameter, inches
Arapahoe Ridge	8,600	10
West Side	3,500	18
Old Town	3,300	30
Austin Industrial	1,800	12

The results of the future system evaluation under the buildout condition are presented in Figures 6-9 to 6-11 and in Table 6-10. This evaluation includes the capacity improvements recommended in the 2006 Master Plan.

Table 6-10 Future / Build-out Wastewater Hydraulic Modeling Results

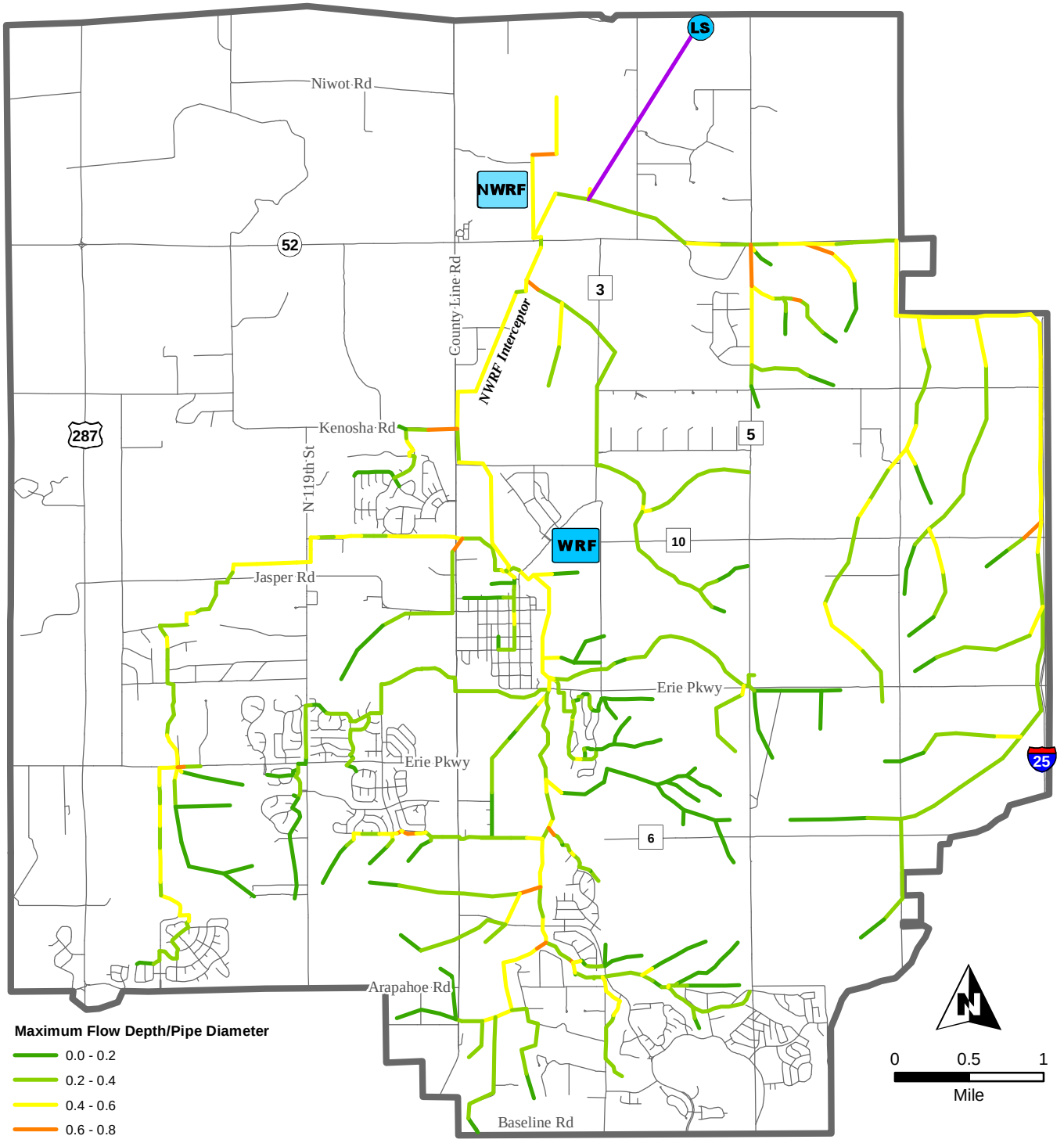
Parameter	Minimum	Maximum	Standard
Flow Velocity at Build-out AAF, fps	0.006	6.48	>2.0
Flow Velocity at Build-out MM, fps	0.006	7.05	<10.0
Flow Velocity at Build-out Peak Hour Flow, fps	0.018	8.50	<10.0
Depth of Flow Ratio at Build-out AAF, d/D	0.014	0.468	<0.4
Depth of Flow Ratio at Build-out MM, d/D	0.015	0.68	<0.8 (for >15"pipe) <0.5 (for <15" pipe)
Depth of Flow Ratio at Build-out Peak Hour Flow, d/D	0.035	0.77	No surcharging

Figure 6-9 shows the depth to diameter (d/D) ratios under maximum month flow conditions. For larger diameter pipes, the d/D should be less than 0.8 and for smaller diameter pipe, it should be below 0.5. A few areas have d/D ratios approaching 0.8 (shown in orange) and should be monitored to ensure that their carrying capacity is not exceeded. Figure 6-10 shows the d/D ratios under peak hour flow conditions. No surcharging was observed at peak hour flows with the recommended capacity improvements from Table 6-9 in place.

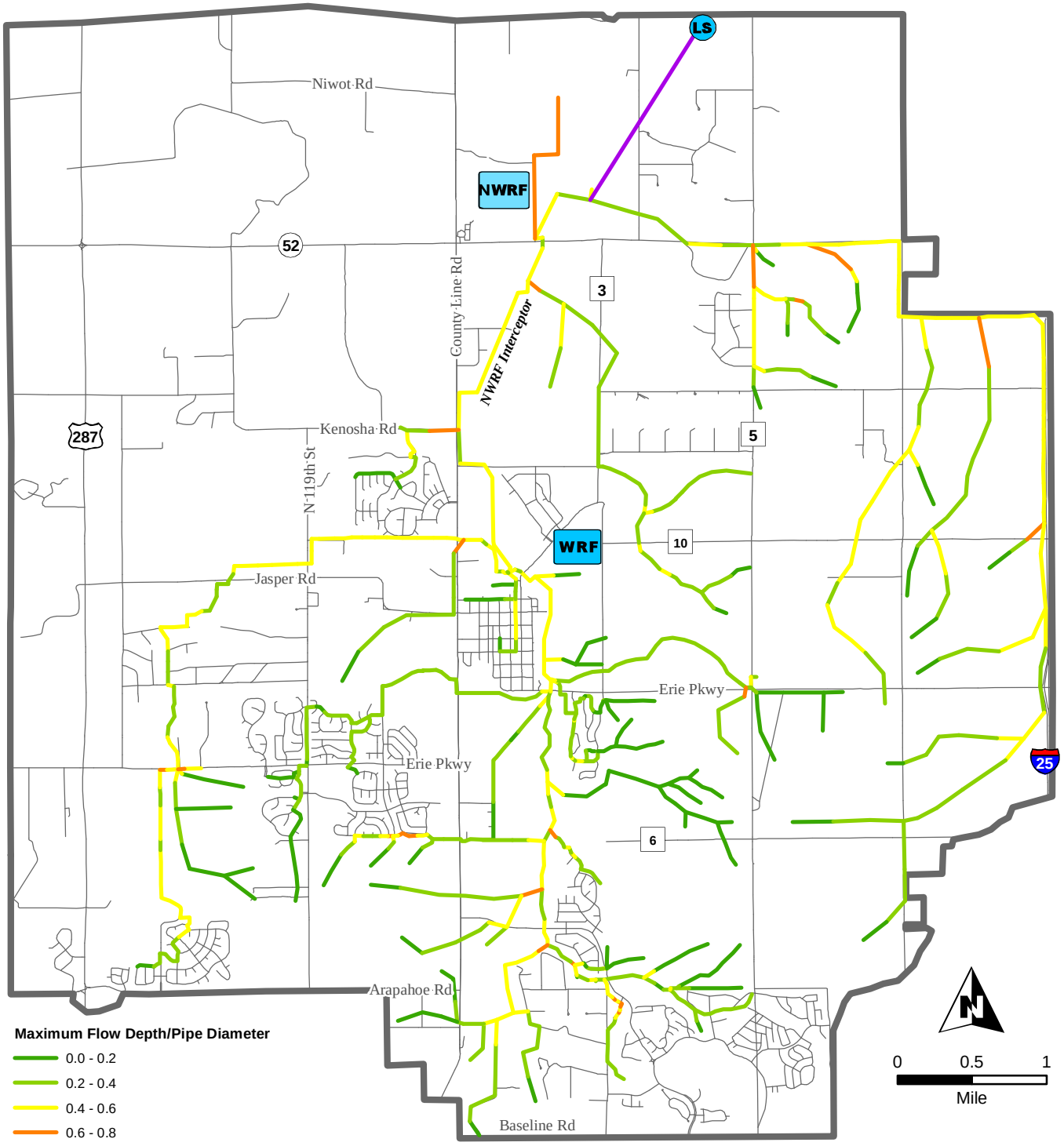
Figure 6-11 illustrates the maximum flow velocities calculated by the model for peak hour flow conditions at buildout. Flow velocities should reach at least 2 fps during peak hour to keep solids in solution and to help scour and resuspend solids that may have been deposited during low flows. Pipes shown in red do not meet the minimum desired flow velocity. Generally, these areas are located at the far upstream end of interceptors where flow rates are low. The West Side interceptor does not meet the minimum desired flow velocity for a long section that runs along County Road 10. These areas may require more frequent cleaning to prevent deposition of solids and blockages.

FUTURE WASTEWATER COLLECTION SYSTEM RECOMMENDATIONS

Recommendations to the existing wastewater collection system from the 2006 Water and Wastewater Master Plan were confirmed by the current modeling effort. These recommendations are compiled into four categories: new major interceptors, extensions to new and existing interceptors, lift stations and force



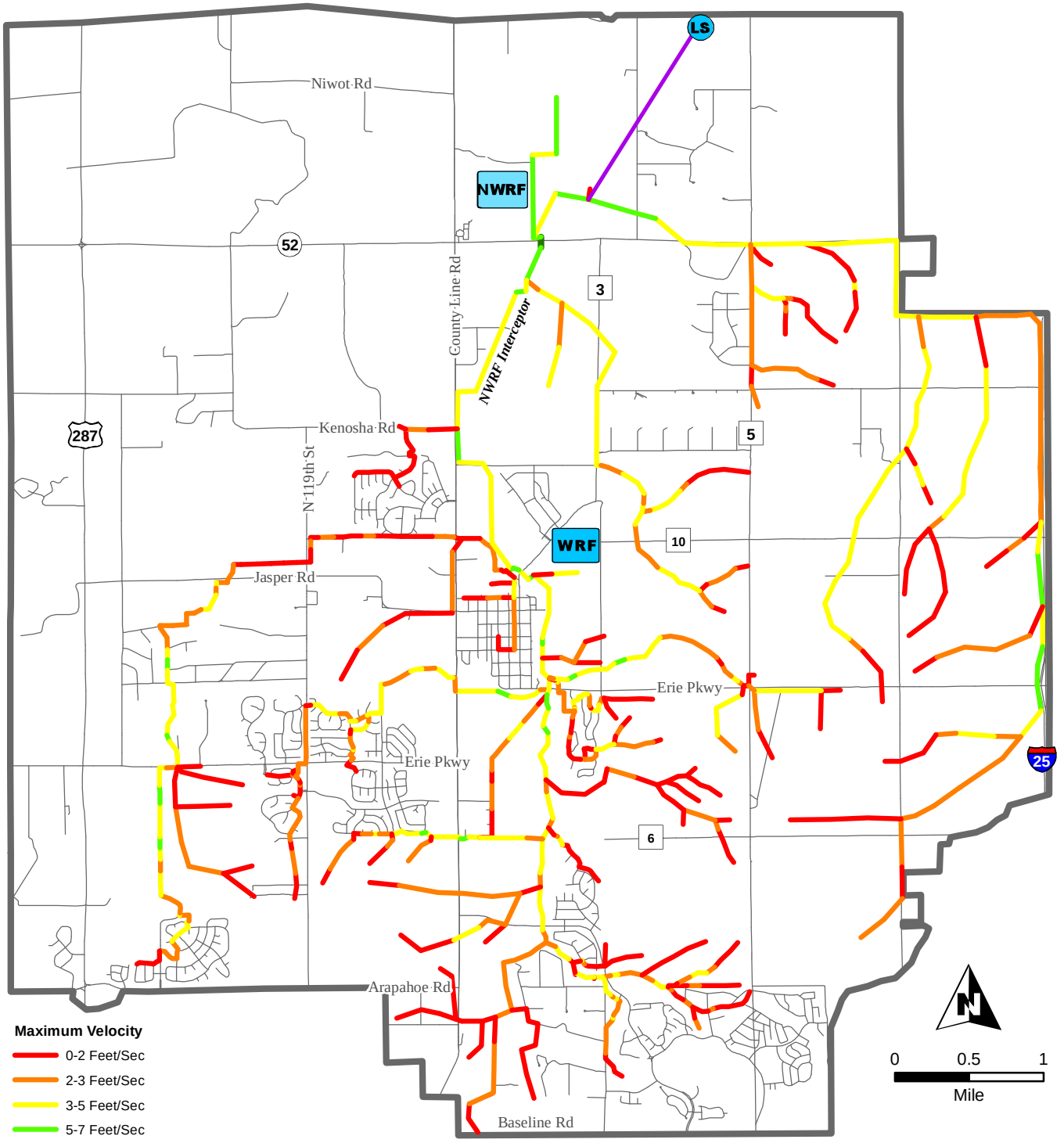
Monthy Maximum at Buildout - Maximum Flow Depth/Pipe Diameter
Figure 6-9



Maximum Flow Depth/Pipe Diameter

- 0.0 - 0.2
- 0.2 - 0.4
- 0.4 - 0.6
- 0.6 - 0.8
- 0.8 - 1.0
- Force Main
- **Planning Area Boundary**

Peak Hour at Buildout - Maximum Flow Depth/Pipe Diameter
Figure 6-10



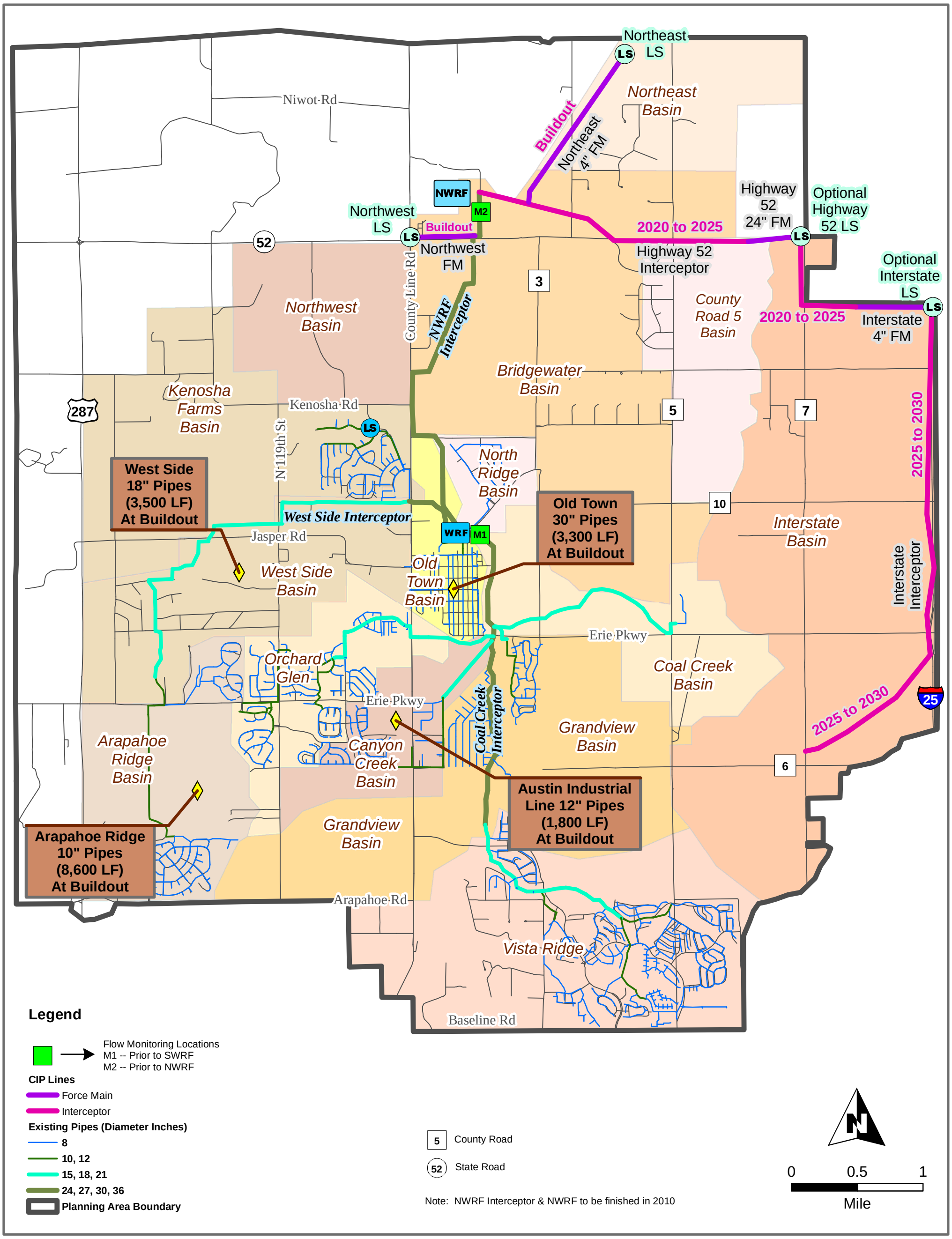
Peak Hour at Buildout - Maximum Velocity
Figure 6-11

mains, and capacity improvements. They are summarized in Tables 6-6 through 6-9. Some alternatives should receive further analysis. Modeling is not a substitute for preliminary engineering and should be used for planning level decisions only. Future analysis should include surveying and mapping, easement investigations, and geotechnical work at a minimum.

The Town has a policy of requiring developers to construct the infrastructure required to provide their developments with water and sewer services. This policy includes provisions for upsizing gravity sewer lines if future development is expected upstream the proposed project. Major interceptors and lift stations can't always be assigned to a single project because of the large areas they will eventually serve. In this case, the Town may elect to provide technical and financial assistance to foster growth. The recommendations listed in Table 6-11 and shown in Figure 6-12 include major interceptors, lift stations, and capacity improvements that the Town will likely take an active role in constructing. These improvements are further described in the following paragraphs. In addition, the interceptor extensions recommended by the 2006 Water and Wastewater Master Plan and summarized in Table 6-7 were confirmed by the current modeling effort and should be constructed. Extensions will probably be constructed by developers.

Table 6-11 Capital Improvement Recommendations

Recommendations	Description	Drainage Basin Served
Category 1 Major Interceptors		
Highway 52 Interceptor – Alternative 1 with lift station	11,400 feet of 18 to 30-inch gravity sewer	Serving three drainage basins: Bridgewater, County Road 5 and Interstate
Highway 52 Interceptor – Alternative 2 with deep gravity sewer and no lift station	13,400 feet of 24 to 30-inch gravity sewer	Serving three drainage basins: Bridgewater, County Road 5 and Interstate
Interstate Interceptor – Alternative 1 with lift station	21,300 feet of 15-inch to 21-inch gravity sewer	Serving one drainage basin: Interstate
Interstate Interceptor – Alternative 2 with deep gravity sewer and no lift station	22,800 feet of 15-inch to 21-inch gravity sewer	Serving one drainage basin: Interstate
Category 2 Lift Stations and Force Mains		
Highway 52 Lift Station and 24" Force Main – Used with Alternative 1 Highway 52 Interceptor only	Pump station firm capacity = 8,000 gpm 1,800 LF of 24-inch force main	Serving one drainage basin: Interstate
Interstate Lift Station and 4" Force Main – Used with Alternative 1 Interstate Interceptor only	Pump station firm capacity = 200 gpm 3,100 LF of 4-inch force main	Serving one drainage basin: Interstate
Category 3 Capacity Improvements for Existing Gravity Sewer Lines		
Arapahoe Ridge	8,600 LF of 10-inch pipelines	Serving Arapahoe Ridge basin under Build-out condition
West Side	3,500 LF of 18-inch pipelines	Serving West Side basin under Build-out condition
Old Town	3,300 LF of 30-inch pipelines	Serving Old Town basin under Build-out condition
Austin Industrial Line	1,800 LF of 12-inch pipelines	Serving Canyon Creek basin under Build-out condition



Recommendations for Collection System
Figure 6-12

Major Interceptors

Highway 52 Interceptor Alternative 1. An interceptor will be required along Highway 52 from Weld County Road 7 to the NWRF. This improvement *may* include a large lift station near Highway 52 and Weld County Road 7, parallel 1,800-foot force mains, and a large gravity line. The 11,400 foot long interceptor will collect flows from several basins including Interstate, County Road 5, parts of Bridgewater, and the Northeast. As development occurs, initial flows are likely to be much smaller than the anticipated buildout flows; therefor, the Town should consider a phased approach.

Interstate Interceptor Alternative 1. Most of the area within the Interstate Basin can flow by gravity to the intersection of Interstate-25 and County Road 12. The proposed interceptor begins near the intersection of County Road 6 and County Road 7 and travels east and north following the eastern edge of the planning area. The interceptor will range in size from 15 to 21-inches in diameter and will be 21,300 feet long. While it is preferred to have developers provide the majority of the new collection system facilities, it may be difficult for a single developer to install this interceptor because it will eventually serve several different developments. If growth occurs along the southern part of the basin first, a developer may not have the financial resources to construct the full length of the interceptor. The Town may need to play a more active role in the development of this interceptor and then seek reimbursement from developers as additional development occurs in the basin.

Highway 52 and Interstate Interceptors Alternative 2. In April 2008, Black and Veatch issued an update to their 2006 Master Plan report titled Technical Memorandum No. 1. This report was in response to a request from the Town to consider a developer's desire to eliminate Highway 52 and Interstate lift stations. Alternate alignments were proposed for both interceptors. The proposed interceptors meet the Town's capacity and velocity criteria for all pipe segments. Eliminating the lift stations will increase the total length of the Highway 52 and Interstate interceptors by 2,000 feet and 1,500 feet, respectively. In addition, the Highway 52 interceptor will increase in size slightly with the smallest diameter pipe changing from 18 to 24-inches. The new alignments will significantly increase the depth of both interceptors, more than 30 feet in some locations.

Deep gravity sewers can be difficult to clean, inspect, and maintain. The deeper sewers will have a lower capital cost and lower operating costs than lift stations.

Lift Stations and Force Mains

The Northeast and Northwest interceptors are not discussed in this section as these improvements will probably be constructed by developers to serve small, defined areas. The anticipated sizes of these lift stations are given in Table 6-8.

Highway 52 Lift Station and 24-inch Force Main. The Highway 52 lift Station and force main are improvements associated with the Highway 52 Interceptor Alternative 1 discussed above. The Highway 52 Lift Station is one of two lift stations that are anticipated to be constructed by the Town. Since the other lift stations will be serving smaller areas, they should be the responsibility of the developer. If the

Highway 52 lift station and force main are constructed, the lift station will need to have a firm pumping capacity of 8,000 gpm. The associated force main will be 24-inches in diameter and 1,800 feet long.

Interstate Lift Station and 4-inch Force Main. The Interstate lift station and force main are improvements associated with the Interstate Interceptor Alternative 1 discussed previously. If the Interstate lift station is constructed, it will need to have a firm pumping capacity of 200 gpm. The associated force main will be 4-inches in diameter and 3,100 feet long.

Highway 52 and Interstate Lift Stations Alternative 2. An alternative to the lift station and force main installation would be to install a gravity sewer appropriately sized in place of the lift station and force main, assuming the depth of the interceptor is low enough to receive the flow from the deep gravity sewer. While the initial construction cost of the lift station and force main may be less, the operation and maintenance (O&M) costs can result in an overall life cycle cost higher than the deeper gravity sewer. Typically, gravity sewers require less operation and maintenance funds, with costs limited to occasional cleaning, internal inspections utilizing an in-line camera, and manhole inspections to ensure that joints are tight and not resulting in inflow and infiltration. Pump station and force main O&M costs include the cost of electricity to power the pumps and heat and light the pump building, routine maintenance and repairs for the pumps, valves, controls and piping appurtenances, SCADA and control system monitoring, facility maintenance, periodic cleaning of the wet well, and staffing costs to check the station for proper operation.

Both the lift station and force main alternatives and the proposed deep gravity interceptors are valid options. The Town may ultimately choose to implement one or the other depending upon the Town's current and future development plans. An accurate cost analysis should be completed prior to determining the best alternative for each situation. Some deep gravity sewers can be cost prohibitive due to the depth of construction and potential for rock removal, groundwater pumping, impact on adjacent structures, and contractors use of special equipment for the depth of construction even though the O&M costs are lower than the pump station alternative. In the cost analysis, the expected life of the gravity sewer would be expected to exceed the pumping station and cost of replacement of the station should be considered.

Temporary Lift Stations. Up to three temporary lift stations may be needed to support growth in outlying areas prior to construction of major interceptors. The temporary lift stations described in Black and Veatch's Technical Memorandum No. 3 (included in Appendix H) are shown on Figure 6-12.

If the Leon Wurl Commercial Area is developed prior to construction of the Interstate and Highway 52 interceptors, a temporary lift station could be constructed near WCR 10 and I-25 to serve the area. The lift station would be sized to accommodate between 830 and 1600 gpm depending on how much of the area was developed. The force main would be routed west to connect with the Coal Creek Interceptor Extension. Continued development along the I-25 corridor will provide additional resources for construction of the Interstate and I-25 interceptors. When completed, the lift station at WCR and I-25 would be abandoned.

Section 10 is another area that may develop either commercially or residentially or both prior to completion of the Interstate and Highway 52 Interceptors. A temporary lift station could be constructed near WCR 5 and I-25 to serve this area. The lift station would be sized to accommodate between 255 and 1050 gpm depending on development. The force main would be routed west to connect with the Coal Creek Interceptor extension. Once a gravity sewer line was available to serve Section 10, the lift station would be abandoned.

A temporary lift station may also be needed to serve development near Bonanza Drive and Highway 7. There are two undeveloped properties at this intersection that could flow by gravity through the airport property; but the airport does not currently have sewer in the area. A small lift station may be constructed to lift flows generated in this area to a gravity line to the northeast.

Capacity Improvements

The 2006 Water and Wastewater Master Plan noted four areas within the existing collection system that are expected to reach 100 percent or more of their capacity at peak hour flows at buildout. Recommendations were made to increase capacity in these areas by adding parallel sewer lines. These recommendations were confirmed by the current modeling effort. The capacity improvements are shown in Figure 6-12 as orange callout boxes.

The Town should keep in mind that recommendations for sizing and placement of interceptors are based on assumptions of when, where, and to what extent growth will actually occur in the future. Future growth patterns may deviate significantly from these assumptions. Monitoring of these areas is recommended and reassessment of the actual flows should be done periodically. Preliminary engineering should be done to confirm the size of each interceptor and recommended capacity improvement.

Arapahoe Ridge. The Arapahoe Ridge Basin is located in the extreme southwest corner of the planning area and includes the proposed Flat Iron Meadow Development. The existing 8- to10-inch line has limited future capacity with constraints at the south end of this line. A parallel10-inch line would provide sufficient capacity for buildout flows, based on the hydraulic model.

West Side. Currently, the West Side line is at less than 40 percent of capacity during peak flows; although the last segments of 18-inch line, before the size increases to 21-inch, are at capacity for the buildout scenario. Full development of Flat Iron Meadow and other developments in the Arapahoe Ridge and West Side basins, could exceed the capacity of this line at some point in the future. A parallel 18-inch line through this stretch may be necessary to provide capacity in this area.

Old Town. The existing line around the Old Town area is currently at less than 40 percent of capacity. This line; however, carries all of the flow from the Vista Ridge, Grandview, Canyon Creek, Orchard Glen, and Coal Creek basins. Future development upstream will exceed the capacity of the line between Leon Wurl Parkway and the line from the North Ridge area. To provide capacity through buildout, a parallel 30-inch line will be required.

Austin Industrial. Residential development south of the existing Canyon Creek development and additional industrial development along County Line Road will exceed the capacity of the existing 12-inch line that serves Canyon Creek and the Austin Industrial Park. A new pipeline along Weld County Road 6 connecting directly to the Coal Creek Interceptor should be considered, as opposed to a installing a parallel line along the existing alignment.

Construction Timeline for Improvements

The population projections and land use map were used for estimating the timeline for construction of major interceptors. When an area has a population of 500 to 1000, this is the trigger for the timeline. A detailed discussion on which parcels were assumed to develop first can be found in Section 4 under the heading *Future Population Growth*.

Interstate Interceptor. Growth projections have populations increasing around the intersection of County Road 7 and County Road 10 between the years 2020 to 2025. This area includes the Erie Corporate Center, low density residential, multiuse, and rural residential land uses. Development at CR 7 and CR 10 would require the section of the interceptor downstream from CR 10 to be built. In the southern part of the basin near County Road 6, land uses include multiuse and low density residential. Population in this area is expected to increase from 2025 to 2030. This increase, along with development along Interstate-25, would require both the Interstate interceptor and the Highway 52 interceptor to be constructed.

The Interstate lift station and force main may or may not be constructed depending on whether or not the deep gravity sewer alternative is selected.

Highway 52 Interceptor. By 2025 to 2030, development north and south of Highway 52 could be populated enough to need this interceptor online. These areas are labeled as Section 4 and SMT Investors 52/5 on the Current Development Activities Map from September 2009. The designated timeline is shown as 2020 to 2025 because construction of the Highway 52 Interceptor may be forced earlier by development upstream in the Interstate basin. The Highway 52 lift station and force main may or may not be constructed depending on whether or not the deep gravity sewer alternative is selected.

FLOW MONITORING

Both the SWRF and NWRF have influent lift stations upstream of their headworks and flow monitoring. As a result, flow monitoring records do not include the daily variations in flow present at the wet well influent and in the collection system. A typical diurnal cycle has flows dropping very low between the hours of 10 pm and 6 am and flows peaking sometime between 8 am and 2 pm. Some facilities see a second peak late in the afternoon as residents return home and make dinner, do laundry, and attend to other household chores. This diurnal curve is dampened or smoothed out by the influent pump stations. As a result, it is impossible to know the actual peak hour flow in the collection system.

When the peak hour peaking factor cannot be determined from historic data, NFRWQPA and DRCOG require the use of a standard formula to estimate the peak hour peaking factor. The formula is

conservative and can result in peak hour peaking factors that are significantly higher than actual field conditions. For the Town of Erie, the DRCOG formula results in peak hour peaking factors of 3.45 for the SWRF when receiving 1.6 MGD of flow. The peak hour peaking factor for the NWRF is estimated by the formula at 3.49 for 1.5 MGD of flow and at 2.90 for 3.9 MGD of flow.

Table 6-12 lists three communities located close to Erie. Each of these communities monitors their wastewater flows at their respective facilities and records both the average daily flow and peak hour flow each day. For each of these communities, the DRCOG formula overestimated the peak hour peaking factor. The City of Northglenn has a peak hour peaking factor of 2.5, but the DRCOG formula estimated a factor of 2.89 which is 16 percent higher. For Loveland, the DRCOG formula was 12 percent higher and for St. Vrain, it was 28 percent higher.

Table 6-12 Peak Hour Peaking Factors for Communities in Colorado

Community	Population	AAF, mgd	Peak Hour Peaking Factor	
			Actual	DRCOG
City of Northglenn[1]	33,250	4.0	2.5	2.89
City of Loveland[2]	63,538	6.0	2.38	2.71
St. Vrain SD[3]	18,000	1.28	2.0[4]	2.78[4]

[1]Flow data from years 1999 – 2001, Source: (Integra Engineering June 2003)
[2]Flow data from years 1994 – 2005, Source: (CH2M Hill March 2010)
[3]Flow data for 2008, Source: (The Engineering Company January 2009)
[4]Peak hour peaking factor selected for future planning at 5.0 MGD

It is highly recommended that the Town of Erie purchase flow meters and place them in-pipe immediately upstream of both the SWRF and the NWRF. Flow monitoring will enable the Town to determine the actual peak hour peaking factor for each facility. Since the gravity sewer lines in the collection system, pump station pumps, pump station wet wells, and WRF headworks and disinfection equipment are all sized based on the peak hour flow, flow monitoring could potentially result in significant cost savings for the Town. For example, if the peak hour peaking factor at the NWRF is significantly lower than the DRCOG formula calculated value, the Town may be able to delay or postpone indefinitely installation of an additional screen and/or the fourth influent pump. In the collection system, knowing the actual peak hour peaking factor may mean the difference between upsizing an interceptor.

The State of Colorado Water Quality Control Division recommends using one full year of influent data to determine peak hour peaking factors. They prefer three years to account for year to year variations and seasonal trends. For this reason, purchase of flow monitors is recommended over hiring a service or renting equipment. Flow monitoring has an added benefit in assisting the wastewater reclamation facility and collection system staff to determine when and if any inflow and infiltration is occurring.

PREVENTATIVE MAINTENANCE

The primary goal of collection systems operation is to transport wastewater uninterrupted from the source to a treatment facility, keeping the wastewater in the system and the groundwater out. A collection system can represent the single largest investment for a community and should be protected and cared for accordingly.

Preventative maintenance management includes planned and scheduled line maintenance or repairs based on past history and includes the systematic inspection and cleaning of the entire system. Preventative maintenance can reduce operation costs during both the short and long term because it is far less expensive to maintain system appurtenances that to repair or replace them. Additionally, with increased attention to the system, the incidence of backups and odor complaints can be greatly reduced and result in savings in labor and cleanup costs not to mention potential liability costs.

Table 6-13 summarizes recommended cleaning and inspection intervals for manholes and sewer lines (Water Environment Federation, 1999). Most municipalities budget to clean between 25 and 30 percent of their collection system each year. For the Town of Erie, the newest portions of the collection system should be inspected once every 5 to 8 years. Current policy is to clean and inspect 25% of the system once every four years. The older portions of the collection system and areas with heavy restaurant use should be inspected at least once every three years and may need more frequent cleaning and televising.

Table 6-13 Recommended Cleaning and Inspection Intervals for Manholes and Sewer Lines

Item	Description	Frequency
Manholes	Inspect interior. Regrout pipes and joints as needed. Verify lid sits flush without rocking. Check for evidence of infiltration. Verify bench is free of debris and gravel.	3 – 5 years
Sewer Lines	Sound pipe, good flow characteristics	5 – 8 years
	Pipe with minor defects, good flow characteristics	3 – 5 years
	Pipe with minor defects, poor flow characteristics	1 – 3 years
	Pipe with significant defects, poor flow characteristics	Annually

A good preventative maintenance program depends on properly trained personnel. State certification programs, correspondence courses, technical schools, and in-house training programs are all sources of essential training to ensure the best results.

SUMMARY

The collection system model developed by Black and Veatch in 2006 was updated to include infrastructure completed since that time. Previous recommendations were verified by this modeling effort. Recommendations for capacity improvements and new major interceptors are summarized visually in Figure 6-12. Extensions of existing interceptors and two small lift stations will likely be constructed by developers and are shown in Figure 6-12. Two new major interceptors will be needed to serve currently undeveloped portions of the planning area. These interceptors are the Interstate interceptor and the Highway 52 interceptor. Both of these interceptors were modeled as traditional gravity sewers with lift stations and as deep gravity sewers. In some locations, the deep gravity sewers were up to 35 feet deep.

Modeling did not identify any surcharging under 2010, 2025, or buildout flows when the recommended improvements were in place. Four areas were identified for capacity improvements including: Arapahoe Ridge, West Side, Old Town, and the Austin Industrial Line.

SECTION 7
WATER QUALITY CHARACTERIZATION

Surface and ground waters throughout the United States are managed through a system of state and federal regulations to protect public health and maintain water quality. The condition of the receiving stream, its capacity to absorb waste loads without degradation, and downstream uses of the water determine the degree and often the type of wastewater treatment required. Water quality standards and effluent discharge regulations are presented in this section to establish the water quality management objectives for the Town of Erie.

WATER QUALITY MANAGEMENT AGENCIES

The Colorado Water Quality Control Commission (WQCC) holds primary authority to establish water quality regulations to meet the goals of the Colorado Water Quality Control Act of 1973, as amended. Colorado regulations governing surface water quality consist of three tiers including (1) use classifications, (2) water quality standards, and (3) effluent discharge permits. Colorado surface waters have been assigned use classifications to protect all current uses and maintain the highest water quality possible. The Colorado WQCC has established water quality standards to protect and maintain designated uses corresponding to each use classification.

The WQCC has delegated responsibilities for implementation of water quality regulations to the Water Quality Control Division (WQCD) of the Colorado Department of Public Health and Environment (CDPHE). Discharge permits issued by the WQCD under the Colorado Discharge Permit System (CDPS) enforce the Colorado surface water use classifications and corresponding water quality criteria.

The Town of Erie includes land in both Weld and Boulder Counties. Each county falls under the jurisdiction of its own water quality management agency. Because both of Erie's Water Reclamation Facilities are within Weld County, North Front Range Water Quality Planning Association is the primary water planning agency for the Town. Portions of the Town's planning area are within Boulder County which was previously managed by Denver Regional Council of Governments and now falls under the State.

North Front Range Water Quality Planning Association

The North Front Range Quality Planning Association (NFRWQPA) is an association of counties, municipalities and districts formed to "promote regional water quality planning cooperation and coordination among local governments and others and between levels of government in Larimer County and Weld County, which constitute the geographical area of interest to its members, and which is referred to herein as the 'Region'." As the designated water quality management planning agency, NFRWQPA is required to prepare and update as needed a Regional Water Quality Management Plan. Commonly referred to as the 208 Plan because the requirements were set forth in Section 208 of the Federal Clean Water Act of 1972, the plan addresses regional water quality management issues. As defined in Federal

and State law the 208 Plan is not a regulatory directive but is a planning document intended to inform and guide water resource related activities based on a comprehensive understanding of the critical issues. The NFRWQPA is the designated water quality management planning agency for the Larimer and Weld county area of Colorado. Its Regional Water Quality Management Plan is entitled "Areawide Water Quality Management Plan (208 Plan) for Larimer and Weld Counties". It was adopted in 1988 and was most recently updated in 2007.

Denver Regional Council of Governments

The Denver Regional Area Council of Governments (DRCOG) was the designated regional water quality management planning agency for the greater Denver area in Colorado. The water quality planning portion of DRCOG was disbanded in 2010 with all review responsibilities returning to the State level. The DRCOG Region included Adams, Arapahoe, Boulder, Clear Creek, Douglas, and Jefferson counties, and the City and County of Broomfield and the City and County of Denver. Referred to as the *Clean Water Plan*, DRCOG's current 208 Plan for the Denver Region was adopted in 1998 and was most recently amended in January 2009. As stated in the Plan's introduction, "The *Clean Water Plan* provides a regional context for protecting and maintaining water quality through integrated watershed management processes."

WATER QUALITY OF RECEIVING WATERS

The Town of Erie's wastewater reclamation facilities discharge to waters of the Boulder Creek Watershed. Boulder Creek, in turn, is tributary to the South Platte River via St. Vrain Creek. Originating in the vicinity of the Town of Fairplay in the center of Colorado, the South Platte River drains most of the northeast quadrant of the state. Boulder Creek drains an area in the west central South Platte Watershed as shown on Figure 7-1.

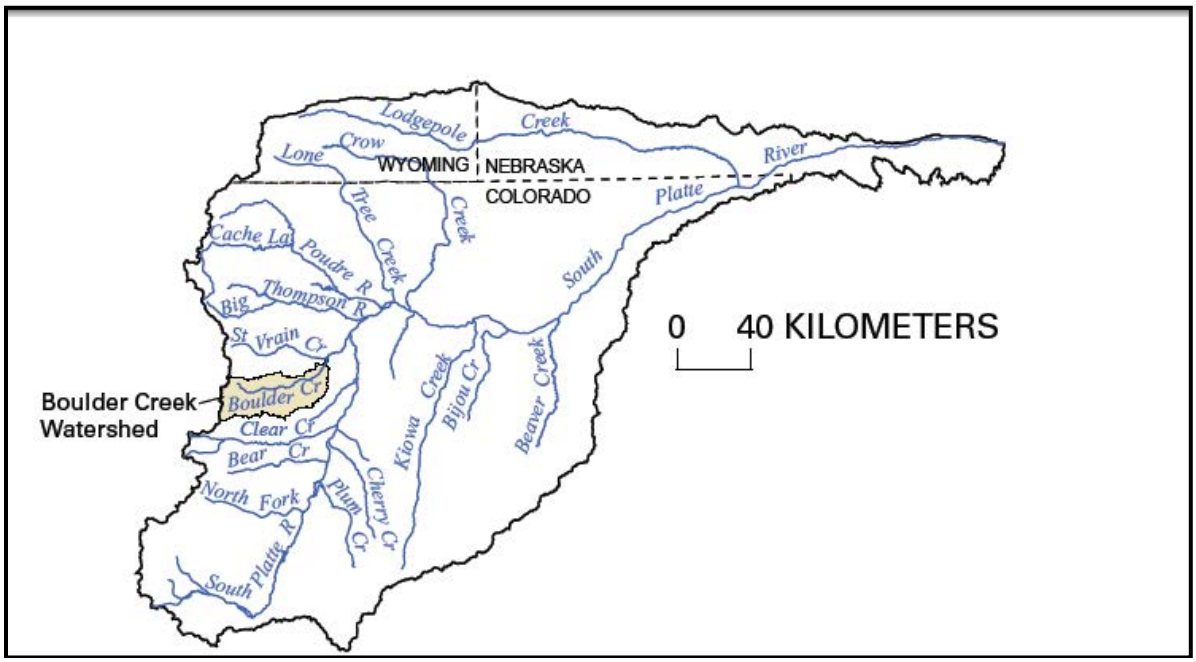


Figure 7-1 South Platte River Watershed

The Town of Erie South Water Reclamation Facility (SWRF) discharges to Coal Creek, a tributary of Boulder Creek approximately three miles upstream of their confluence. Coal Creek is designated as Water Body Identification (WBID) stream segment COSPBO07b which means the South Platte River Basin, Boulder Creek Sub-basin, and Stream Segment 07b. This segment is composed of the "Mainstem of Coal Creek from Highway 36 to the confluence with Boulder Creek".

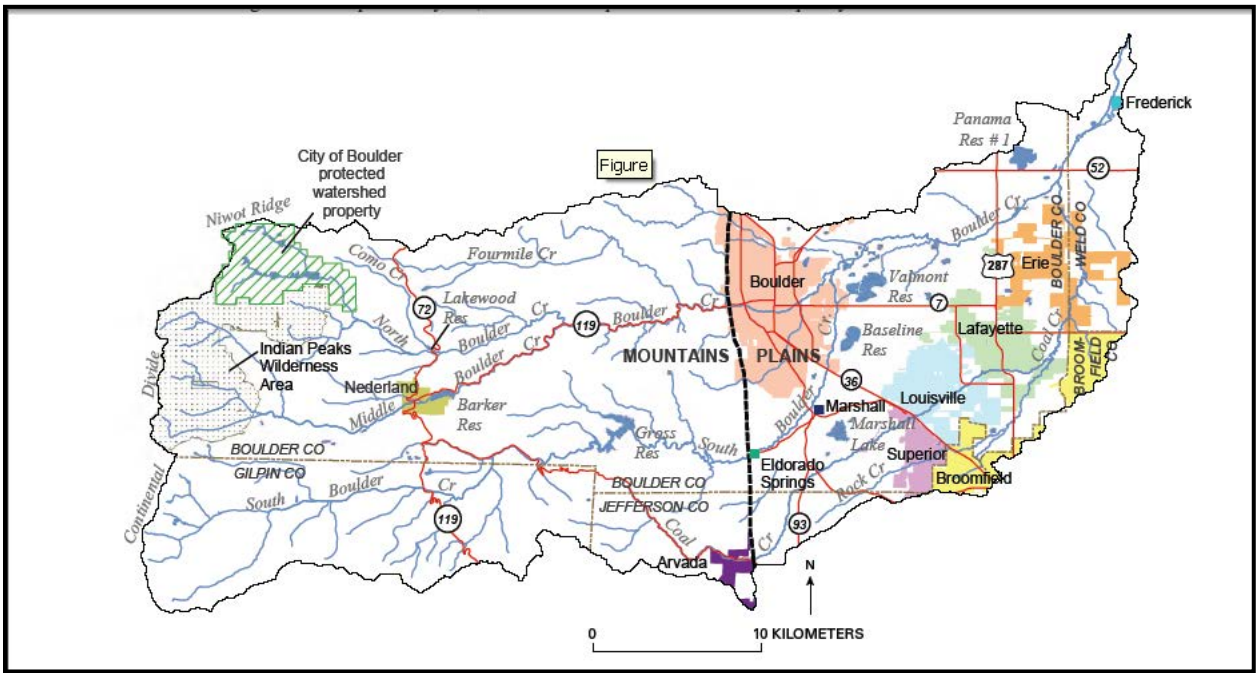


Figure 7-2 Boulder Creek Watershed

The North Water Reclamation Facility discharge point is directly into Boulder Creek approximately one mile downstream of the confluence with Coal Creek and approximately six miles above the confluence of Boulder Creek with St. Vrain Creek. The designation for this stretch of Boulder Creek is WBID stream segment COSPBO10 which means the South Platte River Basin, Boulder Creek Sub-basin, and Stream Segment 10, comprised of the main stem of Boulder Creek from the confluence with Coal Creek to the confluence with St. Vrain Creek. Figure 7-2 shows the Boulder Creek Watershed and the relative location of the Town of Erie at the eastern end.

Water Quality Standards

Water quality standards are developed on a basin-specific basis and are adopted for particular stream segments by the Water Quality Control Commission. Colorado's water quality standards include four major components:

- Numeric criteria are estimations of concentrations of chemicals and degree of aquatic life toxicity allowable in a water body without adversely impacting its designated uses.
- Narrative criteria are generally water quality criteria used to limit the toxicity of waste discharges to species. Narrative criteria currently exist for sedimentation and temperature.

- Antidegradation provisions establish that beneficial uses must be maintained and protected.
- Beneficial use classifications consist of categories of uses for which surface waters are to be protected. Colorado uses ten use classifications including four aquatic life protection, three classes of recreation protection, and classifications for agriculture and public water supply.

Protection for each beneficial use requires a different numeric standard for each constituent. The governing standard for each individual water quality parameter is the limit established as necessary to protect the most sensitive designated use. Stream segment COSPBO07b (Coal Creek) is classified for Warm Water Aquatic Life Class 2, Recreation Class E, and Agriculture (State of Colorado amended 4/12/10, effective 6/30/10). Coal Creek's classification was changed from Use Protected to Reviewable in 2009.

Loss of Use Protected (UP) status may ultimately result in lower permit limits for the SWRF, especially metals, because an anti-degradation analysis must be completed as part of each permit renewal. Anti-degradation analysis limits the total load of metals and other pollutants to a receiving water in order to protect baseline water quality. In this situation, it is possible for a permittee to receive discharge limits that are more stringent than the water quality based effluent limits (WQBELs). Use Protected status allowed some level of degradation so long as downstream uses such as water supply were protected. Erie may be able to justify higher effluent limits based on economics and technical feasibility. The highest limits attainable for any parameter would be the Water Quality Based Effluent Limits (WQBEL) which is calculated using the stream standard and any available in-stream dilution. This approach would allow for more manageable effluent metals even after a future expansion.

Stream segment COSPBO10 (Boulder Creek) is classified for Warm Water Aquatic Life Class 1, Recreation Class E, water supply and Agriculture (State of Colorado amended 4/12/10, effective 6/30/10). The standards in Table 7-1 have been assigned to stream segments COSPBO07b and COSPBO10 in accordance with Regulation 38 (State of Colorado amended 4/12/10, effective 6/30/10).

Parameters of concern for the Boulder Creek Watershed include ammonia, manganese, arsenic, zinc, silver, copper, cadmium, lead, mercury (Denver Regional Council of Governments June 1998). In addition to these parameters, Coal Creek is currently listed as impaired for *E. coli* (Environmental Protection Agency 2008).

Metals standards are generally shown as Table Value Standards (TVS) as their acute and chronic toxicity are known to vary with hardness and the species of fish present. The Table Value Standards require hardness values to calculate the in-stream standards for the various metals. Insufficient paired hardness and flow data were available for the lower reaches of Coal Creek to permit a regression analysis for low flow conditions to be made (Oppelt 2006). As an alternative, a mean hardness was calculated based on sampling data collected by the Town of Erie from locations upstream and downstream of the Erie WRF.

The calculated mean hardness of 247 mg/l was used in the the formulas contained in the TVS to calculate the in-stream water quality standards for metals for the SWRF, with the results shown in Table 7-2.

Daily flow data for Boulder Creek near the point of discharge of the North WRF was not available to conduct a regression analysis based on the low flow. Therefore, the Division's alternative approach to calculating hardness was used, which involves computing a mean hardness. The mean hardness at the NWRF discharge location was computed to be 172 mg/l based on sampling data from CDPH&E station 000033, (Boulder Creek at Boulder-Weld County Line) located on Boulder Creek less than ¼ mile upstream from proposed Erie North WRF discharge (Regulatory Management, Inc. March 2006). There were ninety-seven data points from this station available for the period of record from January 1995 through December 2004. The results are shown in Table 7-3.

The chronic dissolved manganese, chronic dissolved iron, and sulfate standards set out in the regulations for Boulder Creek are specified as WS meaning "water supply," and the regulations specify that the standards are only applicable in cases where there is a water supply in existing use on the stream segment. Boulder Creek downstream from the confluence with Coal Creek is not an actively used water supply and therefore the standards for these parameters do not apply at this time (Regulatory Management, Inc. March 2006). However, it should be noted that the chronic manganese standard based on Table Value Standards (TVS) does apply, as it is an aquatic life protection standard and not dependent upon water supply use.

-- Tables Follow --

Table 7-1 Water Quality Standards for Coal Creek and Boulder Creek

Parameter	*In-Stream Water Quality Standard*			
	Coal Creek WBID COSPBO07b		*Boulder Creek WBID COSPBO10*	
Physical and Biological				
Dissolved Oxygen (DO)	5.0 mg/l		5.0 mg/l	
pH	6.5-9.0 su		6.5-9.0 su	
E. Coli	126/100 ml		126/100 ml	
Inorganic				
NH_3	Acute / Chronic	TVS	Acute / Chronic	TVS
Chlorine	Acute	0.019 mg/l	Acute	0.019 mg/l
	Chronic	0.011 mg/l	Chronic	0.011 mg/l
Free cyanide		0.005 mg/l		0.005 mg/l
Sulfide		0.002 mg/l		0.002 mg/l
Boron		0.75 mg/l		0.75 mg/l
Nitrite		0.5 mg/l		0.5 mg/l
Nitrate		100 mg/l		10 mg/l
Chloride				250 mg/l
Sulfate				WS
Metals				
Arsenic	Acute	340 µg/l	Acute	340 mg/l
	Chronic	100 µg/l(Trec)	Chronic	0.02 (Trec)
Dissolved Cadmium	Acute/Chronic	TVS	Acute/Chronic	TVS
Total Recoverable Trivalent Chromium	Acute/Chronic	TVS		50 µg/l
Dissolved Hexavalent Chromium	Acute/Chronic	TVS	Acute/Chronic	TVS
Dissolved Copper	Acute/Chronic	TVS	Acute/Chronic	TVS
Dissolved Iron			Chronic	WS
Total Recoverable Iron	Chronic	1,000 ug/l	Chronic	1,000 µg/l
Lead	Acute/Chronic	TVS	Acute/Chronic	TVS
Manganese	Acute/Chronic	TVS	Acute/Chronic	TVS
Dissolved Manganese			Acute/Chronic	WS
Total Mercury	Chronic	0.01 µg/l	Chronic	0.01 µg/l
Nickel	Acute/Chronic	TVS	Acute/Chronic	TVS
Selenium	Acute	TVS	Acute	TVS
Silver	Acute/Chronic	TVS	Acute/Chronic	TVS
Zinc	Acute/Chronic	TVS	Acute/Chronic	TVS

Table 7-2 TVS-Based Metals Water Quality Standards for Erie South WRF

Calculated using the Following Value for Hardness				247 mg/l as $CaCO_3$
Parameter	*In-Stream Water Quality Standard*			*Formula Used*
Cadmium, (Diss)	Acute	6.0	µg/l	$[1.13667-0.041838\ln(hardness)][e^{.915(\ln(hardness))-3.1485}]$
	Chronic	0.84	µg/l	$[1.101672-0.041838\ln(hardness)][e^{(0.7996(\ln(hardness))-4.451)}]$
Hexavalent Chromium, (Diss)	Acute	16	µg/l	Numeric Standards provided, formula not applicable
	Chronic	11	µg/l	Numeric Standards provided, formula not applicable
Copper, (Diss)	Acute	31	µg/l	$e^{(0.9422(\ln(hardness))-1.7408)}$
	Chronic	19	µg/l	$e^{(0.8545(\ln(hardness))-1.7428)}$
Lead, (Diss)	Acute	170	µg/l	$[1.46203-0.145712\ln(hardness)[e^{1.273(\ln(hardness))-1.46)}]$
	Chronic	6.6	µg/l	$[1.46203-0.145712\ln(hardness)[e^{1.273(\ln(hardness))-4.705)}]$
Manganese, (Diss)	Acute	4035	µg/l	$e^{(0.3331(\ln(hardness))+6.4676)}$
	Chronic	2229	µg/l	$e^{(0.3331(\ln(hardness))+5.8743)}$
Nickel, (Diss)	Acute	1006	µg/l	$e^{(0.846(\ln(hardness))+2.253)}$
	Chronic	112	µg/l	$e^{(0.846(\ln(hardness))+0.0554)}$
Selenium (Diss)	Acute	18.4	µg/l	Numeric Standards provided, formula not applicable
	Chronic	4.6	µg/l	Numeric Standards provided, formula not applicable
Silver, (Diss)	Acute	9.61	µg/l	$\frac{1}{2}\ e^{(1.72(\ln(hardness))-6.52)}$
	Chronic	1.52	µg/l	$e^{(1.72(\ln(hardness))-9.06)}$
Zinc, (Diss)	Acute	310	µg/l	$0.978e^{(0.8525(\ln(hardness))+1.0617)}$
	Chronic	269	µg/l	$0.986e^{(0.8525(\ln(hardness))+0.9109)}$

Table 7-3 TVS-Based Metals Water Quality Standards for Erie North WRF

Calculated using the Following Value for Hardness				172 mg/l
Parameter	*In-Stream Water Quality Standard*			*Formula Used*
Cadmium, Dissolved	Acute	4.4	µg/l	$[1.13667-0.041838\ln(hardness)][e^{.915(\ln(hardness))-3.1485)}]$
	Chronic	0.64	µg/l	$[1.101672-0.041838\ln(hardness)][e^{(0.7996(\ln(hardness))-4.451)}]$
Hexavalent Chromium, Dissolved	Acute	16	µg/l	Numeric Standards provided, formula not applicable
	Chronic	11	µg/l	Numeric Standards provided, formula not applicable
Copper, Dissolved	Acute	22.4	µg/l	$e^{(0.9422(\ln(hardness))-1.7408)}$
	Chronic	14.2	µg/l	$e^{(0.8545(\ln(hardness))-1.7428)}$
Lead, Dissolved	Acute	115.9	µg/l	$[1.46203-0.145712\ln(hardness)[e^{1.273(\ln(hardness))-1.46)}]$
	Chronic	4.5	µg/l	$[1.46203-0.145712\ln(hardness)[e^{1.273(\ln(hardness))-4.705)}]$
Manganese, Dissolved	Acute	3577	µg/l	$e^{(0.3331(\ln(hardness))+6.4676)}$
	Chronic	1976	µg/l	$e^{(0.3331(\ln(hardness))+5.8743)}$
Nickel, Dissolved	Acute	741	µg/l	$e^{(0.846(\ln(hardness))+2.253)}$
	Chronic	82	µg/l	$e^{(0.846(\ln(hardness))+0.0554)}$
Selenium Dissolved	Acute	18.4	µg/l	Numeric Standards provided, formula not applicable
	Chronic	4.6	µg/l	Numeric Standards provided, formula not applicable
Silver, Dissolved	Acute	5.2	µg/l	$\frac{1}{2}\ e^{(1.72(\ln(hardness))-6.52)}$
	Chronic	0.81	µg/l	$e^{(1.72(\ln(hardness))-9.06}$
Zinc, Dissolved	Acute	228	µg/l	$0.978e^{(0.8525(\ln(hardness))+1.0617)}$
	Chronic	197	µg/l	$0.986e^{(0.8525(\ln(hardness))+9109)}$

Ambient Water Quality

Stream flow and water quality data from monitoring sites close to the discharge point are used to establish the ambient water quality, determine the assimilative capacities of the receiving waters for pollutants of concern and set the permit limits for effluent discharges. Table 7-4 presents ambient water quality data for Coal Creek as presented in the Water Quality Analysis issued with the new discharge permit in September 2010 .

Table 7-4 Ambient Water Quality for Coal Creek

Parameter	*Number of Samples*	*15th Percentile*	*50th Percentile*	*85th Percentile*	*Mean*	*Chronic Stream Standard*
Temp (ºC)	83	8.4	12	18	13	N/A
pH (SU)	85	7.6	7.7	7.9	7.7	6.5 – 9.0
E. coli	79	46	225	1,000	205	126
Hardness (mg/l	49	210	238	274	247	N/A
As, Trec (µg/l)	1	0	0	0	0	100
As, Dis (µg/)	0	0	0	0	0	340
Cd, Dis (µg/l)	4	0	0	0	0	0.80
Cr, Dis (µg/l)	4	0	0	0	0	N/A
Cr+2, Dis (µg/)	4	0	0	0	0	149
Cr^{+6}, Dis (µg/l)	1	11	11	11	11	11
Cu, Dis (µg/l)	4	0	0	1.2	0.55	19
CN, Free (µg/l)	1	0	0	0	0	N/A
Fe, Trec (µg/l)	4	238	355	418	330	1,000
Pb, Dis (µg/l)	4	0.0	0.10	4.4	2.0	6.3
Mn, Dis (µg/l)	4	9.5	12	13	11	2190
Hg, Tot (µg/l)	4	0	0	0	0	0.01
Ni, Dis (µg/l)	1	0	0	0	0	0
Se, Dis (µg/l)	15	0	1.9	3	1.7	4.6
Ag, Dis (µg/l)	4	0	0	0	0	1.4
Zn, Dis (µg/l)	4	0	2.8	8.0	3.9	257
Sulfate (mg/l)	0	0	0	0	0	N/A
Chloride (mg/L)	0	0	0	0	0	NA
P, Tot (mg/l)	3	0.019	0.040	0.047	0.033	N/A
Nitrate (mg/L)	70	0.11	0.3	0.54	0.41	100
Nitrite (mg/l)	83	0.0026	0.0062	0.01	0.0067	0.5
NH_3, Total (mg/l)	4	0.032	0.085	0.11	0.073	TVS
TSS (mg/)	3	0	0	0	0	N/A

Table 7-5 presents ambient water quality data for Boulder Creek as presented in the Water Quality Analysis issued with the NWRF discharge permit in 2011. Background water quality is based on data from the Colorado Water Quality Control Division Site 000033, Boulder Creek at Boulder-Weld County line. This site is immediately above the proposed discharge point for the NWRF and below Coal Creek, the last major tributary and above the Goulding-Plumb and the Idaho Creek ditches, the last major diversions. The period of record is January 2003 through July 2008.

Table 7-5 Ambient Water Quality for Boulder Creek

Parameter	*Number of Samples*	*15th Percentile*	*50th Percentile*	*85th Percentile*	*Mean*	*Chronic Stream Standard*	*Notes*
Temp (ºC)	86	6	13	23	14		
DO (mg/l)	77	8.6	11	13	4.1		
pH (SU)	81	8	8.6	8.9	8.5	6.0 – 9.0	
E. coli	83	44	116	435	364	126	1
As, Dis (µg/l)	71	0.95	1	3.6	2.3	340	
Cd, Dis (µg/l)	84	0	0.45	0.5	0	0.73	2
Cr+3, Trec (µg/l)	0	0	0	0	0	50	2
Cr^{+6}, Dis (µg/l)	0	0	0	0	0	11	2
Cu, Dis (µg/l)	84	0	2.6	5.9	3	17	
Fe, Dis (µg/l)	84	16	37	108	93	300	2,4
Fe, Trec (µg/l)	84	209	480	1777	1471	1000	
Pb, Dis (µg/l)	84	0	0.35	1.3	1.1	5.4	2,4
Mn, Dis (µg/l)	84	9.2	24	49	32	50	2
Se, Dis (µg/l)	84	0.38	1	4.8	2.3	4.6	
Ag, Dis (µg/l)	84	0	0.14	0.14	0.3	1.1	2,3
U, Dis (µg/L)	23	4	7	10	7.2	3310	
Zn, Dis (µg/l)	84	6	19	30	21	229	2,3
Sulfate (mg/l)	28	81	125	170	132	250	2
Nitrite+Nitrite as N	28	1.3	3.9	5.9	3.6		
NH_3, Total (mg/l)	84	0.1	0.1	2.3	0.75	TVS	
TSS (mg/)	52	33	230	980	175	200	1

Low flows were calculated for the Coal Creek Watershed in a Total Maximum Daily Load (TMDL) analysis for the Boulder Creek basin. These low flow values are summarized in Table 7-6. The Water Quality Analysis notes that although Coal Creek is a gaining stream, there aren't any confluences on Coal Creek between the Louisville, Superior, and Erie treatment facilities. The increases in flow during times of low flow are almost solely attributable to effluent discharges. Therefore, the true dilution flow available in Coal Creek is limited to 0.3 cfs (acute) and 0.6 cfs (chronic) (Oppelt 2006). A more recent analysis completed by CDPHE in 2011 found the acute and chronic low flow in Coal Creek to be 0.1 cfs.

Table 7-6 Low Flows for Coal Creek as Calculated in the Boulder Creek TMDL for Ammonia

Stream Reach	Acute Flow, cfs	Chronic Flow, cfs
Coal Creek above Louisville	0.3	0.6
Louisville Discharge	5.3	5.3
Superior through Rock Creek	3.7	3.7
Lafayette Discharge	6.8	6.8
Erie South WRF	2.48	2.48
Erie North WRF	6.4	12.2

Note: 2.48 cfs is equivalent to 1.6 MGD

TOTAL MAXIMUM DAILY LOADS AND/OR WASTE LOAD ALLOCATIONS

The Erie SWRF and NWRF are currently subject to effluent limitations for total ammonia based on a TMDL for the Saint Vrain Watershed, including Boulder and Coal Creeks, that was completed in 2003

and updated in 2008 (William M. Lewis Jr. 2008). Effluent ammonia and nitrate limits for the SWRF calculated in the 2008 TMDL are presented in Table 7-7. The original TMDL was calculated using the Colorado Ammonia Model (CAM). The updated TMDL used the AMMTOX model calculate effluent ammonia limits for all dischargers (William M. Lewis Jr. 2008). The Saint Vrain Watershed TMDL analysis also included an analysis of nitrate concentrations and low flows for the entire watershed. The TMDL and waste load allocation (WLA) were implemented to control past ammonia problems. The imposed limits have been effective at reducing in-stream ammonia concentrations and currently neither segment COSPBO07b or COSPBO10 are listed as impaired for ammonia.

Table 7-7 Effluent Ammonia and Nitrate Limits for SWRF as Calculated in the 2008 TMDL Analysis

	J	F	M	A	M	J	J	A	S	O	N	D
NH_3-N Acute	4.84	2.80	3.31	2.93	2.25	3.77	2.00	1.87	2.09	4.43	8.95	7.78
NH_3-N Chronic	4.83	2.8	3.08	2.69	2.25	2.42	2.00	1.87	2.09	3.17	4.78	4.98
NO_3-N	23.12	24.84	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	24.43	25.00

All limits are in mg/L as Nitrogen. Source: (William M. Lewis Jr. 2008)

Low flows are typically determined using DFLOW for individual gage stations. For the TMDL, low flows were established by a complete hydrologic accounting of tributary flows and ungaged flows (including seepage) as well as ditch withdrawals for acute and chronic low flows (William M. Lewis Jr. 2008). The hydrologic analysis resulted in quantitative estimates of system-wide low flows under recurrence intervals corresponding to acute (1E3) and chronic (30E3) conditions used for water quality protection by the State of Colorado.

As mentioned previously in Section 3, Colorado's Water Quality Control Division adopted water quality based nutrient standards for Total Nitrogen, Total Phosphorus, and chlorophyll a. At a minimum, both the SWRF and NWRF will be required to meet the Total Inorganic Nitrogen (TIN) and Total Phosphorus (TP) limits for existing facilities set forth in Regulation 85 of 15 mg/L and 1 mg/L, respectively. Because both facilities are rated for less than 2 mgd, implementation will be delayed until the year 2022. If the water quality based nutrient standards are adopted in the South Platte River Basin hearings in 2017, both facilities could be required to meet much more stringent effluent limitations for TIN and TP. The SWRF could be required to meet limits as low as 2 mg/L for TIN and 0.15 mg/L for TP since it discharges into an effluent dominated stream.

WATERSHED ISSUES

U.S. Geological Survey Circular 1284 *State of the Watershed: Water Quality of Boulder Creek, Colorado* published in 2006 recognized the following issues directly related to water quality in the watershed:

- Impacts from population growth and urbanization
- Water quality issues related to septic tank disposal systems
- Pollution associated with non-point sources particularly storm water runoff

- Potential effects of currently unregulated organic compounds including pharmaceuticals, hormones, and residuals from cleaning products, pesticides and herbicides that are not removed by ordinary wastewater treatment processes.

There are numerous groups devoted to studying and improving water quality within the Boulder Creek Watershed including the Boulder Area Sustainability Information Network (BASIN), the National CZO Program Boulder Creek Critical Zone Observatory, and Boulder Creek Watershed Initiative.

BASIN is one of eight Environmental Monitoring for Public Access and Community Tracking (EMPACT) projects funded with an EPA grant. BASIN's stated goals are to improve existing environmental monitoring, provide timely and usable information about the watershed to the public, create a state of the art information management and public access infrastructure, and to develop education and community programs to utilize watershed information (BASIN 2007). The BASIN web site includes stream flow data and snow pack data for the Boulder Creek watershed as well as links to relevant environmental news and legislation.

The Boulder Creek Critical Zone Observatory is a research enterprise sponsored by the University of Colorado at Boulder (University of Colorado at Boulder n.d.). CZO is studying how erosion and weathering control the architecture and function of the Critical Zone, the weathered, hydrologically active near surface environment. Areas of interest include depth to bedrock, character of weathered materials, microbial ecology, stream water inorganic and organic chemistry, and meteorological parameters.

The Boulder Creek Watershed Initiative (BCWI) is a non-profit organization founded in 1997. BCWI's stated goals include: educating the community about the watershed, increasing opportunities for community involvement in monitoring and protecting the watershed, increasing awareness and communication between different cities, agencies, communities, and schools in the watershed, and conducting education seminars (Boulder Creek Watershed Initiative n.d.). The most recent newsletter posted on their web site is dated Winter 2009. BCWI receives funding from a variety of sources including the City of Boulder.

REQUIRED LEVEL OF TREATMENT

The required level of treatment depends on the designated uses of the receiving stream and in-stream dilution during low flows. This section addresses current and future permit limits for both the SWRF and NWRF.

Current Permit Limits

The Town of Erie South WRF (SWRF) is authorized to discharge treated effluent to Coal Creek under Colorado Discharge Permit System (CDPS) Permit Number CO-0045926. The permit was most recently renewed in September 2011. The current effluent limits are presented in Table 7-8. In addition to the limits in Table 7-8, there are monitoring and reporting requirements for a variety of metals.

Table 7-8 Town of Erie South WRF Effluent Limits as of January 2011

Parameter	Discharge Limits		Rationale
	30-Day Average	Daily Maximum	
Flow, MGD	1.6	Report	Design Capacity
$CBOD_5$, mg/l	25	40 (7-day avg.)	State Effluent Regulations
Temp Daily Max (^oC) March-Nov, starting January 1, 2012		Report	
Temp Daily Max (^oC) Dec-Feb, starting January 1, 2012		Report	
Temp Daily Max (^oC) March-Nov, starting January 1, 2012		Report (7-day avg)	
Temp Daily Max (^oC) Dec-Feb, starting January 1, 2012		Report (7-day avg)	
Total Suspended Solids, mg/l	30	45 (7-day avg.)	State Effluent Regulations
Oil & Grease, mg/l	N/A	10	State Effluent Regulations
pH, s.u. maximum-minimum	N/A	6.5-9.0	Water Quality Standards
Total Residual Chlorine, mg/l	0.004	0.02	State Effluent Regulations
E. coli, #/100 ml	126	252 (7-day avg.)	State Effluent Regulations
Nitrite as N (mg/L)	NA	Report	
Total Ammonia (as N), mg/l until April 30, 2017			
January	6.56	Report	Water Quality Standards
February	6.56	Report	
March	7.5	Report	
April	7.5	Report	
May	7.5	Report	
June	7.5	Report	
July	7.5	Report	
August	7.5	Report	
September	7.5	Report	
October	7.5	Report	
November	7.5	Report	
December	6.56	Report	
Total Ammonia (as N), mg/l Starting May 1, 2017			
January	4.4	4.8	Water Quality Standards
February	2.8	2.8	
March	3.1	3.3	
April	2.7	2.9	
May	2.3	2.3	
June	2.4	3.8	
July	2.0	2.0	
August	1.9	1.9	
September	2.1	2.1	
October	3.2	4.4	
November	4.4	9.0	
December	4.4	7.8	

The Town's new North Water Reclamation Facility (NWRF) was constructed in 2010 and became operational in early 2011. The NWRF is authorized to discharge treated effluent to Boulder Creek under Colorado Discharge Permit System (CDPS) Permit Number CO-0048445. The permit was issued in January 2011. The current effluent limits are presented in Table 7-9.

Table 7-9 Town of Erie North WRF Effluent Limits as of January 2011

Parameter	Discharge Limits		Rationale
	30-Day Average	Daily Maximum	
Flow, MGD	1.5	Report	Design Capacity
BOD_5, mg/l	30	45 (7-day avg.)	State Effluent Regulations
Total Suspended Solids, mg/l	30	45 (7-day avg.)	State Effluent Regulations
Oil & Grease, mg/l	N/A	10	State Effluent Regulations
pH, s.u. maximum-minimum	N/A	6.5-9.0	Water Quality Standards
Total Residual Chlorine, mg/l	0.068	0.072	State Effluent Regulations
Fecal coliform, #/100 ml	126	252 (7-day avg.)	State Effluent Regulations
Total Ammonia (as N), mg/l			
January	4.8	4.8	Water Quality Standards
February	2.8	2.8	
March	3.1	3.3	
April	2.7	2.9	
May	2.3	2.3	
June	2.4	3.8	
July	2	2	
August	1.9	1.9	
September	2.1	2.1	
October	3.2	4.4	
November	4.8	9	
December	5	7.8	
Metals, ug/L			2-year Rolling Average
As, TR[1], through 12/31/2014	Report	Report	
As, TR[1], start 01/01/2015	0.12	NA	
As, PD[2]	NA	1276	
Cd, PD, through 12/31/2014	1.9	18	
Cd, PD, start 01/01/2015	1.9	18	0.11
Cr^{+3}, PD, through 12/31/2014	NA	1215	
Cr, TR, through 12/31/2014	NA	Report	
Cr, TR, start 01/01/2015	NA	189	28
Cr^{+6}, D[3]	68	61	11
Cu, PD, through 12/31/2014	75	82	Report
Cu, PD, start 01/01/2015	75	82	4.9
CN, WAD[4], through 12/31/2014	NA	19	Report
CN, WAD, start 01/01/2015	NA	19	2.8
Fe, D, through 12/31/2014	Report	NA	Report
Fe, D, start 01/01/2015	1302	NA	102
Fe, TR	3713	NA	1151
Pb, PD, through 12/31/2014	27	526	Report
Pb, PD, start 01/01/2015	27	526	0.81
Mn, PD	2302	14207	
Mn, D, through 12/31/2014	Report		Report
Mn D, start 01/01/2015	55		18
Hg, T[5], through 12/31/2014	0.062	NA	Report
Hg, T, start 01/01/2015	0.062	NA	0.0093
Ni, PD, through 12/31/2014	591	3249	Report
Ni, PD, start 01/01/2015	591	3249	87
Se, PD, through 12/31/2014	4.6	56	Report
Se, PD, start 01/01/2015	4.6	56	2.4
Ag, PD, through 12/31/2014	6.1	26	Report
Ag, PD, start 01/01/2015	6.1	26	0.51
U, TR, through 12/31/2014	NA	Report	Report
U, TR, start 01/01/2015	NA	113	33
Zn, PD, through 12/31/2014	NA	915	Report
Zn, PD, start 01/01/2015	NA	915	192

[1]TR = Total Recoverable, [2]PD = Potentially Dissolved, [3]D = Dissolved, [4]WAD = Weak Acid Dissociable, [5]T = Total

Future Permit Limits

As noted above, the discharge permit for the Erie SWRF contains compliance schedules for temperature monitoring and effluent ammonia. A rule making hearing for the South Platte River has been scheduled for 2017 to consider adoption of new water quality based nutrient criteria. If adopted, both the SWRF and NWRF *may* have to meet discharge limits for total nitrogen low enough to maintain a stream standard of 2.0 mg/L total nitrogen (Table 7-8). As of June 2011, the lowest effluent limits that could be imposed on a non-exempt facility are 1.0 mg/L phosphorus as P and 15 mg/L total inorganic nitrogen as N. Lower limits are a possibility in the future if water quality goals are not met by the technology based effluent limitations on nutrients set forth in Regulation 85.

WATERSHED CHARACTERISTICS AND ISSUES

U.S. Geological Survey Circular 1284 *State of the Watershed: Water Quality of Boulder Creek, Colorado* published in 2006 presents a detailed description of the Boulder Creek Watershed (Murphy 2006). The following draws heavily on the information in that document.

The Boulder Creek Watershed is approximately 447 square miles in area and is located in the Front Range of the Colorado Rocky Mountains, east of the Continental Divide. The watershed includes all the land area that drains water into Boulder Creek. The watershed has great variation in geology, climate, and land cover. Tributaries of Boulder Creek include North, Middle, and South Boulder Creeks, Fourmile Creek, Coal Creek, and Rock Creek, along with several smaller streams. These streams generally flow from west to east. Boulder Creek empties into Saint Vrain Creek which empties into the South Platte River. The communities of Boulder, Louisville, Lafayette, Erie, Superior, and Nederland are in the watershed, along with parts of Arvada, Broomfield, and Frederick. In 2000, about 185,000 people lived in the Boulder Creek Watershed. The last major flood down Boulder Creek was in May of 1894 when nearly 60 hours of continuous rain triggered a 100-year flood event (BASIN 2007).

The Boulder Creek Watershed lies within two physiographic provinces. The mountainous upper watershed is part of the Southern Rocky Mountains Province and is characterized by deep, steeply sloping valleys. The flatter, lower watershed is part of the Colorado Piedmont Section of the Great Plains Province and slopes gently to the northeast. The Town of Erie is located in the far eastern end of the watershed (Figure 7-2). This area is characterized by grassland, agriculture, and urban/suburban development. Coal mining was the original principal activity in the area followed by oil and natural gas extraction. Today sand and gravel is mined along Boulder Creek and agricultural activities include grazing and growing alfalfa, wheat, corn and barley. Conversion of grasslands and agricultural lands to urban/suburban development has increased the past 30 years. Streams in the lower Boulder Creek Watershed are heavily affected by this human activity and urbanization. While reservoirs have increased in number and sand and gravel pits have filled with water forming ponds, natural stream flows have been diminished by diversions for urban and agricultural uses and urban runoff and discharges from wastewater treatment facilities (WWTF) affect flow and water quality.

The water quality of Boulder Creek downstream from the City of Boulder 75th Street wastewater treatment plant is strongly influenced by the plant effluent. The concentrations of pollutants in the

effluent are regulated by CDPHE to minimize detrimental effects, but there is still a significant impact. The degree of impact is directly related to the stream flow, with the effect being greater when flows are low and are reduced when higher stream flow provides more dilution. Diversions downstream from the plant remove much of the wastewater chemical load from the creek. The stream flow is replenished by irrigation return flows, flow from tributaries, and ground water. These inflows reduce the concentration of nutrients, metals and other wastewater effluent constituents but can increase the concentrations of others such as sodium, magnesium, and sulfate (Murphy 2006).

Originating on the eastern flank of Thorodin Mountain near the town of Wondervu, Coal Creek carves a canyon that empties onto the plains west of Rocky Flats, then swings east and north through the Town of Superior, Louisville, Lafayette and Erie before joining Boulder Creek. Coal Creek drains about 14% of the Boulder Creek Watershed. Coal Creek merges with Boulder Creek about eight miles downstream from the Boulder WWTP. Coal Creek receives wastewater effluent from Erie, Lafayette, Louisville, and Superior WWTPs, which are permitted to discharge a total of 10.8 million gallons per day of effluent to Coal Creek or its tributary, Rock Creek. Although stream flow may be affected seasonally by agricultural return, the natural flow of Coal Creek is small. As a result, the stream is normally effluent dominated, composed almost entirely of wastewater effluent when it enters Boulder Creek. With minimal dilution, concentrations of dissolved solids, nutrients, and organic contaminants in Coal Creek often are elevated relative to Boulder Creek and cause an increase in these contaminants in Boulder Creek.

Aquatic Life and Habitat

Natural conditions in the Boulder Creek Watershed can be harsh for fish and other aquatic life. Stream flow originates primarily as snowmelt and thus varies widely both seasonally and annually. In the lower watershed, streams are slower moving and subjected to intense sunlight, causing temperature, dissolved oxygen, and pH to vary drastically, particularly in late summer (Murphy 2006). These conditions lead to a relatively low number of native fish species able to survive in the watershed. Humans have substantially altered the natural hydrologic regime by diverting water from streams and building reservoirs, straightening stream channels, decreasing flow during high-flow periods, increasing flow during low-flow periods, and causing daily and hourly flow variations. In addition, nutrient loading is higher due to wastewater effluent, habitat has been fragmented, and non-native fish have been introduced.

Historically, much of lower Boulder Creek was channelized for flood control. Channelization removes pools and riffles, which are important habitat for fish. In the 1980s the City of Boulder restored much of Boulder Creek within the city for recreation, esthetics, and fish habitat. Fish habitat structures were built, banks were stabilized, and riparian areas were revegetated all of which had positive effects on the stream habitat.

Non-native rainbow trout, brown trout, and brook trout were stocked in Boulder Creek soon after settlement and are now the principal fish species in the mountain streams of the watershed and within the city of Boulder (Murphy 2006). These fish outcompete the native greenback cutthroat trout, a federally listed threatened species. In lower Boulder Creek, native white suckers and fathead minnows, along with non-native common carp, are the most abundant species. Studies of Coal Creek found that native creek

chub and fathead minnow were most abundant (Murphy 2006). These fish species dominate because they tolerate the extreme variations of temperature, dissolved oxygen, and turbidity found in the streams of the lower watershed.

Several non-native species are threatening ecosystems in Boulder Creek, including the New Zealand mudsnail and the Eurasian watermilfoil, an aquatic plant (Murphy 2006). These species were accidentally introduced, have no natural predators in the watershed, and spread rapidly. They negatively affect aquatic ecosystems by outcompeting native species and by reducing biodiversity. A native species of algae, the diatom *Didymosphenia geminata*, is also affecting Boulder Creek. This diatom was once rare and restricted to pristine lakes and streams. In recent years, however, it has formed excessive growths in Boulder Creek, as well as many streams of Western North America. Loss of native species and biodiversity can lead to a decline in population and diversity of fish, because their food supply has been affected.

Existing Facilities

Figure 7-3 shows sewage treatment plants and water supply intakes within a five mile radius of the Town of Erie's wastewater treatment facilities. Table 7-10 lists six nearby wastewater dischargers with the potential to affect the receiving waters of the Town of Erie wastewater treatment facilities.

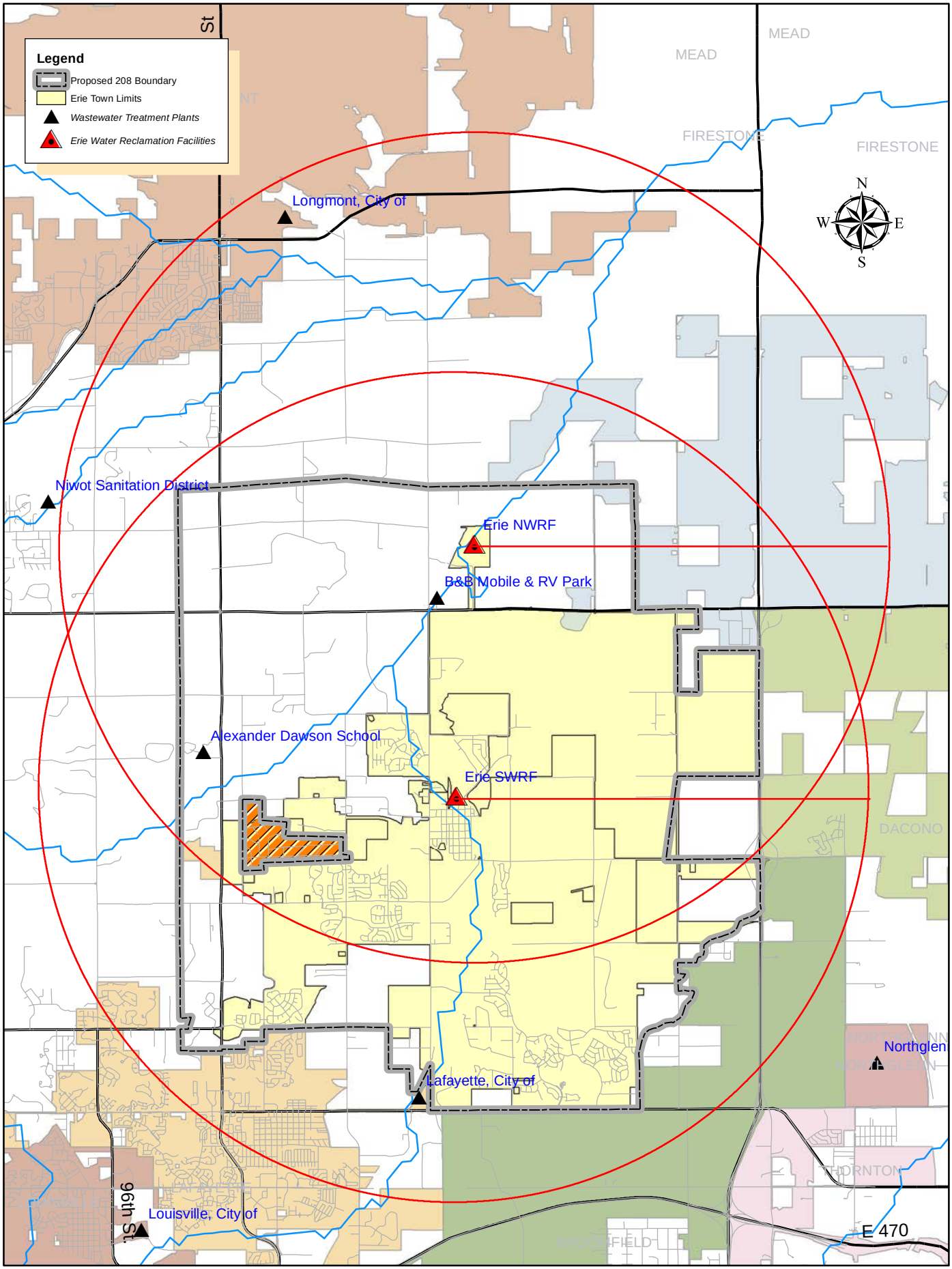
Table 7-10 Permitted Wastewater Dischargers

Discharger	*Permit Number*	*Discharge Location*	*Permitted Capacity*
City of Boulder WWTF	CO-0024147	Boulder Creek	25 mgd
City of Louisville WWTF	CO-0023078	Coal Creek	3.4 mgd
Superior Metro District	CO-0043010	Rock Creek	2.4 mgd
City of Lafayette WWTF	CO-0023124	Coal Creek	4.4 mgd
Alexander Dawson School	CO-0045021	Groundwater	0.3 mgd
B&B Mobile and RV Park	COG-588107	Boulder Creek	0.015 mgd

In the Boulder Creek Watershed, there are thirteen other wastewater treatment facilities with NPDES permits (Table 17): Alexander Dawson School (0.3 MGD), B&B Mobile and RV Park (0.015 mgd), Boulder Mountain Lodge (0.0052 MGD), Greenbriar Restaurant (0.006 MGD), Lake Eldora Water and Sanitation District (0.03 MGD), Town of Nederland (0.189 MGD), PSCO-Valmont (0.045 MGD), Red Lion Inn (0.009 MGD), Sacred Heart of Mary (0.437 MGD), San Lazaro Mobile Home Park (0.13 MGD), San Soucci Mobile Home Park (0.018 MGD), Mountain Shadows Montessori, Boys and Girls Club, Dakota Ranch (0.001 MGD), Gold Lake Ranch (0.0054 MGD) and Seventh Day Adventist (0.04 MGD) (Denver Regional Council of Governments June 1998).

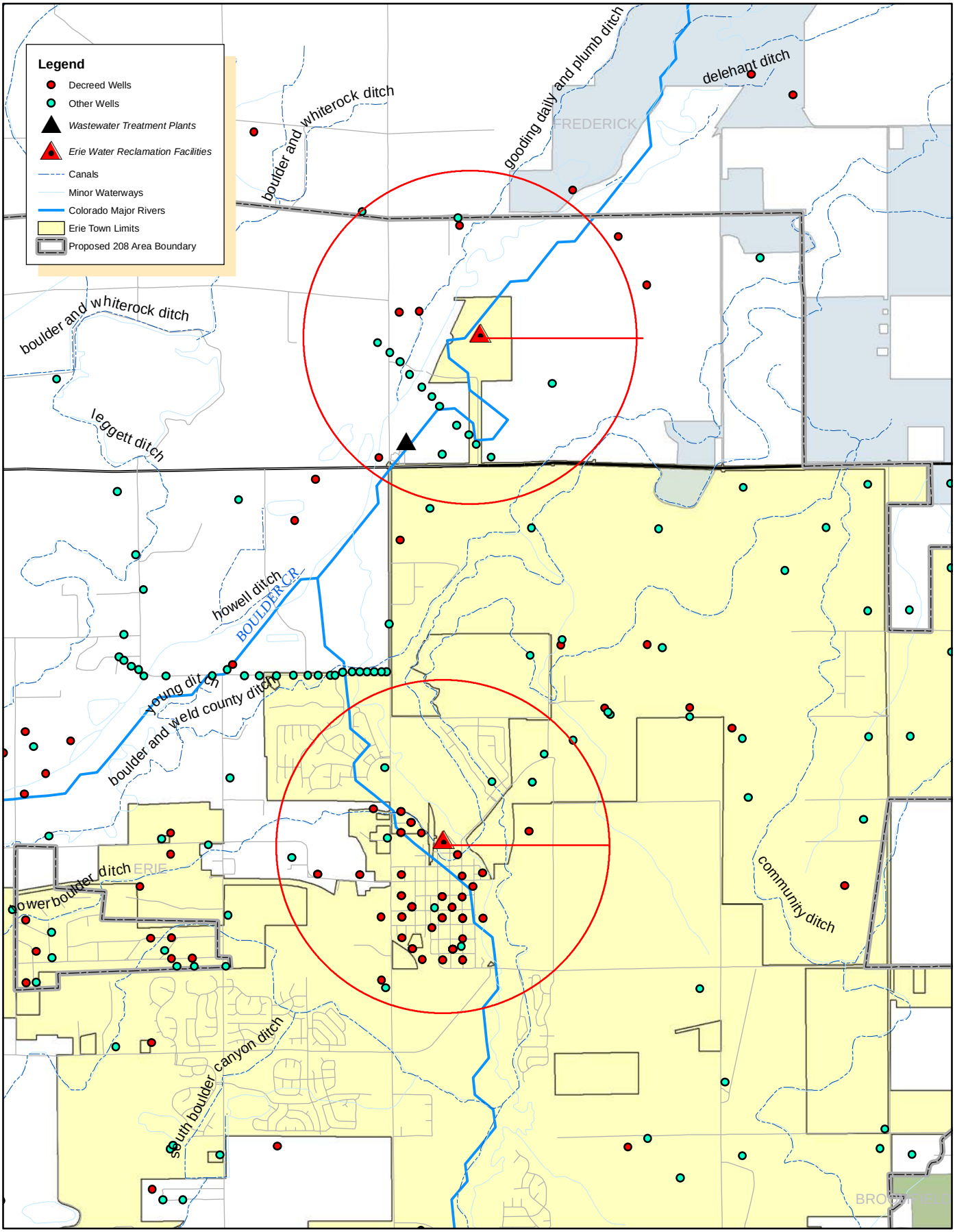
Potable Water Wells

Figure 7-4 shows potable water well locations and an approximate indication of the topography within a one-mile radius of each of the Town of Erie's wastewater treatment plants. Wells under the direct influence of surface water (GWUDI) can be impacted by wastewater treatment plant discharges.



Circles show 5-mile radius from each reclamation facility.

Figure 7-3 Wastewater Treatment Plants Near Town of Erie



Circles show 1-mile radius from each reclamation facility.

Figure 7-4 Wells within 1-Miles of SWRF and NWRF

Water Diversions

Boulder Creek and its tributaries are part of a complex water-management system. Diversions remove water from streams for municipal, industrial, and agricultural use. Water is brought into and out of the watershed by trans-basin diversions, and reservoirs store water for a reliable year-round supply. Wastewater treatment plants contribute treated effluent that can account for a substantial portion of flow in streams in the lower watershed during low-flow conditions. There are 1,617 diversions in the St. Vrain Watershed including 49 groundwater diversions, 166 seeps, 326 springs, and 12 transbasin diversions. Transbasin diversions include water from the Colorado Big Thompson (CBT) Project and Windy Gap received at the Erie Water Treatment Plant (WTP), Louisville WTP, Superior WTP, and Boulder Reservoir.

Figure 7-5 maps diversions in the Boulder Creek watershed that are within a 5-mile radius of either the SWRF or NWRF. Diversions on Boulder Creek are highlighted in lime green while diversions on Coal creek are highlighted in bright pink. Diversions along Boulder and Coal Creeks are labeled with the structure identification number assigned by the Colorado Decision Support Systems (CDSS). Diversions that do not draw water from either Boulder or Coal Creek are shown smaller and in dark green. Table 7-11 identifies the fifty-one diversions on Coal and Boulder Creeks.

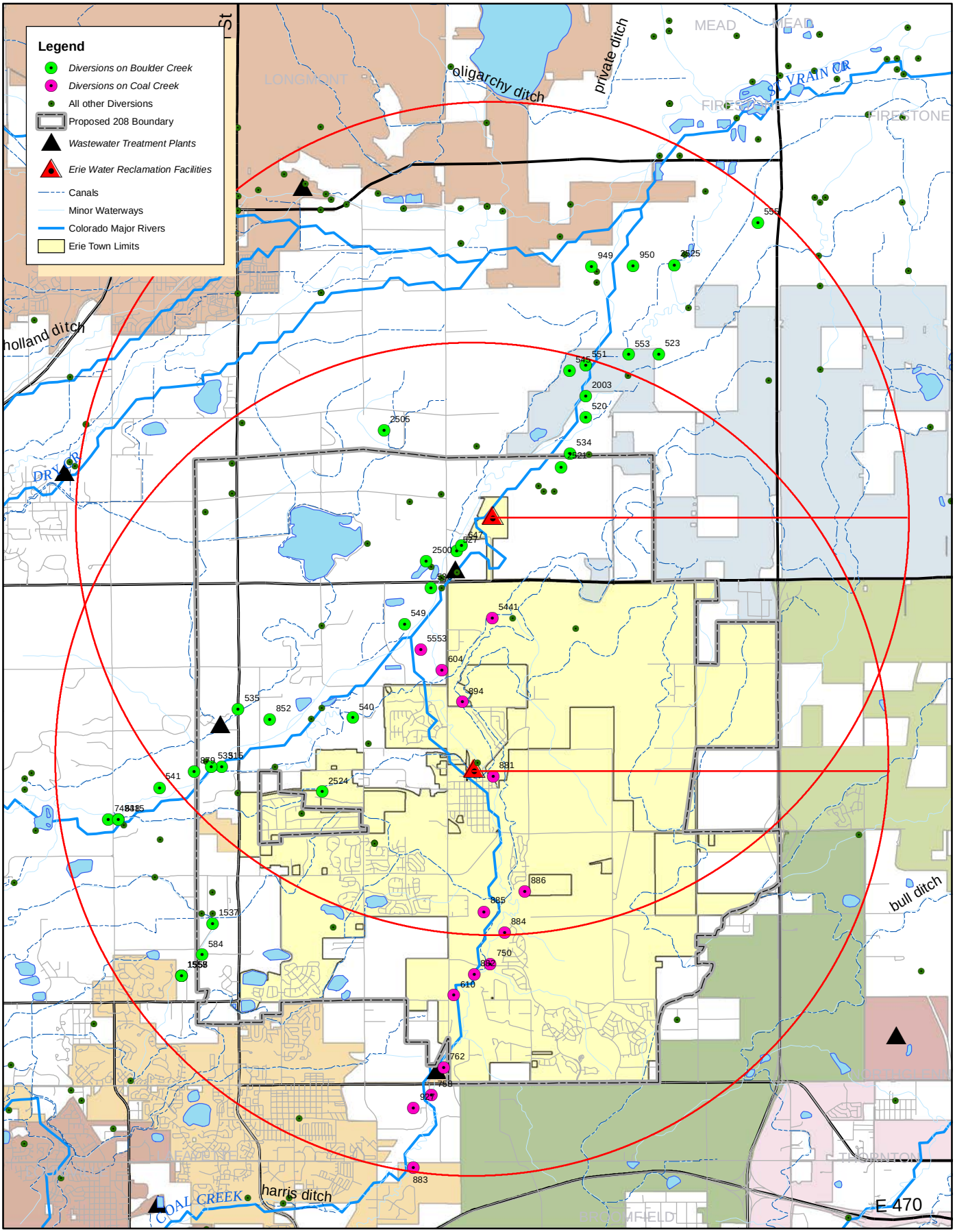
Table 7-11 Water Diversions on Boulder and Coal Creeks

ID	Diversion Name	Water Source	Latitude	Longitude
515	Boulder Weld Cty Ditch	Boulder Creek	40.055593	-105.106478
520	Carr Tyler Ditch	Boulder Creek	40.116072	-105.023727
523	Delehant Ditch	Boulder Creek	40.126913	-105.007096
527	Godding Dailey Plumb Ditch	Boulder Creek	40.092885	-105.053086
533	Houck 1 Ditch	Boulder Creek	40.055604	-105.108873
534	Houck 2 Ditch	Boulder Creek	40.109706	-105.027301
535	Howell Beasley Ditch	Boulder Creek	40.065548	-105.102736
536	Howell Ditch	Boulder Creek	40.086517	-105.058982
538	Lower Boulder Ditch	Boulder Creek	40.046478	-105.129957
540	Martha M Mathews Ditch	Boulder Creek	40.063977	-105.076819
541	Mathews Ditch	Boulder Creek	40.051975	-105.120614
545	N K Smith Tyler Ditch	Boulder Creek	40.124149	-105.027331
549	Plumb Ditch	Boulder Creek	40.080234	-105.064974
551	Rural Ditch	Boulder Creek	40.125077	-105.023661
553	Smith Emmons Ditch	Boulder Creek	40.126912	-105.013895
555	Taylor Ditch	Boulder Creek	40.149706	-104.984457
584	Bull Head Gulch Irr Sys	Boulder Creek	40.022865	-105.111154
604	Archuleta Seepage Ditch	Coal Creek	40.072161	-105.056574
610	Erie Coal Cr Ditch	Coal Creek	40.015629	-105.054235
647	Armstead Ditch	Boulder Creek	40.093809	-105.051927
748	Lafayette Pl Bldr Cr Dup	Boulder Creek	40.046467	-105.132307

ID	Diversion Name	Water Source	Latitude	Longitude
750	Vista Ridge Pump & PL	Coal Creek	40.02104	-105.04597
758	Beck Pump Station	Coal Creek	39.99828	-105.05942
762	Lafayette Ditch No. 4	Coal Creek	40.003046	-105.056667
852	Dunlavy Runoff	Boulder Creek	40.06372	-105.095601
879	Lafayette Boulder C PL 2	Boulder Creek	40.054749	-105.112812
881	Lafayette Ditch 2	Coal Creek	40.053709	-105.045081
882	Lafayette Ditch 3	Coal Creek	40.019235	-105.049569
883	Lafayette Ditch 5	Coal Creek	39.985716	-105.063556
884	Lafayette Ditch 6	Coal Creek	40.026455	-105.042658
885	Lafayette Ditch 7	Coal Creek	40.03012	-105.047274
886	Lafayette Ditch 8	Coal Creek	40.033646	-105.038114
894	Lewis Seepage Ditch	Coal Creek	40.066631	-105.051988
927	Wakeka Ditch 1	Coal Creek	39.996051	-105.063462
949	Longmont Op Lakes PL 1	Boulder Creek	40.142228	-105.022202
950	Longmont Op Lakes PL 2	Boulder Creek	40.142351	-105.012768
1537	Bull Head Gulch PL	Boulder Creek	40.028282	-105.108781
1555	Kugeler Pipeline 1	Boulder Creek	40.019256	-105.11584
1557	Kugeler PL 1 Alt PT 1	Boulder Creek	40.019256	-105.11584
1558	Kugeler PL 1 Alt PT 2	Boulder Creek	40.019256	-105.11584
2003	Rural Ditch In Ditch Recharge Area	Boulder Creek	40.11969	-105.02371
2305	Erie Sewer	Waste	40.05607	-105.04864
2415	Boulder Creek	Boulder Creek	40.04648	-105.12996
2500	Bckey Farm Aug	Boulder Creek	40.091102	-105.060036
2505	Oxford Farm Aug 80-009	Boulder Creek	40.113919	-105.069449
2521	Shad Pit M-2001-094 Aug	Boulder Creek	40.10742	-105.02933
2524	Meadow Creek Farm Pit M-1973-017 Aug	Boulder Creek	40.05113	-105.08382
2525	Duckworth Pit M-2003-091 Aug	Boulder Creek	40.14246	-105.00347
2702	ERIE WTP	Transbasin Water	40.027547	-105.081436
5441	James A Lewis Drain Line	Coal Creek	40.081203	-105.04509
5553	Lewis Pump Plant	Coal Creek	40.075762	-105.061348

Impaired Waters

Water bodies with ambient water quality below the assigned water quality standards are identified as impaired waters and included on the State's 303(d) List of Impaired Waters. For impaired waters CDPHE establishes a maximum pollution load that the water body can assimilate and still attain standards. This maximum pollution load is referred to as the Total Maximum Daily Load or TMDL. CDPHE quantifies the TMDLs and allocates the allowable loads among the contributing sources both point and non point (wasteload allocation). The wasteload allocation serves as the basis for setting effluent limits for discharges such as wastewater treatment plants.



Circles show 5-mile radius from each reclamation facility.
Numbers indicated structure ID number given by
State Water Engineer. For key, see text.

Figure 7-5 Water Diversions within 5-Miles of SWRF and NWRF

Both stream segments COSPBO07b and COSPBO10 are listed on the current Colorado 303(d) list effective April 30, 2010 as impaired waters for *E. coli*. As a result, the chronic discharge limits for *E. coli* are limited to the in-stream standard of 126 per 100 ml for both of Erie's treatment plants.

SECTION 8
ALTERNATIVES ANALYSIS

This section evaluates treatment process alternatives to provide future capacity at both the north and south water reclamation facilities and to meet future requirements for nutrient removal (nitrogen and phosphorus). The Town is currently required to nitrify at both WRFs, but is not required to remove either nitrate or phosphorus. The regulatory trend across Colorado has been towards lower total inorganic nitrogen and phosphorus limits. The nutrient criteria rule making hearing on May 14, 2012 adopted statewide nutrient standards and interim effluent limits for total nitrogen and phosphorus. Lower total nitrogen and phosphorus limits are a possibility for the Town of Erie as early as 2022.

INFLUENT FLOWS AND LOADS

Influent flows and loads are discussed in detail in Section 4. For convenience, the flows and loads selected for planning purposes are summarized in Tables 8-1, 8-2, and 8-3 below. For planning purposes, the years 2025 and buildout were selected.

Table 8-1 Selected Future Flows for Planning Purposes

Parameter	2025	Buildout
Annual Average Flow Generation Rate, gpcd	90	90
Maximum Month Flow Generation Rate, gpcd	118	118
Annual Average Daily Flow, mgd	3.70	6.26
Maximum Month Flow, mgd	4.80	8.12
Peak Day Flow, mgd	6.30	10.66
Maximum Month Flow to SWRF, mgd	1.60	1.60
Annual Average Flow to SWRF, mgd	1.23	1.23
Peak Hour Peaking Factor for SWRF	3.52	3.52
Peak Hour Flow to SWRF, mgd	4.35	4.35
Maximum Month Flow to NWRF, mgd	3.20	6.52
Annual Average Flow to NWRF, mgd	2.46	5.02
Peak Hour Peaking Factor for NWRF	3.14	2.79
Peak Hour Flow to NWRF, mgd	7.74	14.02

Table 8-2 Projected Future Average and Maximum Month Flows

Date	Population	ADF	MM
2010	18,135	1.63	2.14
2015	26,525	2.39	3.13
2020	33,525	3.02	3.96
2025	40,680	3.66	4.80
2030	49,625	4.47	5.86
Buildout	68,820	6.19	8.12

Table 8-3 Future Loads for Town of Erie from Historic Generation Rates

Year	Population1	BOD, ppcd		TSS, ppd		NH_3-N, ppd	
		AA	MM	AA	MM	AA	MM
2010	18,135	2,902	3,773	3446	4135	326	414
2015	26,525	4,244	5,517	5,040	6,048	477	606
2020	33,525	5,364	6,973	6,370	7,644	603	766
2025	40,680	6,509	8,462	7,729	9,275	732	930
2030	49,625	7,940	10,322	9,429	11,315	893	1,134
Buildout	68,820	11,011	14,314	13,076	15,691	1,239	1,574

AAF = Annual Average Flow, MM = Maximum Month Average, AA = Annual Average
AA per capita generation rates utilized are 0.16 ppd BOD, 0.19 ppd TSS, and 0.018 ppd NH_3N
Maximum Month Peaking Factors Applied are BOD = 1.30, TSS = 1.20, Ammonia = 1.27

COST ESTIMATE METHODOLOGY

Cost estimates are often prepared at several points during project planning and design. The expected level of accuracy is directly proportional to the level of engineering effort applied and known details. Each category of estimate must be carefully planned from conceptual, facilities plan, preliminary design, and final engineer's estimate of probable cost.

A common misunderstanding in budget level cost estimates is the percent contingency. Many owners feel that a contingency represents monies that may or may not be spent depending on unforeseen factors. In actuality, the contingency represents portions of a future project that have not been detailed within the planning budget. These details such as valves, electrical conduit, panel covers, lighting, etc. will be included in the final design even though they are not explicitly listed in the cost estimates. Final project costs depend on a variety of conditions that are not under the control of the owner, engineer, or contractor including the process of raw materials and equipment and site conditions.

The construction cost estimates presented in this section are based on the installed cost of materials and equipment. The material and equipment estimates were developed with cost data from recently completed projects, published literature, and manufacturer's quotations. Cost factors shown in Table 8-4 have been added to the materials and equipment costs to develop total project costs for each alternative. No adjustment has been made for inflation.

Table 8-4 Cost Factors Used to Develop Estimated Total Project Costs

Cost Factor	Percent of Construction	Description
Contingencies	10	Unknown conditions and conflicting utilities
Planning and Engineering	15	Planning, design, construction observation, training, O&M manual, startup services

The construction cost estimates represent order-of-magnitude costs for each alternative. The American National Standards Institute (ANSI) and the American Society of Civil Engineers (ASCE) define order-of-magnitude estimates as "an approximate estimate made without detailed engineering data". Cost estimates were prepared by Farnsworth Group and Indigo Water Group based on vendor supplied information and experience with past projects.

NORTH WATER RECLAMATION FACILITY

The North WRF was constructed in 2010 and became operational in early 2011. The NWRF is an Integrated Fixed Film Activated Sludge System (IFAS) with an initial treatment capacity of 1.5 MGD and 3,233 ppd of BOD; expandable to 3.6 MGD and 7,750 ppd of BOD. To expand to 3.6 MGD, the following improvements will be required:

- Addition of a second mechanical bar screen.
- Addition of a fourth influent pump.
- Increase percent fill of floating Kaldnes media in the aeration basins from 25% to 65% in the first aerobic zones and to 60% in the second aerobic zones.
- Construction of a third secondary clarifier.
- Addition of a second scum pump for the secondary clarifiers.
- Addition of a second return activated sludge (RAS) pump.
- Addition of a second internal recycle pump.
- Addition of a second sludge dewatering pump.
- Addition of a parallel solids dewatering process including floc tank, polymer addition system, rotary screen thickener, and screw press.

The expansion to 3.6 MGD may be needed before 2015. The SWRF has a rated capacity of 1.6 MGD and the NWRF has a capacity of 1.5 MGD for a total capacity of 3.1 MGD (maximum month flow). The State WQCD requires that municipalities be in the planning stages for their next expansion when they reach 80% of the hydraulic or organic load capacity of their treatment facilities and to be under construction when 95% of capacity is reached. The 80% and 95% hydraulic capacity triggers for the Town of Erie are 2.48 MGD and 2.95 MGD, respectively. The combined influent flows for both the SWRF and NWRF may reach 3.13 MGD by 2015.Expansion timing is dependent on population growth and per capita generation rates for flow and BOD.The NWRF was brought on-line in February 2011. The SWRF was taken off line and all wastewater flows were directed to the NWRF at that time. The NWRF is currently operating at 73 percent of its permitted hydraulic capacity. The NWRF was designed using conservative assumptions and has performed better than conservative design calculations predicted. The NWRF capacity could be increased from 1.5 to 1.75 or 2.0 mgd through a paper rerating based on actual operating data and/or Biowin modeling. A paper rerating would increase the timeline for construction of additional capacity at the NWRF and SWRF. A paper rerating will require a Site Application Amendment.

A second physical expansion of the NWRF may be needed sometime between the years 2025 and 2030. The NWRF can be expanded by adding additional treatment trains to mirror the existing design. Costs associated with expansion of the NWRF were previously provided by Burns and McDonnell as part of the NWRF process design report.

The NWRF was designed to remove ammonia, nitrate, and phosphorus. Phosphorus removal will be done biologically through an anaerobic zone. Biological phosphorus removal can reliably remove phosphorus down to 0.8 mg/L as P. Phosphorus limits are expected to tighten significantly throughout

Colorado. The Town of Erie could receive effluent permit limits as low as 0.05 mg/L as P as early as 2022. Chemical precipitation with either alum or ferric chloride followed by tertiary filtration will be required to remove phosphorus below 0.8 mg/L as P. Chemical addition can be done in the secondary clarifier wet wells or after clarification followed by filtration with the existing cloth media filter. Chemical addition plus filtration can reduce effluent P to less than 0.05 mg/L. Additional filter capacity will be needed to treat future flows. Another alternative is to use an adsorptive process like the BluePro filter which adsorbs phosphorus with an iron-oxide coated sand. The BluePro process can reduce effluent P to 0.01 mg/L.

The Town has the ability to divert flows from the SWRF to the NWRF and to control the percentage of flow received by each facility. In addition, the Town has selected extremely conservative per capita wastewater generation rates of 90 gpcd (average) and 118 gpcd (maximum month) in anticipation of attracting future industrial and commercial users to the service area. Historic per capita flow generation rates over the past ten years have averaged 62 gpcd (average) and 67 gpcd (maximum month). Moreover, historic per capita flow generation rates have been very consistent with a range of 62 to 69 gpcd (average) and 67 to 73 gpcd (maximum month). Conservative planning criteria and the ability to move wastewater between treatment facilities gives the Town a high degree of flexibility for planning future expansions.

One possible expansion scenario is shown in Figure 8-1. Two potential flow curves are shown on the graph based on maximum month per capita wastewater generation rates of 70 gpcd (historic) and 118 gpcd (selected for planning purposes). With this scenario, the SWRF maintains a rated capacity of 1.6 mgd and the NWRF is expanded. If maximum month flows increase to 118 gpcd with the influx of industrial and commercial users, the NWRF will need to be expanded between 2015 and 2020, again between 2025 and 2030, and once last time as the facility approaches buildout. A rerating would be done prior to 2015. If maximum month flows remain near the historic average of 70 gpcd, the NWRF will not need to be expanded until the year 2025 and the second expansion would not be needed until sometime after 2030 if at all. Under the lower flow scenario, the second expansion may not be necessary to accommodate the projected buildout population.

Erie may elect to add capacity at the SWRF instead of the NWRF. Alternatives for the SWRF are discussed within this Chapter. Regardless of which facility is expanded or if both facilities are expanded, the total amount of capacity required will be dictated by population growth and per capita generation rates. As noted previously, if per capita generation rates are consistent with historic trends, the schedule for expansion will stretch over a much longer period of time and the total amount of capacity needed at buildout will be substantially less.

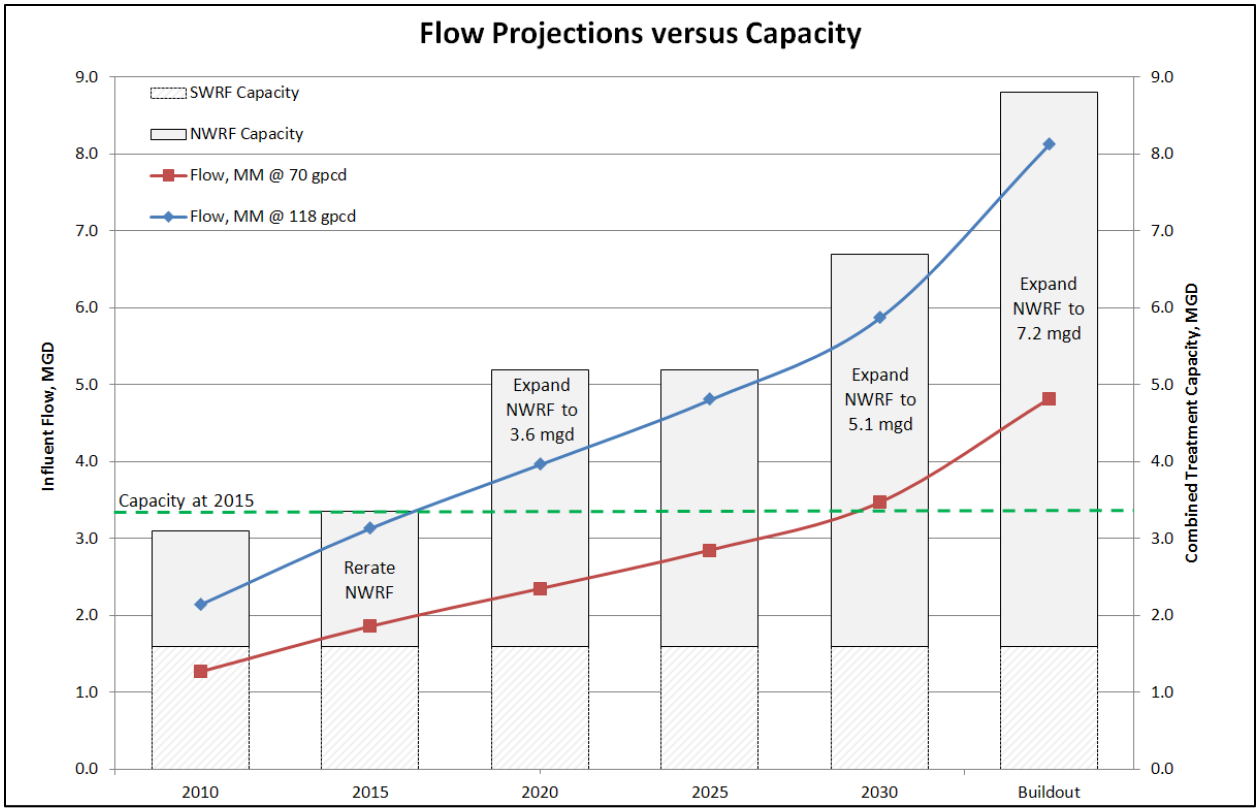


Figure 8-1 Expansion Scenario Timing

SOUTH WATER RECLAMATION FACILITY

The SWRF has a rated capacity of 1.6 MGD and 3,870 ppd of BOD. For a variety of reasons discussed throughout this WUP, the SWRF will be required to remove ammonia to lower levels in the future as well as both nitrate and phosphorus. The SWRF will not be able to meet future permit limits as it is currently configured.

The new discharge permit for the SWRF was issued by the WQCD on August 16, 2012. The permit requires the SWRF to meet new, more restrictive daily maximum and monthly average ammonia effluent limits beginning May 1, 2017. The permit does not include a phosphorus limit. The proposed compliance schedule in the draft permit allows approximately 5 years to implement a solution.

Consistently meeting the low daily maximum and monthly average ammonia limits is problematic even with the existing high level of treatment at the SWRF. Based on final effluent data collected from January 2003 through October 2009 (371 data points), the proposed daily maximum ammonia limits would have been exceeded on 72 occasions, and the 30-day average ammonia limits would have been exceeded on 12 occasions.

SWRF ALTERNATIVES ANALYSIS

The SWRF site is constrained by a canal to the west, by the old lagoon cells to the north, by a road to the east, and by a canal and private residences to the south. The nearest residences are approximately 600 feet from the SWRF property line. The Town has expressed a strong preference that future improvements remain within the existing site footprint. Two technologies that could potentially be used to increase the nitrogen removal capacity of the SWRF without constructing additional aeration basins or clarifiers are membrane bioreactors (MBR) and integrated fixed-film activated sludge (IFAS).

The SWRF aeration basins, digesters, and biosolids holding tanks are situated as two parallel treatment trains as shown in Figure 8-2. Concrete walls divide the two larger tanks to form each of the individual processes. The basin dimensions and volumes are presented in Table 8-5. If the digesters and solids handling tanks were converted to become part of the secondary treatment process, a maximum volume of 1.19 MG would be available for the secondary treatment process.

Table 8-5 Dimensions and Volumes of Existing Basins

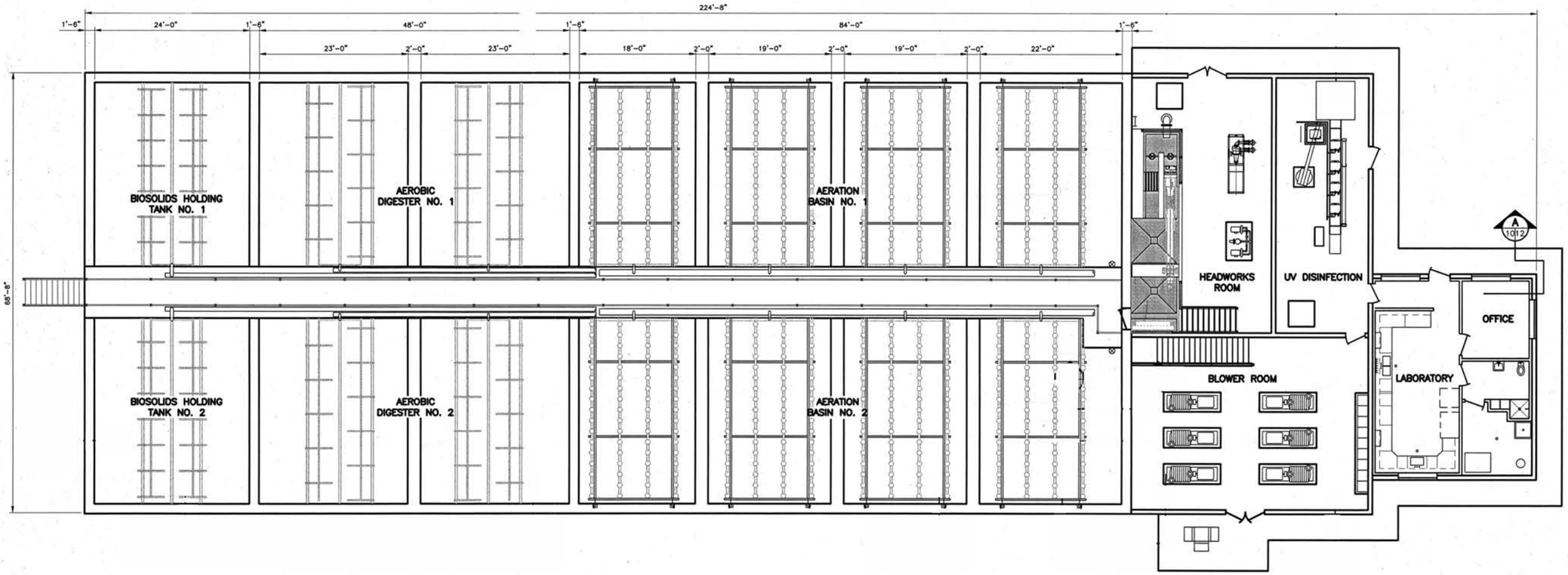
Parameter	Value
Aeration Basin Dimensions, each, feet	84 by 32 by 16.5
Aeration Basin Volume, each, MG	0.32
Aeration Basin Volume, total, MG	0.64
Aerobic Digester Dimensions, each, feet	48 by 32 by 17
Aerobic Digester Volume, each, MG	0.185
Aerobic Digester Volume, total, MG	0.37
Biosolids Holding Tank Dimensions, each, feet	24 by 32 by 17
Biosolids Holding Tank Volume, each, MG	0.092
Biosolids Holding Tank Volume, total, MG	0.184
Total Available Basin Volume	1.19 MG

The following alternatives were evaluated for the SWRF:

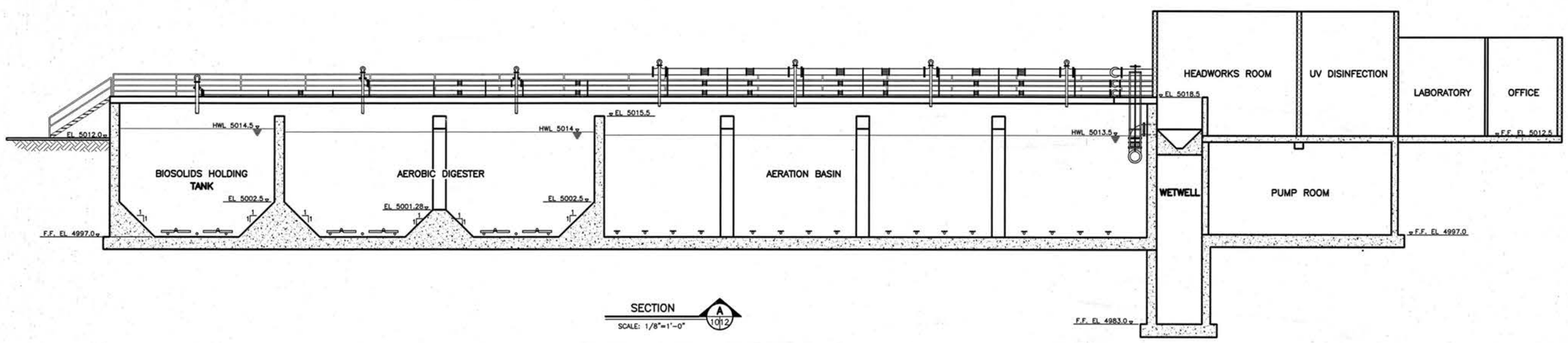
- No action coupled with rerating to a lower capacity. Lowering the rated capacity will decrease the organic and hydraulic loading rate to the SWRF and increase ammonia removal capacity. Effluent ammonia concentrations are expected to be below 1 mg/L as N. This alternative will not remove nitrate or phosphorus.

- Convert the SWRF from conventional to extended aeration activated sludge by converting the aerobic digesters and solids holding tanks to additional aeration basins. With this alternative, waste activated sludge (WAS) would be placed into the NWRF interceptor for treatment at the NWRF. The capacity of the solids processing equipment at the NWRF would need to be increased. Existing pumps and blowers would be reused. To achieve effluent ammonia concentrations consistently below 1 mg/L as N, the SWRF would need to be derated slightly to 1.4 mgd. This alternative will not remove nitrate or phosphorus.

- Convert the SWRF from conventional to extended aeration activated sludge by constructing additional aeration basins. Existing pumps and blowers would be reused. The aerobic digesters would be preserved under this alternative; however, to maintain a rated capacity of 1.6 mgd, some



Plan View



Profile View

Figure 8-2 SWRF Existing Process Layout

type of solids thickening would likely be needed prior to aerobic digestion. Effluent ammonia is expected to be below 1 mg/L as N. This alternative will not remove nitrate or phosphorus.

- Transform the SWRF into a membrane bioreactor plant. With this alternative, the secondary clarifiers could be repurposed for influent flow equalization and a new building and tanks would be constructed to house the membranes. Additional blowers and pumps would be required in addition to existing equipment. This alternative will remove ammonia to less than 1 mg/L as N and remove total nitrogen to less than 10 mg/L as N. This alternative will not remove phosphorus. The aerobic digesters would be preserved under this alternative; however, to maintain a rated capacity of 1.6 mgd, some type of solids thickening would likely be needed.

- Convert the SWRF activated sludge basins to Integrated Fixed Film Activated Sludge (IFAS). This alternative would mirror the recently completed NWRF. The existing aeration basins would be retrofitted to accommodate the IFAS equipment. Additional blower capacity may be required. The aerobic digesters would be preserved under this alternative; however, to maintain a rated capacity of 1.6 mgd, some type of solids thickening would likely be needed. This alternative will remove ammonia to less than 1 mg/L as N and total nitrogen below 10 mg/L as N.

If the SWRF is modified or expanded, the Water Quality Control Division (WQCD) may require both new and existing biological processes be completely covered to mitigate odors and aerosols due to the proximity of residences. Covering the biological processes will significantly increase construction costs.

The SWRF may be required to remove phosphorus to low levels as early as 2022. Phosphorus removal can be accomplished biologically or chemically. Many facilities use a combination of biological and chemical treatment methods followed by filtration to achieve phosphorus concentrations as low as 0.05 mg/L. The following alternatives were evaluated for the Erie SWRF:

- Chemical precipitation of phosphorus followed by tertiary filtration. Three different tertiary filters were evaluated including:
    - Hydrotech cloth media disc-filter by Kruger
    - Ultrafiltration by Miller Leaman
    - Volcano continuous downflow by Lighthouse Filter
- Adsorption of phosphorus with the BluePro filter.

The SWRF location has two, unused lagoon pond cells from the original treatment plant. These ponds could be repurposed for reuse water storage. The ponds would need some earthwork to remove vegetation, reshape the slopes, and prepare the surface for a synthetic liner. Reuse quality water can be produced with a membrane bioreactor plant or by either the activated sludge or IFAS treatment processes followed by tertiary filtration.

The SWRF does not have any solids thickening or dewatering processes. Operations staff thicken solids in the aerobic digesters by allowing the solids to settle and decanting supernatant back to the plant headworks. The finished biosolids are between 1.5 and 2.5 percent solids. Higher solids concentrations could be achieved with longer settling times and/or chemical addition. Longer settling times generates odors that are unacceptable to both the Town and nearby residents. The potential for installation of dewatering equipment near the aerobic digesters was evaluated to eliminate the need for decanting and to increase the treatment capacity of the digesters. The aerobic digesters have adequate capacity to treat residuals generated by influent flows up to 1.2 as they are currently operated. Additional pre-thickening will be needed to treat influent flows up to 1.6 mgd.

No Action Alternative

The SWRF could continue to operate as it is currently configured. The ability of the SWRF to consistently remove ammonia to levels below 1 mg/L as N was calculated based on a maximum month flow of 1.6 mgd, a BOD load of 3870 ppd, a minimum water temperature of 15 degrees Celsius, an average sludge age of 10 days, and a design MLSS concentration of 3500 mg/L. Calculations are provided in Appendix I. To achieve the desired effluent ammonia goal, 1.4 million gallons of aeration basin capacity is needed. The existing aeration basins contain 0.64 million gallons. To both nitrify and denitrify, 2.1 mg of aeration basins are needed. Denitrification calculations assume an effluent nitrate concentration of less than 10 mg/L as N. To meet the effluent ammonia treatment goal of 1 mg/L, the rated capacity of the SWRF should be decreased to a maximum month flow of 0.9 mgd and load of 2160 ppd of BOD.

In 2009, the Erie SWRF received a maximum month flow and load of 1.035 mgd and 2400 ppd BOD. Sludge age ranged between 10 and 15 days. Effluent ammonia concentrations ranged between 0.7 and 3.0 mg/L as N. The 2009 operating data agrees well with the calculated ammonia removal capacity. Maintaining effluent ammonia concentrations consistently below 1 mg/L, will require lower hydraulic and organic loading than 2009 operating conditions. As noted previously, the SWRF would have exceeded the daily maximum ammonia limits in the new discharge permit on 72 occasions between January 2003 and October 2009 and the 30-day average ammonia limits would have been exceeded on 12 occasions. Reducing the organic load will increase ammonia removal capacity.

If this alternative is pursued, it is recommended that the rated and permitted capacity be decreased to 0.9 mgd. Decreasing the flow below 1 mgd will substantially decrease monitoring requirements and eliminate the requirement for Whole Effluent Toxicity (WET) testing. If nitrate limits are added to the discharge permit in the future, the Town will need to further de-rate the SWRF to 0.6 mgd.

Decreasing the rated capacity of the SWRF will necessitate adding additional capacity at the NWRF in the future. The NWRF is an open site with adequate room for future expansions.

Extended Aeration Activated Sludge

To meet future permit limits and maintain the rated capacity of 1.6 mgd and 3870 ppd BOD, the SWRF will need substantial modifications to transform it from a conventional activated sludge process into an extended aeration activated sludge process or a system with similar characteristics.

Table 8-6 compares the SWRF's current design parameters to the State Design Criteria recommended in Policy 96-1. The SWRF design generally conforms to the State's definition for conventional activated sludge. Space loading is slightly higher than the recommended maximum. Conventional activated sludge systems can remove ammonia, but generally are not capable of removing ammonia to extremely low levels, less than 2 mg/L, consistently. Conventional systems are not designed to remove nitrate or to remove phosphorus biologically.

Table 8-6 Comparison of SWRF Design Parameters to State Design Criteria

Parameter	Current Design at 1.6 MGD and 3,870 ppd	State Design Criteria	
		Conventional	Extended Aeration
Treatment Goals	BOD and Ammonia Removal	BOD and Ammonia Removal	BOD, Ammonia, and Nitrate Removal. May remove phosphorus biologically.
Hydraulic Retention Time, hours	9.6	4 – 8	24
Space Loading, lbs BOD/1000 cf	43	< 40	< 25[1]
MLSS[2] Concentration, mg/L	4,000	1,500 – 4,000	2,000 – 5,000
Food to Microorganism Ratio	0.23	0.2 to 0.5	0.05 – 0.2
Mean Cell Residence Time, days[3]	10	3 – 15[3]	20 – 40[3]

[1]Published values from a variety of sources recommend space loading be kept below 15 ppd/1000 cf
[2]MLSS = Mixed Liquor Suspended Solids
[3]MCRT values not specified by Policy 96-1. Source: (Metcalf and Eddy 2003)
[4]State Design Criteria taken from Policy 96-1: Design Criteria Considered in the Design of Wastewater Treatment Facilities

Converting the SWRF to extended aeration activated sludge will require decreasing the space loading from 43 lbs to less than 20 lbs of BOD per 1000 cf of aeration basin and increasing the hydraulic residence time from 9.6 to 24 hours. Either of these modifications will nearly triple the footprint of the activated sludge process.

Aeration basin capacity may be added to the SWRF in two ways; by converting existing basins to activated sludge or by constructing additional basins. The existing aerobic digesters and solids holding tanks could be converted to additional aeration basin space. The total aeration basin volume required for ammonia removal is 1.4 million gallons. Converting both the digesters and solids handling tanks into aeration basins would yield a total volume of 1.19 mg. Process and design calculations indicate that the SWRF would be capable of processing 1.4 mgd and 2218 ppd of BOD. This is less capacity than the current rating of 1.6 mgd. Waste activated sludge would be conveyed to the NWRF for processing via the NWRF interceptor or new aerobic digesters could be constructed.

Conversion of the existing basins will require that the existing aeration headers, piping and diffusers be removed from the solids holding tanks and aerobic digesters and be replaced with fine bubble diffusers. The concrete walls separating the digesters and solids holding tanks from the aeration basin will need to be modified to allow flow through the basins to accommodate treatment. The concrete floor/walls will need to be repaired where air headers, piping, and diffusers were removed. Much of the existing process piping will be taken out of service or rerouted. A planning level cost estimate for converting the existing basins is presented in Table 8-7. Costs do not include modifications to the solids handling process or tertiary filtration.

Table 8-7 Estimated Construction Cost to Convert Existing Basins to Activated Sludge

Item	Quantity	Unit	Unit Cost	Est. Cost
Convert Aerobic Digesters to Aerobic Zones				
Diffuser Removal & Demolition	2	ea	6,500	13,000
Modify Walls for Flow & Misc. Wall Repairs	2	ea	10,000	20,000
Convert Biosolids Holding Tanks to Aerobic Zones				
Diffuser Removal & Demolition	2	ea	3,500	7,000
Wall Modification & Repairs	2	ea	10,000	20,000
Provide and Install Aeration Headers, Diffusers, Valves and Appurtenances	2	ls	55,000	110,000
WAS pumps and piping	2	ea	18000	36000
Yard Piping Modifications	1	ls	65,000	65,000
Site Work and Restoration	1	ls	20,000	20,000
Subtotal				**291,000**
Mobilization/Demobilization	6%	Percent		17,000
Contractor OH&P	12%	Percent		35,000
Insurance and Bonds	3%	Percent		9,000
Subtotal				**352,000**
Contingency	20%	Percent		70,000
Total Estimated Construction Cost				**422,000**
Engineering	15%	Percent		63,000
Total Estimated Project Cost				**$485,000**

The existing blowers, RAS pumps, and secondary clarifiers have adequate capacity and do not need to be upgraded. Process piping will require rerouting. The costs presented in Table 8-7 and 8-8 do not include solids handling processes at either the SWRF or NWRF that would be needed to process and stabilize waste solids. Either a new digester could be constructed at the SWRF location or waste solids can be transferred to the NWRF.

The benefits of converting the existing basins include: minimal construction activity at the SWRF, elimination of digester settling and decanting as well as its associated odors, and production of higher volumes of reuse water than the no action alternative. Disadvantages include the slightly decreased treatment capacity of 1.4 mgd, lack of process redundancy, the need to increase solids handling capabilities at the NWRF, and increased cleaning frequency for the NWRF interceptor. This alternative would not remove nitrate or phosphorus.

The SWRF lacks redundancy with only two aeration basins and two secondary clarifiers. At a rated capacity of 1.4 mgd, it will not be possible to take basins out of service for cleaning and maintenance without diverting a portion of the influent flow to the NWRF. Because Erie has the ability to divert flows on demand to the NWRF, lack of process redundancy is not problematic.

Aeration basin capacity may also be added by constructing additional basins. Table 8-8 presents estimated construction costs for adding two additional aeration basins. Each aeration basin will be slightly larger than the existing aeration basins for a total aeration basin volume of 1.4 million gallons. The existing blowers, RAS and WAS pumps, and secondary clarifiers have adequate capacity and do not need to be upgraded. Process piping will require rerouting.

Table 8-8 Estimated Construction Cost for Extended Aeration Activated Sludge

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	5,000	cy	10	50,000
Site Grading	1	ls	40,000	40,000
Buried Piping	1	ls	65,000	65,000
Aeration Basin Concrete	1,033	cy	500	656,333
15 hp RAS/WAS Pumps	3	ea	21,000	63,000
Diffusers	2	ls	55,000	110,000
Process Piping and Valving	1	ls	40,000	40,000
Pump building	250	sf	200	50,000
Electrical	20%	Percent		204,867
Subtotal				**1,279,200**
Mobilization/Demobilization	6%	Percent		2,000
Contractor OH&P	12%	Percent		5,000
Insurance and Bonds	3%	Percent		1,000
Subtotal				**1,287,200**
Contingency	20%	Percent		257,000
Total Estimated Construction Cost				**1,544,200**
Engineering	15%	Percent		232,000
Total Estimated Project Cost				**$1,776,200**

The new aeration basins could be located north or south of the existing treatment trains. Locating the basins to the south shortens pipe runs to and from the secondary clarifiers, but brings the treatment process closer to residences.

Advantages of constructing additional basins include maintenance of the rated capacity of 1.6 mgd, increased process redundancy, maintenance of solids handling capabilities, elimination of the need for increased solids handling capacity at the NWRF, no increase in cleaning frequency for the NWRF interceptor, and production of higher volumes of reuse water than both the no action and basin conversion alternatives. The primary disadvantages of this alternative are the expansion of the SWRF footprint which could result in public opposition and the greater likelihood that the WQCD will require a building or cover to be constructed over all of the biological treatment process basins.

While this alternative will not remove either nitrate or phosphorus, it does set the stage for meeting future permit limits. If the SWRF receives nitrate and phosphorus limits in the future as expected, the new aeration basins could be expanded to match the size and length of the existing treatment trains. For the two existing treatment trains, the solids holding tanks would be converted to anaerobic zones for biological phosphorus removal, the aerobic digesters would be converted to anoxic zones, and the aeration basins would remain as aerobic zones. Anaerobic and anoxic zones would be added to the two new treatment trains.

In addition to modifying process piping, propeller style mixers would be installed in the anaerobic and anoxic zones. Internal recycle pumps will be needed to cycle nitrate rich wastewater from the ends of the aerobic zones to the beginning of the anoxic zones. Typical recycle ratios are between 200 and 400 percent of influent flow depending on treatment goals. A 200 percent internal recycle can remove up to 67 percent of total influent nitrogen. A 400 percent internal recycle can remove up to 83 percent of total influent nitrogen.

Membrane Bioreactor

Membrane bioreactors (MBRs) combine an activate sludge process with membrane filtration. The membrane is a solids separation process and takes the place of the secondary clarifier.

In a conventional or extended aeration activated sludge system, the secondary clarifier limits the solids concentration in the aeration tank. The MBR replaces the secondary clarifier by using membranes instead of settling to separate the treated wastewater from the MLSS. The membranes allow the purified water to pass through the pores while creating a complete barrier to any solid greater than 0.5 μm (Metcalf and Eddy 2003). Because the membranes create a solids barrier, the process is not subject to gravity settling solids limitations or floating sludge associated with denitrification in the secondary clarifiers. The MBR is not limited by poor sludge settling caused by excessive sludge ages, filaments, or nutrient deficiency.

Conventional activated sludge systems are limited to MLSS concentrations between 1,500 and 4,000 mg/L. MBRs may be operated with MLSS concentrations of up to 10,000 mg/L. Because of the higher solids concentrations, a smaller tank volume is needed to maintain a desired food to microorganism ratio. Food to microorganism ratios (pounds of organic load entering the plant relative to the pounds of biomass in the system) between 0.05 and 0.10 are typically used for extended aeration activated sludge processes. If the MLSS concentration is twice as high in the MBR as it would be for an activated sludge plant, then the same amount of treatment can be accomplished in half the volume. For existing facilities with restricted footprints, like the Erie SWRF, MBRs can substantially increase treatment capacity without increasing the overall plant footprint.

The membranes are made from synthetic polymers with the most common being polyvinylidene difluoride (PVDF) and polythylsulphone (PES) (Judd 2006). Membranes are available in extruded fiber and flat plate configurations. The flat plate configuration was developed specifically for wastewater treatment. Water flows from outside the fiber or flat sheet, through the membrane, and into a collection header while solids remain in the aeration basin. Individual membranes are grouped or packaged together to form standard size cassettes or modules. Figure 8-3 shows a typical membrane fiber and a membrane cassette.

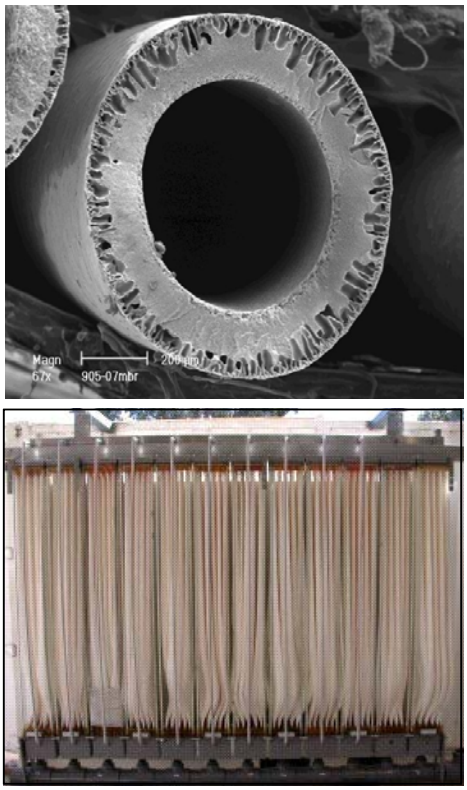


Figure 8-3 Membrane Bioreactor

MBR technology produces high quality effluent equivalent to a conventional activated sludge plant effluent followed by micro-filtration. MBR effluent quality meets the reuse standards specified in the California Title 22 Reuse regulations. Typical effluent BOD_5 and TSS concentrations are 10 mg/L and 2 mg/L, respectively. Complete nitrification (NH_3-N < 1 mg/L) can be achieved as well as total inorganic nitrogen concentrations below 8 mg/L. For Erie SWRF planning purposes, an effluent ammonia concentration of < 1.0 mg/L as N and an effluent total inorganic nitrogen limit of < 10.0 mg/L as N were assumed.

MBR technology is well established with four major manufacturers (Kubota, US Filter, Zenon, and Mitsubishi Rayon). As of 2006, there were over 2,250 MBR facilities in operation around the world. Of these, only 221 are in the United States with Zenon leading the market with 155 installations (Judd 2006). There are only a few MBRs currently operating in Colorado. All membrane systems currently in operation in Colorado are the extruded fiber type.

The limiting operational and design parameter for membrane systems is flux. Flux is the volume of water that can be moved through the membrane over a given period of time. Flux is typically expressed as $m^3/m^2/s$ or gal/sf/min. The different membrane manufacturers recommend different flux rates for average and peak flow conditions. Generally, the peak flow cannot be more than twice the average flow as higher flux rates increase fouling and may damage the membranes. Flux is influenced by water temperature, fouling, and other variables. As water temperature drops, the viscosity of the water increases and becomes more difficult to push through the pores of the membrane. Membrane systems are sized for the flux rate at the lowest expected water temperature. Additional capacity is then added to compensate for peak flows, membrane fouling, and the need to periodically remove membrane modules out of service for cleaning.

Fine screening and grit removal is important for membranes because the accumulation of trash and fibrous material such as hair and paper found in municipal wastewater can hinder performance and

ultimately diminish membrane life. Each membrane manufacturer has different guidelines for screening. Most state a maximum opening of 2 mm; however, some allow up to 3 mm (Water Environment Federation 2006). Screens with openings in this range can be controlled to allow for additional matting which reduces the effective screen opening even further. Grit can damage membranes by abrasion, especially in the turbulent environment created by the air scour. If membranes are selected as the preferred alternative for the Erie SWRF, the plant headworks will need to be upgraded to include fine screening.

An operational issue with membranes is fouling. Fouling is a result of biological and chemical deposits forming on the membrane surface and within the membrane pores. Higher flux rates force particulates deeper into the pores and can cause "irreversible fouling". Membrane systems incorporate air scour at the membrane surface to minimize the attachment of biological solids. They also use a relaxation step which temporarily drops the flux rate through the membrane and loosens solids and biological growth on the membrane surface. Solids are more easily removed by the air scour during relaxation.

Air scour and relaxation of the membranes is supplemented with chemical cleaning to remove "irreversible" fouling. This type of cleaning usually uses a combination of chemically enhanced backwash (daily), maintenance cleaning with a higher chemical concentration (weekly), and intensive recovery chemical cleaning (once or twice per year). Maintenance cleaning is conducted in situ. It helps to maintain membrane permeability and reduces the frequency of recovery cleanings. Maintenance cleaning typically takes between 30 and 60 minutes for a complete cycle while recovery cleaning takes much longer. Table 8-9 lists recommended cleaning protocols for four different manufacturers.

Table 8-9 Membrane Cleaning Protocols

Vendor	Type	Chemical	Concentration, %	Protocols
Mitsubishi	CIP[1]	NaOCl Citric Acid	0.3 0.2	Backflow through membrane (2 hr) + soaking (2 hr)
Zenon	CIA[2]	NaOCl Citric Acid	0.2 0.2 – 0.3	Backpulse and recirculate
Memcor	CIA[2]	NaOCl	0.01	Recirculate through lumens, mixed liquors, and in-tank air manifolds
Kubota	CIP[1]	Citric Acid NaOCl Oxalic Acid	0.2 0.5 1	Backflow and soaking (2 hr)

[1]CIP = Cleaning in place, without membrane tank draining: chemical solutions generally backflushed in-to-out.
[2]CIA = Cleaning in air, where membrane tank is isolated and drained; the module is rinsed before being soaked in cleaning solution and rinsed after soaking to remove excess reagent.
Source: (Judd 2006)

Design Criteria for SWRF MBR System. Membranes were considered as one alternative for adding nutrient removal capacity to the SWRF. Design criteria used for preliminary process sizing are presented in Table 8-10. For planning purposes, equipment quotes were obtained from two different membrane manufacturers: Siemens and GE Water and Process Technologies. Manufacturer's information is included in Appendix J. A major advantage of the membrane alternative is the ability to remove both ammonia and nitrate to low levels. Membranes produce reuse quality water and would not require tertiary filtration.

Table 8-10 Design Criteria for an MBR System

Parameter	Value
Average Daily Flow, mgd	1.26
Maximum Month Flow, mgd	1.60
Peak Hour Flow, mgd	2.74
Maximum Month BOD Load, ppd	3870
Water Temperature Range, degrees Celsius	10 – 20
Effluent BOD and TSS, mg/L	5 / 5
Effluent Ammonia, mg/L as N	< 1
Effluent Total Nitrogen, mg/L as N	< 10
Total Number of Membrane Tanks	3
Membrane Module Type	Siemens, B40N + Mempulse
Number of Membrane Racks per Tank	8
Number of Membrane Modules per Tank	128
Membrane Area, per tank, sf	51,805
Membrane Tank Dimensions, feet	13.5 L, 10.9 W, 9.8 D
Net Flux at Peak Hour Flow, gpd/sf	17.6
Maximum MLSS Concentration, mg/L	14,500
Average Air Flow per Membrane Tank, scfm	328
Peak Air Flow per Membrane Tank, scfm	678
MLSS Feed Pump	3 at 15 hp each
Filtrate Pump	3 at 10 hp each
MBR Tank Blowers	3 at 25 hp each
Compressed Air for Air Scour	2 at 7.5 hp each
Chemical Feed for Clean-in-Place, chlorine	1900 gallons per year at 12.5% solution
Chemical Feed for Clean-in-Place, citric acid	1443 gallons per year at 50% solution

Retrofitting the membrane cartridges into the existing secondary clarifiers was discussed with two different membrane manufacturers. Both expressed reluctance in placing the membrane cartridges into the round tanks because the tank configuration would severely impair the membrane hydraulics and air scour efficiency. The internal recirculating flow through the membrane tanks back to the bioreactor will be four times the influent flow (4Q) which the clarifiers are not designed to meet. Recirculation encourages denitrification and helps to minimize membrane fouling. Accommodating the high internal recycle flows would require new, larger diameter process piping between the aeration basins and the clarifiers. Instead, each manufacturer recommended constructing separate tanks for the membrane cartridges and maintaining the existing aeration basins for biological treatment.

While the existing blowers should be capable of supporting the biological processes within the existing aeration basins, additional blowers and a compressed air system will be needed to support the membranes. The membranes will also need two sets of pumps: MLSS influent pumps to pull MLSS from the existing aeration basins to the new membrane tanks and permeate pumps to draw a vacuum on the membranes and remove treated effluent from the membrane tanks. The blowers and pumps are listed in Table 8-10 and are included in the estimated construction costs as part of Membrane Equipment.

A new building will be needed to house the membrane tanks and associate equipment. The 50 ft by 45 ft (2,250 sf) membrane building could be located south of the existing aeration basins. The building fits within the site footprint, but is new construction and may result in public opposition to the SWRF improvements. Figure 8-4 shows the proposed process schematic.

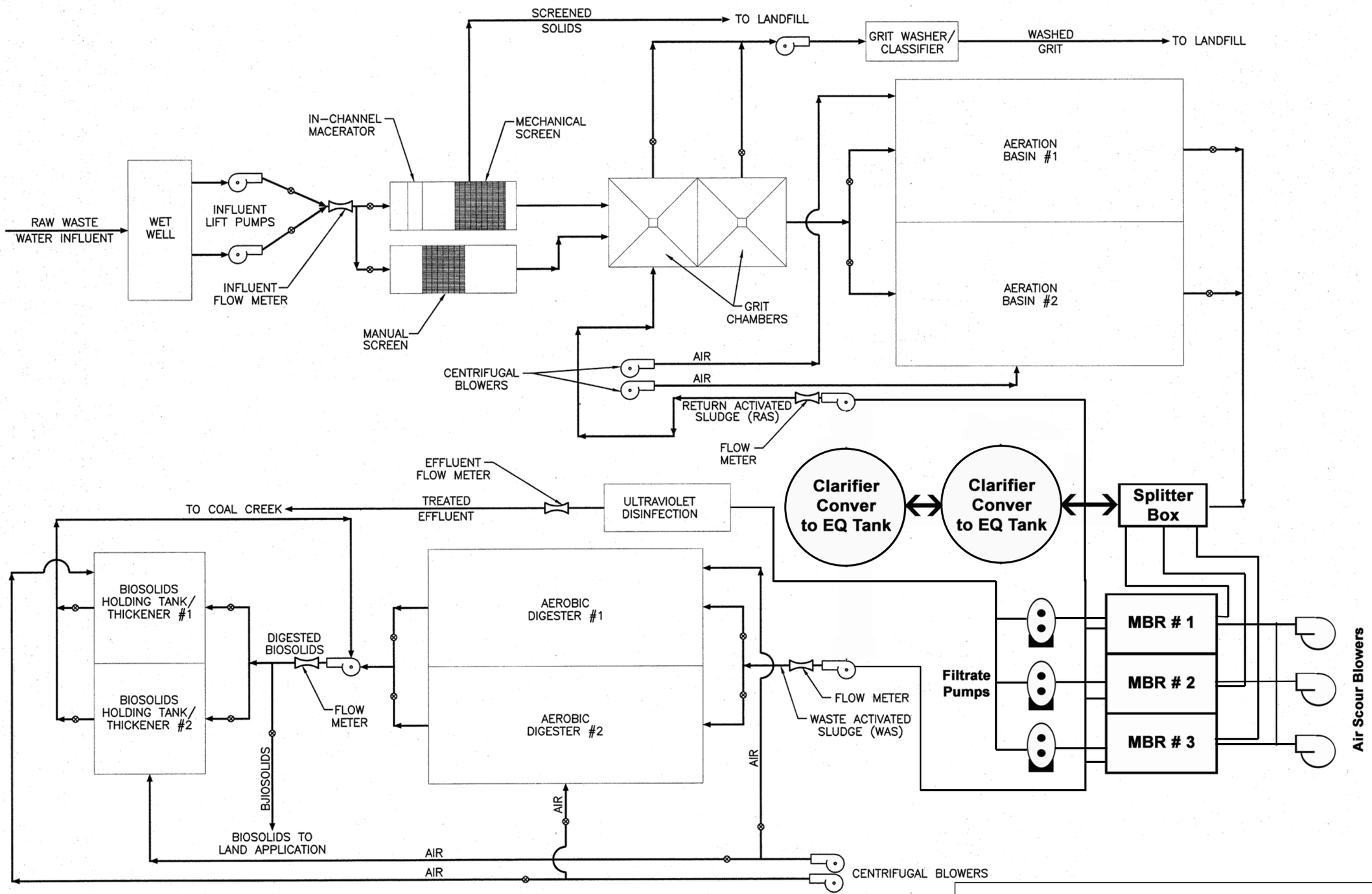


Figure 8-4 Membrane Bioreactor Process Schematic

Estimated Construction Costs for SWRF MBR System. Estimated costs for material and equipment for major components of the membrane bioreactor system are shown in Table 8-10. Costs include new buildings for the membranes and associated equipment. MBR construction costs may be reduced by incorporating some flow equalization into the final design which would reduce the membrane equipment cost from $1.9 million to $1.5 million (Siemens). The existing secondary clarifiers could be used for flow equalization. Membrane equipment costs for GE Water and Process Technologies were substantially higher at $3.1 million. The costs presented in Table 8-11 do not include modifications to the solids handling processes that would be needed to process and stabilize waste solids or upgrades to the site electrical service.

Table 8-11 Estimated Project Cost for Membrane Bioreactor (MBR) System

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	1,000	cy	10	10,000
Site Grading	1	ls	40,000	40,000
Fine screen	1	ea	65,000	65,000
MBR Building, 50 ft by 45 ft	1	sf	200	450,000
MBR Basin Concrete	236	cy	500	117,778
Membrane Equipment	1	ls	1,881,000	1,881,000
Process Piping and Valving	1	ls	50,000	50,000
Subtotal				2,613,778
HVAC	15%	Percent		392,067
Electrical	20%	Percent		522,756
Subtotal				**6,142,378**
Mobilization/Demobilization	6%	Percent		368,543
Contractor OH&P	12%	Percent		737,085
Insurance and Bonds	3%	Percent		184,271
Subtotal				**7,432,277**
Contingency	20%	Percent		1,486,000
Total Estimated Construction Cost				**8,918,277**
Engineering	15%	Percent		1,338,000
Total Estimated Project Cost				**$10,256,277**

The estimated construction cost to convert the SWRF to a 1.6 mgd membrane bioreactor process is $10.2 million. Construction costs are significantly higher than either of the extended aeration activated sludge alternatives for two reasons. First is the cost of the membrane equipment itself which includes the membranes, new blowers and aeration grids, permeate pumps, and chemical feed equipment. Second is the cost of constructing a new 50 ft by 45 ft building to house the system.

Membrane bioreactor systems have higher operating costs and electrical demand than other processes. Electrical demand includes aeration, recirculation pumps, filtrate pumps, chemical feed pumps, compressors, etc., of the MBR system. For the SWRF, electrical demand is expected to increase by 1,100 kWh/day during average daily flows and loads and 1,400 kWh/day during maximum month flows and loads. This electrical demand is in addition to the demand exerted by the existing blowers and pumps

which would continue to operate. At $0.07 per kWh, an MBR would increase electrical costs at the SWRF by $35,770 each year not including peak demand charges.

The membrane modules will need to be replaced somewhere between every five (5) and ten (10) years with the current technology. While the costs have decreased over the past several years, these modules can still be classified as expensive. The membranes "dry out" due to the flexible polymers leaching out, the closing/plugging of the pores, and the membranes becoming somewhat hard or brittle.

Membrane bioreactors have many positive attributes including consistent, high quality effluent that meets California Title 22 regulations for reuse. MBRs are relatively simple to operate and require less process control than a conventional activated sludge system provided the operator has been properly trained and pays strict attention to the proper operation, corrective maintenance, and preventative maintenance tasks. With an MBR, the Town may be able to operate both the SWRF and NWRF with existing staff or one additional operator. Selecting an MBR system would ensure that the SWRF produces the best quality effluent possible with currently available treatment technologies and would position the Town to meet future effluent limits.

To summarize, the advantages and disadvantages of converting to an MBR system include:

Advantages	Disadvantages
• Maintains rated capacity of 1.6 mgd. • Solids settling and filaments are no longer an operational concern. • Removes ammonia to <1 mg/L as N. • Removes total nitrogen to <10 mg/L. • Compatible with chemical phosphorus removal. • Produces reuse quality water without tertiary filtration. • Ease of operation and reduced process control monitoring. The Town may be able to operate both the NWRF and SWRF with existing staff. • Maintains aerobic digesters and solids handling capacity at SWRF.	• Highest construction cost of all alternatives evaluated. • Highest annual operating cost of all alternatives evaluated. • Headworks modification needed to accommodate fine screening down to 2 mm. • New basins are needed to house membranes. • New building is needed to house ancillary equipment. • Requires periodic replacement of the membranes. Complete replacement expected once every 5 to 10 years. • Requires on-site chemical storage for membrane cleaning. • Potential for public opposition. • Solids thickening is needed to increase the aerobic digester capacity while preventing odor generation.

Integrated Fixed Film Activated Sludge

Integrated Fixed Film Activated Sludge Systems (IFAS) combine fixed film with activated sludge. Erie selected IFAS technology for the new NWRF which became operational in early 2011. Free-floating plastic media are added to the aeration basin to provide a surface area for fixed film growth. The media are mixed throughout the basin by the aeration system. The biological solids that attach to and grow on the plastic media are called biofilm. The biofilm is retained in the basin and boosts the total mass of biological solids available for treatment and effectively increase sludge age. The biofilm solids have a long effective sludge age (>30days) that sustains nitrification even when the MLSS in the suspended phase has a much shorter sludge age.

The plastic media are prevented from leaving the aeration basins by large metal screens. MLSS and treated wastewater flow through the screens to the secondary clarifiers. IFAS systems increase BOD removal, nitrification and denitrification capacity in conventional activated sludge processes like the one at the Erie SWRF without increasing the volume of existing aeration basins. They cannot increase hydraulic capacity.

One limitation of traditional activated sludge processes is the solids loading rate to the secondary clarifiers which is expressed as pounds per day per square foot (lb/sf*d). The clarifier solids loading rate typically limits the MLSS concentration in the aeration basins to less than 4,000 mg/L. Facilities with large clarifiers can accommodate higher MLSS concentrations. With IFAS, most of the biological solids are retained in the aeration basin. This allows the treatment plant to maintain a large mass of biological solids while simultaneously keeping the clarifier solids loading rate low.

IFAS systems produce high quality effluent. Typical effluent BOD_5 and TSS concentrations are 10 mg/L and 10 mg/L, respectively. Complete nitrification (NH_3-N < 1 mg/L) can be achieved as well as total inorganic nitrogen concentrations below 8 mg/L. For Erie SWRF planning purposes, an effluent ammonia concentration of < 1.0 mg/L as N and an effluent total inorganic nitrogen limit of < 10.0 mg/L as N were assumed. IFAS systems do not produce reuse quality water without tertiary filtration.

IFAS technology is well established with multiple manufacturers including: Anox-Kaldnes, Entex, Brentwood, Linpor, and others. Anox-Kaldnes has more than 300 installations worldwide with two of their larger installations located in Colorado. The City and County of Broomfield WWTP is rated for 8 mgd and has been operational since 2002. The South Adams County WWTP is rated for 5.5 mgd and has been operational since 2003. Other nearby IFAS installations include the Crow Creek and Dry Creek treatment facilities in Cheyenne, Wyoming; Johnstown, Colorado; and Oak Creek, Colorado.

A fine screen is recommended upstream of the IFAS to prevent material such as hair from interfering with the plastic media. An IFAS needs a 6 mm screen which has larger openings than the 2 mm screen required for MBR systems. The SWRF has a Vulcan Step Screen with 6 mm openings and is adequate to support the IFAS process.

IFAS systems tend to operate with higher dissolved oxygen concentrations than traditional activated sludge systems. The Broomfield IFAS maintains a DO between 4.5 and 6.0 mg/L in their aerated zones.

The existing activated sludge blowers do not have enough capacity to support an IFAS and would need to be replaced or modified to supply an additional 3000 scfm of air to the process. IFAS systems require 75 to 85 percent more air than a conventional activated sludge process. Higher aeration demand results in higher operating costs. Some IFAS systems have been operated with DO concentrations closer to 3 mg/L in an effort to encourage simultaneous nitrification and denitrification within the biofilm.

Periodic access to the grid of aeration diffusers located below the media is required for maintenance. If the fill volume of the plastic media type IFAS system is relatively low (20 to 25%), then the media may be left in the basin as it is dewatered. After dewatering, the media could be pushed to the side to gain access to the diffusers. At fill volumes greater than 25%, it is recommended that the media be pumped from the media cell to another tank that does not contain media or distribute them to the basins remaining in operation. Note that the media can deteriorate if subjected to ultra violet radiation so the media must be protected from exposure to sunlight when a tank is dewatered.

Design Criteria for SWRF IFAS System. Design criteria used for preliminary process sizing are presented in Table 8-12. For planning purposes, an equipment quote was obtained from Anox-Kaldnes, the original developer of the technology. Manufacturer's information is included in Appendix J. Like the MBR process, IFAS would remove both ammonia and nitrate and is capable of producing total nitrogen concentrations below 10 mg/L as N. IFAS is compatible with both biological and chemical phosphorus removal.

Table 8-12 Design Criteria for an IFAS System

Parameter	Value
Average Daily Flow, mgd	1.26
Maximum Month Flow, mgd	1.60
Peak Hour Flow, mgd	3.90
Maximum Month BOD Load, ppd	3870
Maximum Month TKN Load, ppd	681
Water Temperature Range, degrees Celsius	8 – 20
Effluent soluble BOD, mg/L	< 20
Effluent Total Nitrogen, mg/L as N	< 10
Number of Process Trains	2
Total Bulk Media Volume, cf	34,079
Total Effective Surface Area, sf	5,193,587
% Fill of Biofilm Carriers in Aerobic Zone 1	65
% Fill of Biofilm Carriers in Aerobic Zone 2	52
Number of Sieve Assemblies, Total	20
Submersible Mixers for Anoxic Zones	2 @ 4.3 hp each
MLSS Internal Recycle Pumps	2 @ 3.2 mgd with 3 ft lift
Design Residual DO, mg/l	3
Total Air Requirement at Design Flow, scfm	3,242 @ 8.0 psig
Average MLSS Concentration, mg/L	3,500

To convert the existing activated sludge process to an IFAS process, each of the existing aeration basins would be divided into two aerobic zones and one anoxic zone with concrete walls. A preliminary site layout is shown in Figure 8-5. The IFAS process schematic is shown in Figure 8-6. Each anoxic zone is

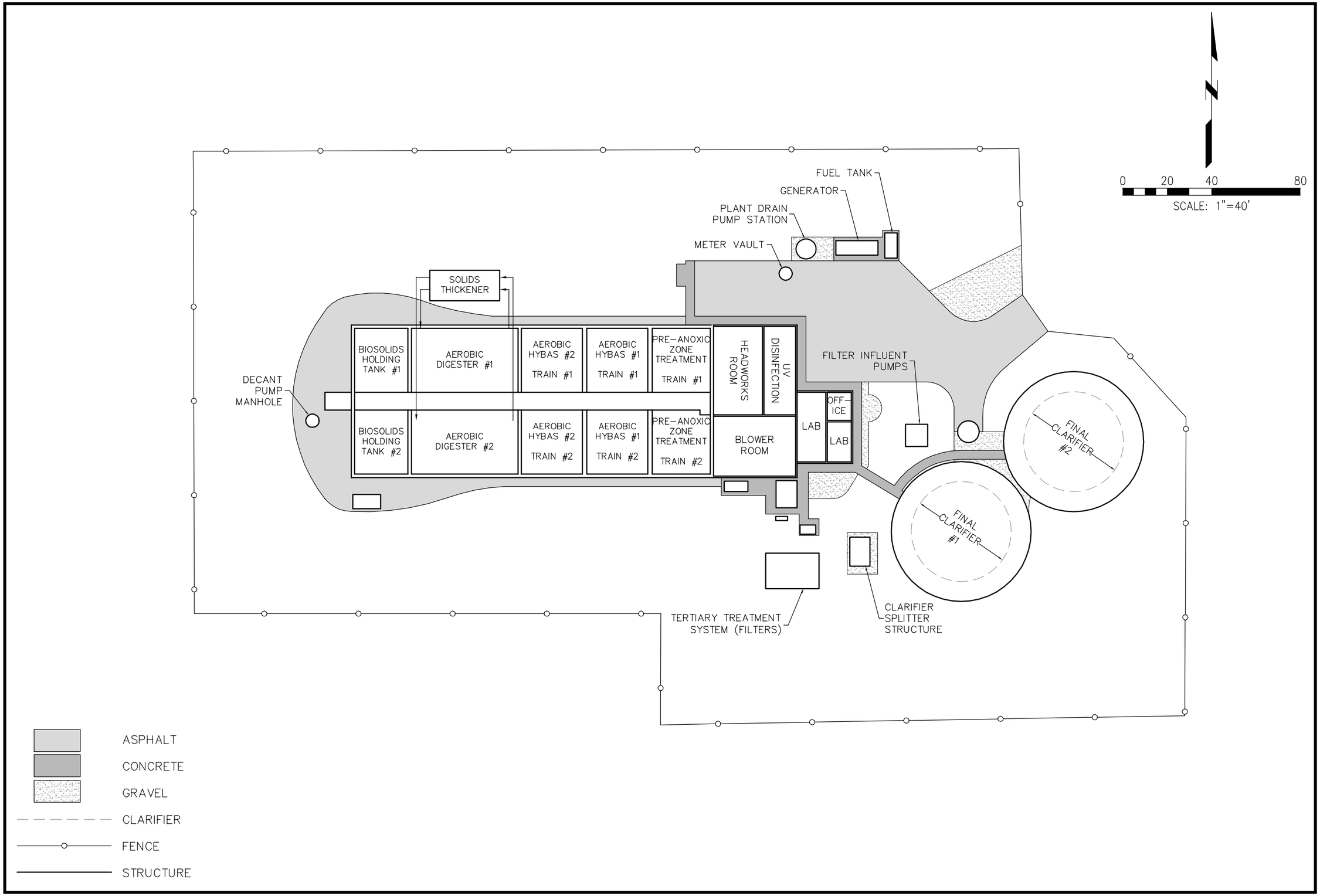


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FIGURE 8-5 SITE LAYOUT FOR INTEGRATED FIXED FILM ACTIVATED SLUDGE
1.6 MGD WASTEWATER TREATMENT FACILITY
PLANT LAYOUT

Project No:	0090672
Book No:	
Drawn by:	TWL
Reviewed:	GD
Date:	06/06/2011

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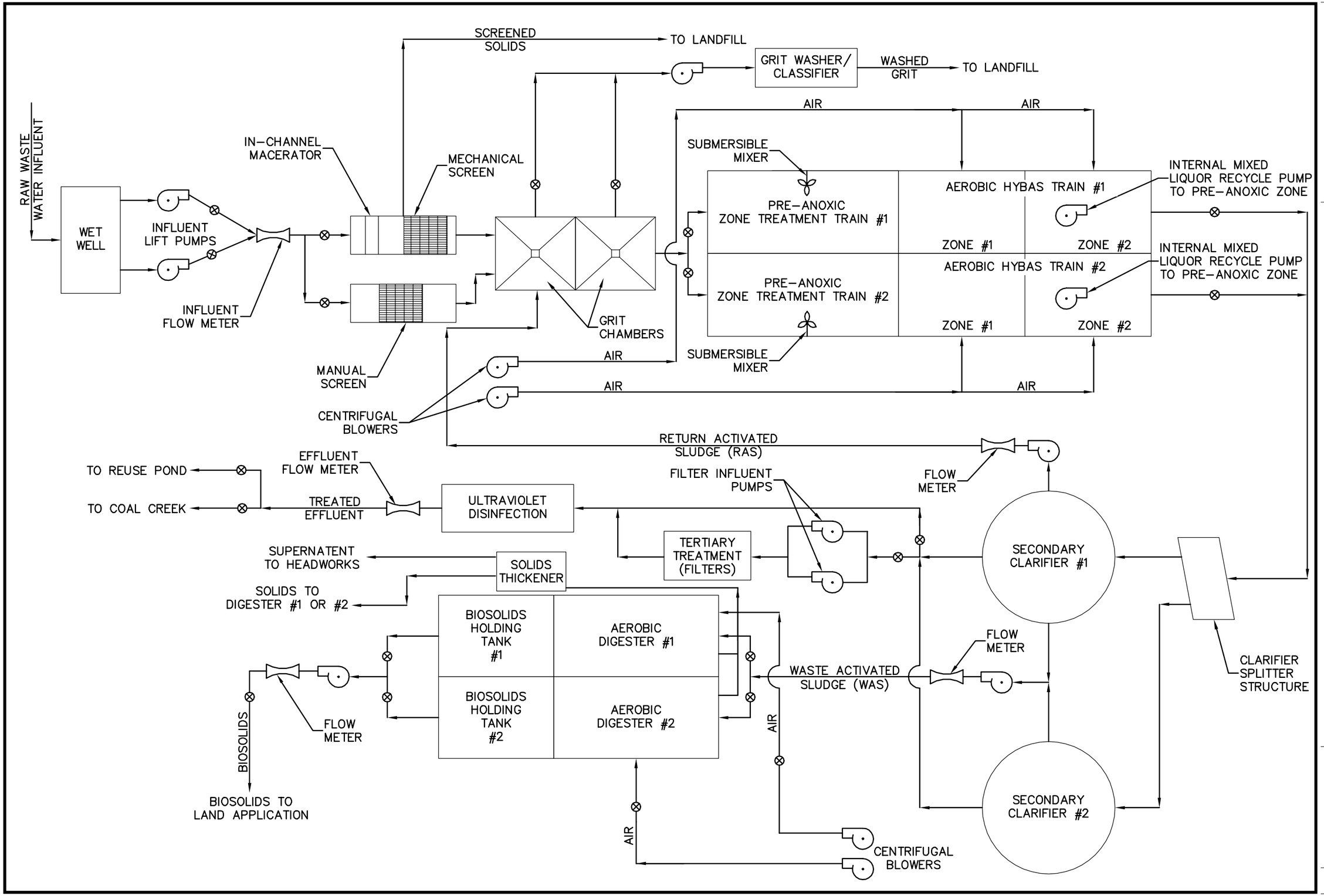

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FIGURE 8-6 IFAS PROCESS SCHEMATIC
1.6 MGD WASTEWATER TREATMENT FACILITY
PROCESS SCHEMATIC

Project No:	0090672
Book No:	
Drawn by:	TWL
Reviewed:	GD
Date:	06/06/2011

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equipped with a propeller type mixer. The mixers are wall mounted on stainless steel slide rail systems for ease of removal. Each is rated for 4.3 hp. Each of the four aerated zones is fitted with new, medium bubble diffusers and five media retention screens. The second aerated zone of each treatment train is equipped with an internal recycle pump for cycling MLSS from the end of the aerated zone to the beginning of the anoxic zone. Recycle rates of up to four times (4Q) the plant influent flow may be used to encourage denitrification.

Instrumentation and controls for the system are provided with the Hybas package. The PLC processor is proposed to be an Allen Bradley unit housed in a NEMA 12 wall mounted enclosure. Field instruments for the system include, two (2) high level float switches, four (4) DO probes, two (2) ammonia analyzers and two (2) nitrate analyzers.

The existing solids holding tanks and aerobic digesters will not be modified. Waste solids from the IFAS process will be stabilized aerobically at the SWRF. As noted previously, the aerobic digesters do not have adequate capacity to process the waste solids resulting from treating 1.6 mgd without additional thickening. Thickening can be done mechanically or with chemical addition and decanting. If biological phosphorus removal is desired, the existing aerobic digesters may be converted to anaerobic zones or a portion of first pre-anoxic zone in each train could be sectioned off to provide an anaerobic zone with a minimum 30-minute hydraulic retention time. In the later case, the internal mixed liquor recycle pumps will return to the beginning of the anoxic zone rather than the anaerobic zone. Chemical phosphorus removal may also be added to this alternative followed by tertiary filtration to achieve effluent phosphorus levels as low as 0.03 mg/L as P.

Estimated Construction Costs for SWRF IFAS System. Estimated costs for material and equipment for major components of the IFAS system are shown in Table 8-13. Costs include modifications to the existing aeration basins and process piping, the IFAS media, pumps, aeration system, and screens, and modifications to the existing blowers to increase capacity. The estimated construction cost to convert the SWRF from activated sludge to a 1.6 mgd IFAS process is $4.5 million. Costs do not include modifications to the solids handling process, tertiary filtration, or upgrades to the site electrical service.

A primary advantage of the IFAS system is that it fits within the existing aeration basins and preserves solids handling capability at the SWRF. It will not require construction of additional tanks or buildings. Maintaining the existing plant footprint may reduce public opposition to upgrading the facility. Unlike the extended aeration activated sludge alternative, an IFAS will remove both ammonia and nitrate. Total effluent nitrogen concentrations should be below 10 mg/L as N. If phosphorus removal is required in the future, the IFAS can be modified to include an anaerobic zone for biological phosphorus removal or may be followed by chemical precipitation and filtration.

The Town selected IFAS for the new NWRF which became operational in early 2011. Selecting the same secondary treatment technology for both facilities will reduce demand for operator training and will allow spare parts to be shared between facilities thereby reducing inventory requirements. The IFAS system does not require chemical cleaning like the MBR.

IFAS systems have higher energy use than traditional activated sludge facilities, but use much less than a membrane bioreactor.

Table 8-13 Estimated Project Cost for Integrated Fixed Film Activated Sludge (IFAS) System

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	1,000	cy	10	10,000
Site Grading	1	ls	40,000	40,000
Demo Existing Aeration Headers and Diffusers	2	ea	12,000	24,000
Concrete walls to divide existing aeration basin	64	cy	500	64,000
IFAS Equipment	1	ls	1,927,000	1,927,000
Modify existing aeration basin blowers	3	ea	75,000	225,000
Process Piping and Valving	1	ls	50,000	50,000
Electrical	20%	Percent		456,800
Subtotal				**2,740,800**
Mobilization/Demobilization	6%	Percent		164,448
Contractor OH&P	12%	Percent		328,896
Insurance and Bonds	3%	Percent		82,224
Subtotal				**3,316,368**
Contingency	20%	Percent		663,000
Total Estimated Construction Cost				**3,979,368**
Engineering	15%	Percent		597,000
Total Estimated Project Cost				**$4,576,368**

To summarize, the advantages and disadvantages of converting to an IFAS system include:

Advantages

- Maintains rated capacity of 1.6 mgd.
- Removes ammonia to <1 mg/L as N.
- Removes total nitrogen to <10 mg/L.
- Compatible with biological and chemical phosphorus removal.
- Fits within existing aeration basins.
- Significantly less expensive than MBR system.
- Lower operating costs than MBR system.
- Reduced potential for public opposition.
- Maintains aerobic digesters and solids handling capacity at SWRF.
- Same technology at both SWRF and NWRF reduces need for operator training.
- Potential for reuse water generation when combined with tertiary filtration.

Disadvantages

- Higher capital cost than extended aeration activated sludge.
- Requires periodic replacement of the plastic media. Complete replacement expected once every 10 to 20 years.
- Higher energy costs than traditional activated sludge.
- Existing blowers require modification to increase total scfm available.
- Solids thickening is needed to increase the aerobic digester capacity while preventing odor generation.

IFAS Future Direction. The SWRF is ideally located for providing reuse water to existing development. The Town may wish to consider converting all of the existing basins (solids handling tanks, digesters, and aeration basins) into a larger, two-train IFAS system capable of treating up to 3.0 mgd. A proposed site layout is shown in Figure 8-7. Under this alternative, the solids handling tanks would be converted to anaerobic zones and the aerobic digesters would become anoxic zones. The aeration basins would be subdivided to form two anoxic IFAS basins per treatment train.

Additional modifications that would be needed to support a larger IFAS system include:

- Replacing the digester and aeration basin blowers
- Adding pumping and screening capacity at the plant headworks
- Replacing the existing grit basin with a vortex grit basin
- Rerouting process piping
- Adding a third secondary clarifier
- Increasing size of RAS/WAS pumps
- Adding additional UV disinfection capacity with more bulbs or a parallel channel
- Addition of a solids pump station to transfer solids to the NWRF interceptor for conveyance to the NWRF for stabilization

Because the concrete basins are existing, modifying the SWRF may be less expensive than building a parallel treatment train at the NWRF location. Moving all solids handling activities to the NWRF would alleviate odor concerns at the SWRF site.

Process Technology Selection

Five process alternatives were evaluated for the SWRF. The least expensive alternative is to de-rate the facility to 0.9 mgd. If this alternative is selected, then additional capacity will be needed at the NWRF. Costs for additional capacity at the NWRF were not evaluated. The next two alternatives evaluated different ways to convert the SWRF into an extended aeration activated sludge process. Both would meet the target ammonia effluent limit of less than 1 mg/L as N, but would not remove nitrate or phosphorus. Since limits on both of these parameters is likely within the next five to ten years, these alternatives, while fiscally attractive, don't represent the best value for the Town over the long-term.

The two remaining alternatives are a membrane bioreactor (MBR) and integrated fixed-film activated sludge (IFAS). The MBR process cannot be retrofitted into the existing basins and will require both new process tanks and a building to house ancillary equipment. Because of this, the MBR alternative is the most expensive by a factor of two. For this reason, it was not considered further.

The recommended alternative is to convert the existing aeration basins into an IFAS process. The IFAS will meet current effluent ammonia limits and will also remove nitrate to below 10 mg/L as N. The IFAS fits entirely within the existing aeration basins and can reuse the existing pumps, process piping, and secondary clarifiers. Additional aeration will be needed; however, the existing blowers may be modified by increasing motor speed.

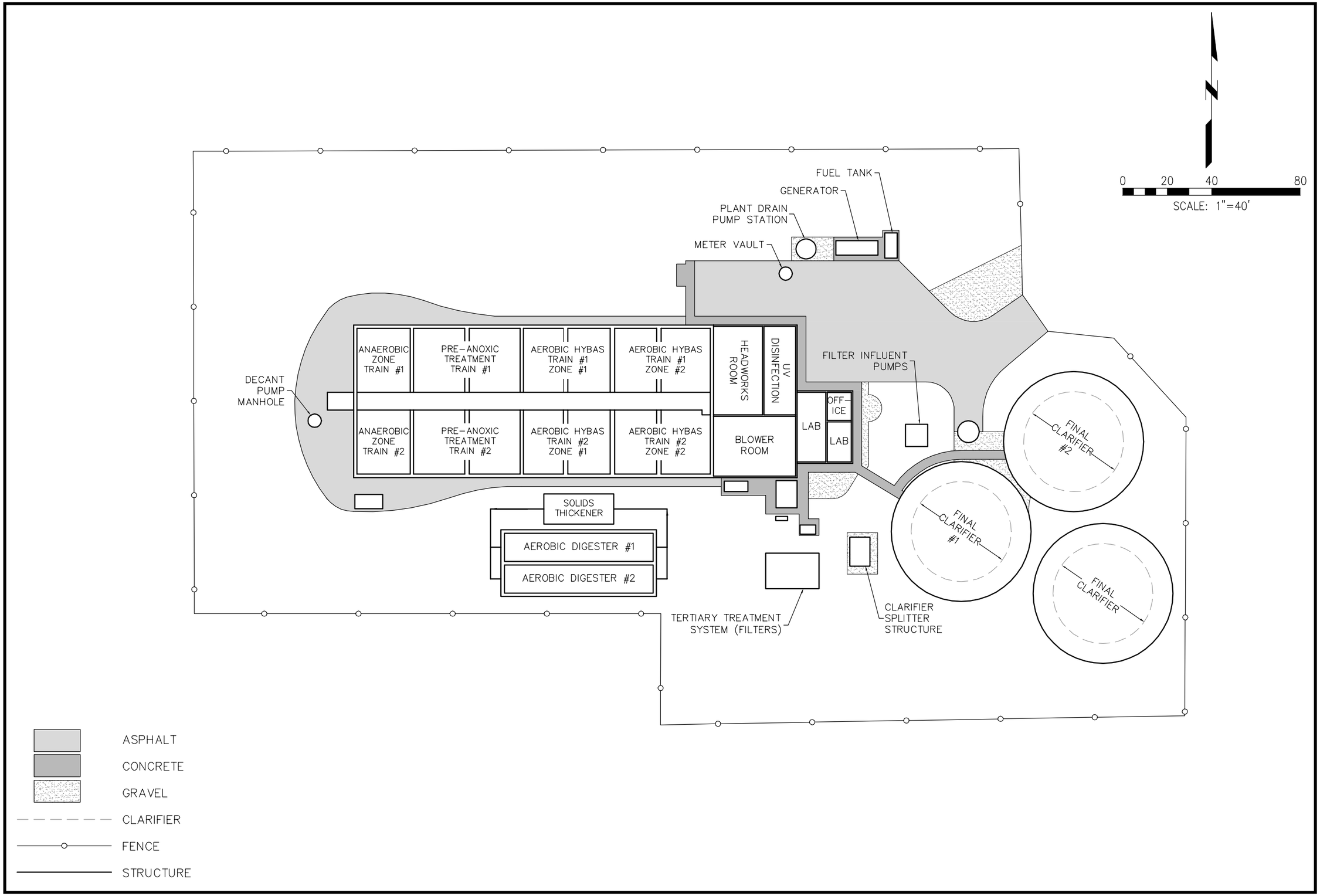

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FIGURE 8-7 IFAS FUTURE DIRECTION SITE LAYOUT
3.0 MGD WASTEWATER TREATMENT FACILITY
PLANT LAYOUT

Project No:	0090672
Book No:	
Drawn by:	TWL
Reviewed:	GD
Date:	06/06/2011

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The IFAS process is compatible with both biological and chemical phosphorus removal. To do biological phosphorus removal, a small anaerobic zone will be needed upstream of the IFAS. The anaerobic zone typically only needs a 30 to 45 minute hydraulic detention time and would be relatively inexpensive to construct. Chemical phosphorus removal can be done by adding either ferric chloride, alum, or lime immediately upstream of a tertiary filter.

Finally, the IFAS process is capable of producing reuse quality water when used in combination with tertiary filtration.

Table 8-14 Process Technology Selection Summary

	No Action	Extended Aeration within Existing Basins	Extended Aeration with New Basins	Membrane Bioreactor	IFAS
Estimated Construction Cost, millions	$0	$0.49	$1.78	$10.3	$4.6
Maintain Rated Capacity of 1.6 mgd	N	N	Y	Y	Y
Rated Capacity of 1.4 mgd	N	Y	N/A	N/A	N/A
Effluent Ammonia < 1.0 mg/L	Y	Y	Y	Y	Y
Effluent Nitrate < 10.0 mg/L	N	N	N	Y	Y
Compatible with either Biological or Chemical Phosphorus Removal	Y	Y	Y	Y	Y
Process Fits within Existing Basins	Y	Y	N	N	Y
Maintains Aerobic Digesters	Y	N	Y	Y	Y
Solids Thickening Required	N	N	Y	Y	Y
Produces Reuse Quality Water without Filter	N	N	N	Y	N
Produces Reuse Quality Water with Filtration	Y	Y	Y	N/A	Y
Reuse Existing Blowers	Y	Y	Y	Y	N
Requires Additional Blowers	N	N	N	Y	Y
Requires Additional Pumps	N	N	Y	Y	Y
New Process Building Needed	N	N	N	Y	N

TERTIARY FILTRATION

Tertiary filtration was evaluated for producing reuse quality water at the SWRF. Erie plans to use non-potable supplies consisting of raw water and reclaimed treated wastewater effluent to meet a portion of its current and projected water needs. These non-potable supplies will be used for irrigation of parks and commercial, industrial, and multi-family residential landscaping, extending Erie's water supplies and reducing water treatment costs. Non-potable water use will also promote water sustainability through the efficient use and reuse of a valuable resource. Erie completed a Non-Potable Municipal Water System Master Plan in September 2007 (CDM September 2007). The Master Plan predicted that the highest non-potable demands at buildout would occur in the southeast portion of the service area, within 2 to 3 miles of Interstate I-25 and/or Highway 7 and around Old Town. The SWRF is better located to serve these areas than the NWRF.

The Lower Boulder Ditch passes within 250 feet of the SWRF and the Cottonwood Extension Ditch (aka Erie Coal Creek Ditch) is within 2000 feet. Either or both of these canals could be used to transfer reuse

water to the north central and eastern portions of the service area by gravity. A non-potable pump station can be used to transfer water to other portions of the service area. Infrastructure needed to maximize water reuse is detailed in the 2007 Master Plan by CDM.

Reuse water quality is governed by Regulation 84 in Colorado. Table 8-15 summarizes reuse water categories and allowed uses. Erie will require Category 2 reuse water for multi-family residential landscaping. If new subdivisions are constructed with duel potable and non-potable water sources for residential use, Category 3 reuse water will be needed. Reuse water may be stored on-site in the existing lagoon cells from the original treatment facility. The two remaining lagoon cells hold a combined volume of 4.6 mg.

Table 8-15 Reuse Water Quality Requirements from Regulation 84

Parameter	Category 1	Category 2	Category 3
E coli / 100 ml[1]	126 / 235	126 / 235	None detected in at least 75% of samples in a calendar month. Single sample maximum of 126
Total Suspended Solids or Turbidity	30 mg/L daily maximum	Not to exceed 3 NTU as a monthly average and not to exceed 5 NTU in more than 5 percent of analytical results during any calendar month.	Not to exceed 3 NTU as a monthly average and not to exceed 5 NTU in more than 5 percent of analytical results during any calendar month.
Industrial Uses	Allowed	Allowed	Allowed
Landscape Irrigation	Allowed for areas with restricted access only.	Allowed for both restricted and non-restricted access. Not suitable for resident controlled use.	Allowed for all uses.
Commercial Uses	Allowed	Allowed	Allowed
Fire Protection	Not allowed	Allowed for non-residential use	Allowed for all uses.

[1]First number is monthly geometric mean. Second number is single sample maximum.

Three alternatives were considered for tertiary filtration: the Kruger Hyrdotech Disc-Filter, Miller-Leahman's ultrafiltration filter, and Lighthouse Filters' Volcano Continuous Downflow Filter. Each filter was sized based on an average daily flow of 1.0 mgd and a peak hour flow of 2.0 mgd. Each filter is capable of producing reuse quality water in accordance with Regulation 84. Each filter is compatible with chemical phosphorus removal and could be expanded for that purpose when the SWRF received phosphorus limits. Chemical storage and feed systems for either a metal salt or alum will also be needed at that time.

Hydrotech Discfilter – Kruger. In the Discfilter process, wastewater flows from the outside into the filter discs from a center trough. The solids are filtered from the wastewater as the clearer water flows through the disc media. It is possible to install a discfilter within the existing hydraulic profile so pumping would not be required. The media consists of thick-nap fabric socks that fit over an underlying support frame. The fabric socks are secured with finger tight wing nuts for easy removal. The filter media is compatible

with chemical dosing for phosphorus removal; however, high concentrations of polymer should be avoided.

The filters are partially submerged to reduce stress on the media and to reduce the degree to which the media fouls. As the media fouls, water level increases above the filter until a predetermined set point is reached and backwash is initiated. The discs begin to rotate, and a pressurized stream of reused, filtered wastewater cleans the disc and allows the dirty water to flow back to the plant for re-treatment. Only 1% to 3% of the total water filtered is used for the backwash. Treatment is a continuous process and the unit remains in operation even when in backwash mode. Since the backwash is triggered automatically, no specific operator attention is required.

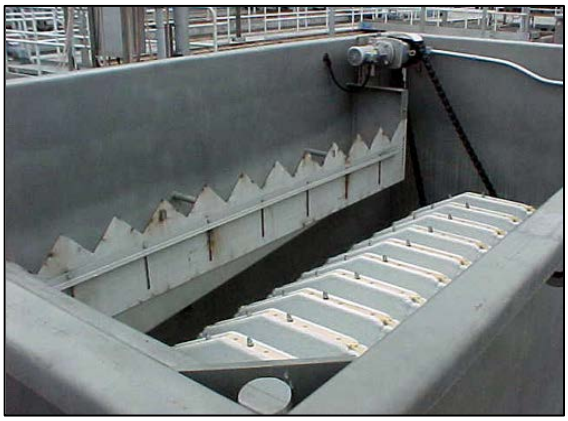

Figure 8-8 Kruger Disc Filter

One advantage to use of the discfilter is that a Kruger Discfilter has already been installed at the North Water Reclamation Facility. There will already be experienced resources available to aid in operation and maintenance. In addition, available spare parts inventory may be kept at a minimum. Materials of construction, stainless steel and FRP, should provide confidence in terms of long life without major reconstruction. Expansion of the system's capacity can be accomplished in workable degrees by the addition of discs or for larger expansions, modules may be added. Maintenance of the backwash headers and the filter segments is convenient to the operator making it much more likely that the equipment will function better and longer than were maintenance tedious.

One disadvantage is in delivery time. Twenty weeks after approval of shop drawings seems excessive, and this is assuming there are no delays. Of the three alternatives considered, the discfilter appears to be the one with the most potential maintenance activity required. It is, however the only technology, of the three options considered, that is approved for use under California's Title 22 Recycled Water Criteria.

Table 8-16 Design Criteria for Kruger DiscFilter

Parameter	Option 1	Option 2
Influent Source	Secondary Clarification	Secondary Clarification
Peak Hour Flow, mgd	2.0	1.0
Average Flow, mgd	1.0	0.5
Peak Influent TSS, mg/L	< 20	< 20
Average Effluent TSS, mg/L	<5	<5
Number of units	1	1
Number of Discs per Unit	6	3
Media Type	Woven Polyester	Woven Polyester
Total filter area per unit, sf	235	117
Submerged filter area per unit, sf	235	117
Filter Pore Size	10 um	10 um

Design criteria for a discfilter at the SWRF are presented in Table 8-16. With Option 1, the filter can process an average daily flow of 1.0 mgd and a peak daily flow of 2.0 mgd. Option 2 has half the

hydraulic capacity. The cost difference between Option 1 and Option 2 is only $20,000. The Town may elect to construct a basin large enough to accommodate the larger filter and add filter discs as needed.

Estimated costs for material and equipment for major components of the Kruger discfilter are shown in Table 8-17. The filter would be installed outside within a concrete basin. The basin could be located either south of the existing process building near the secondary clarifiers or just north of the process building and closer to the lagoon cells. Estimated costs include the filter components, construction of the concrete basin, process piping, electrical work, and ancillary equipment. Estimated costs do not include any upgrades to the site electrical service if required. The estimated construction cost to convert the SWRF from activated sludge to a 1.6 mgd IFAS process is shown in Table 8-17.

Table 8-17 Estimated Construction Cost for Kruger Discfilter.

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	100	cy	10	1,000
Concrete	50	cy	500	25,000
Cover with Hatch	1	ls	15,000	15,000
Cloth Media Filter	1	ls	183,000	183,000
Valves	1	ls	25,000	25,000
Sitework, Supports and Anchors	1	ls	5,000	5,000
Connection Piping	150	lf	220	33,000
Electrical	20%	Percent		57,400
Subtotal				**344,400**
Mobilization/Demobilization	6%	Percent		20,664
Contractor OH&P	12%	Percent		41,328
Insurance and Bonds	3%	Percent		10,332
Subtotal				**416,724**
Contingency	20%	Percent		83,000
Total Estimated Construction Cost				**499,724**
Engineering	15%	Percent		75,000
Total Estimated Project Cost				**$574,724**

A major advantage of the discfilter is that it does not need to be housed within a building. Kruger discfilters are situated in outdoor tanks at the Plum Creek Wastewater Treatment Facility in Castle Rock, Colorado. Operations staff discovered nuisance algae growth on the filter after initial installation, but was able to eliminate the problem by building a cover for the filter tanks.

Ultrafiltration – Miller-Leaman. Ultrafiltration will produce the cleanest effluent of the three processes proposed. Total suspended solids greater in size than 0.1 micron are removed making turbidity or cloudiness 0.1 NTU or less; the effluent has the same appearance as drinking water. It will reduce or eliminate the need for disinfection following filtration. Whether or not a disinfection process step may be eliminated will be a

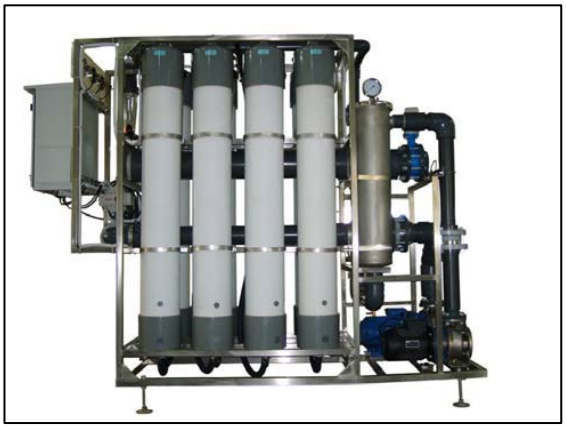

Figure 8-9 Ultrafiltration Unit

regulatory agency decision. The technology removes bacteria and viruses. It is a low pressure hollow tube membrane system that resembles long strands of spaghetti. The water to be filtered is on the outside of the membrane filter strand and is filtered through to the inside by pressure differential. Air scour is provided to extend the periods between cleanings. The unit is sized to allow a comfortable daily filtration rate of 45 gallons per square foot of membrane media.

The major advantage is that it produces the best effluent of the three alternatives. Pollutants physically larger than 0.1 micron simply cannot end up in the effluent unless there is a break in a membrane. Once placed in operation, it is relatively automatic. The operator needs to periodically check settings and pressure readings; however, in the absence of changes, little needs to be done. Attention needs be paid over time as deterioration of the membrane surfaces will be slow. Periodic chemical cleaning will be required as scale and excess slime builds on the membrane surfaces. This technology/equipment is manufactured in Florida and is installed primarily in the eastern United States. Miller-Leaman, the manufacturer, has not applied for acceptance under California's Title 22 Recycled Water Criteria.

The manufacturer has proposed their ML-36 unit which is capable of filtering 360 gpm or 0.518 mgd at peak hour conditions. Average flows should be half the peak hour to avoid permanently fouling the membranes. The ML-36 units are modular as shown in Figure 8-9. Two units at $200,000 each would be needed to treat an average daily flow of 0.5 mgd and four units would be needed to treat an average daily flow of 1.0 mgd.

Table 8-18 Estimated Construction Costs for Miller-Leahman Ultrafiltration Filter

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	100	cy	10	1,000
Concrete	50	cy	500	25,000
Building	200	sf	200	40,000
Miller-Leahman Filter	1	ls	800,000	800,000
Valves	1	ls	25,000	25,000
Sitework, Supports and Anchors	1	ls	5,000	5,000
Connection Piping	150	lf	220	33,000
25 hp Vertical Turbine Pump	2	ea	23,000	46,000
Wetwell for system feed	1	ls	40,000	40,000
Electrical	20%	Percent		203,000
Subtotal				**1,218,000**
Mobilization/Demobilization	6%	Percent		20,664
Contractor OH&P	12%	Percent		41,328
Insurance and Bonds	3%	Percent		10,332
Subtotal				**1,290,324**
Contingency	20%	Percent		258,000
Total Estimated Construction Cost				**1,548,324**
Engineering	15%	Percent		232,000
Total Estimated Project Cost				**$1,780,324**

Estimated costs for material and equipment for major components of the Ultrafiltration filter are given in Table 8-18. The ultrafiltration filter is the most costly of the three filters evaluated by a factor of two.

The filter does not fit within the hydraulic profile of the existing facility and would need a small pump station to serve it. The filter units must be placed within a building to prevent freezing and to protect the electronics. Because of the high costs associated with this filter technology, it was not considered further.

Volcano Continuous Downflow Filter – Lighthouse Filters. The Volcano Sand Filter is a continuously, backwashing filter which eliminates the need to shut down for back wash like conventional sand filters. Wastewater to be filtered enters the unit above the granular media. As it flows downward through the media, solids and solids-associated pollutants are retained in the granular media. The filtered water flows out through access ports. At the bottom of the layer of granular media, the dirty media is pumped upwards for cleaning. This cleaning is a continuous action. An air-lift pump raises the dirty media to a washing apparatus that lets the dirty water flow back for retreatment and the cleaned media to fall back into the filtration layer. The process continues in this cyclic manner. The unit requires little operator attention beyond periodic checks to ensure that media is in fact being returned and cleaned. Attention to characteristics of the wastewater applied is also necessary to be aware of any likelihood of the formation of mud balls or scaling areas.

This alternative demands the least operator attention of the three, and it produces the poorest quality effluent of the three. All three provide tertiary quality effluents. The unit as presented is comfortably sized for 800 gallons per minute at the conventional filtration rate of 4 gallons per minute per square foot. With regard to California's Title 22 Recycled Water Criteria, the Volcano technology had not submitted sufficient documentation for the California Department of Public Health (CDPH) to make an informed acceptance. While the technology has been listed in the CDPH documentation, it has yet to be accepted.

Table 8-19 Estimated Construction Cost for Volcano Filter

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	100	cy	10	1,000
Concrete	50	cy	500	25,000
Cover with Hatch	1	ls	15,000	15,000
Miller-Leahman Filter	1	ls	187,600	187,600
Valves	1	ls	25,000	25,000
Sitework, Supports and Anchors	1	ls	5,000	5,000
Connection Piping	150	lf	220	33,000
25 hp Vertical Turbine Pump	2	ea	23,000	46,000
Wetwell for system feed	1	ls	40,000	40,000
Electrical	20%	Percent		75,520
Subtotal				**453,120**
Mobilization/Demobilization	6%	Percent		20,664
Contractor OH&P	12%	Percent		41,328
Insurance and Bonds	3%	Percent		10,332
Subtotal				**525,444**
Contingency	20%	Percent		105,000
Total Estimated Construction Cost				**630,444**
Engineering	15%	Percent		95,000
Total Estimated Project Cost				**$725,444**

The Volcano filter will require a small pump station to feed the filter. Although the construction cost estimate presented in Table 8-19 doesn't include a building, the filter should be housed to prevent freezing. A building will increase construction costs substantially.

Recommendation. The Kruger disc-filter is the least costly of the three alternatives evaluated. It has been used in Colorado for phosphorus removal and for generating reuse quality water. It meets California Title 22 reuse regulation requirements. Two major advantages of this filter are that it fits within the hydraulic profile of the SWRF so a pump station is not needed and that it does not need to be inside a building. Erie selected the same technology for the NWRF. Utilizing it at the SWRF will reduce operator training requirements and spare parts inventories. The Kruger disc filter is the recommended filter.

BIOLOGICAL PHOSPHORUS REMOVAL

Biological phosphorus removal can achieve effluent phosphorus concentrations of 1 mg/L without chemical addition. It is accomplished by a specialized group of facultative, heterotrophic bacteria called the PAOs or phosphate accumulating organisms. The PAOs are thought to be *Acintobacter* but this has not been proven definitively in a laboratory setting. PAOs have a unique ability to accumulate up to 40% of their cell mass as polyphosphates through a process called luxury uptake. The PAOs can be forced into doing luxury uptake by sequentially exposing them to anaerobic and aerobic conditions.

In the anaerobic zone, the PAOs are unable to breathe because there isn't any dissolved oxygen, nitrate, or sulfate available. The anaerobic zone contains high concentrations of volatile fatty acids (VFAs). These short chain hydrocarbons are the preferred food source of PAOs. Under normal growth conditions, the PAO bacteria are unable to pick up and store the VFAs while they are in the anaerobic zone because they don't have access to an energy source like oxygen or nitrate. As the PAOs cycle between anaerobic and aerobic conditions, they will begin to accumulate excess phosphorus which is stored as large poly-P granules within the bacteria. Phosphate-phosphate chemical bonds are energy rich. This is why all cells use Adenosine Tri-Phosphate (ATP) as an internal battery. The PAOs take advantage of this chemical characteristic and use it to store large quantities of energy while they are in the aerobic zone. As soon as they are cycled back into the anaerobic zone, the PAOs use the stored energy to pick up VFAs. They release phosphorus in the process. The VFAs are combined into a new compound called poly-B-hydroxybutarate or PHB.

The MLSS is normally between 1 and 2 percent phosphorus by weight. In a biological phosphorus removing activated sludge process, phosphorus concentrations may be as high as 5 or 10 percent. Because new bacteria are being grown in the system all the time, phosphorus is removed from the wastewater when high-phosphorus sludge is wasted from the process.

Any of the activated sludge processes evaluated for the SWRF can be modified to accommodate biological phosphorus removal. A small anaerobic zone is added to the front of the biological process. The hydraulic retention time should be between 30 minutes and 2 hours with 45 minutes being typical. Return activated sludge and raw influent are directed through the anaerobic zone. An anaerobic zone may be added to any of the activated sludge processes evaluated for the SWRF for minimal cost.

Unlike nitrification, biological phosphorus removal is not highly dependent on sludge age or water temperature. It is dependent on the influent BOD to P ratio. The minimum ratio of BOD to P needed to consistently achieve effluent P concentrations below 1 mg/L as P is 20:1. At lower ratios, there may not be enough soluble BOD available to accommodate both phosphorus and nitrate removal.

CHEMICAL PHOSPHORUS REMOVAL

Chemical precipitation with either metal salts or lime is widely practiced at wastewater facilities to remove phosphorus. Chemical precipitation can be done in combination with biological phosphorus removal or independently. Chemical precipitation can achieve effluent phosphorus concentrations as low as 0.4 mg/L when used alone or as low as 0.05 mg/L when done in conjunction with tertiary filtration.

The two most common metal salts used are aluminum sulfate (alum) and ferric chloride. Lime may also be used. Chemical precipitation removes only the orthophosphate fraction of phosphorus. Influent ortho-phosphate is typically between 50 and 80 percent of total phosphorus. Polyphosphates will not react with metal salts or lime, but will be converted to phosphate during biological treatment. Organically bound phosphorus makes up the smallest fraction of total influent phosphorus and is typically less than 1 mg/L. Colloidal and particulate phosphorus will be removed by sedimentation processes. If chemical precipitation is done after secondary clarification, most of the influent phosphorus will have either been removed by sedimentation or been converted to the ortho form.

The metal salt dose is commonly described in terms of the moles of metal added (Me_{dose}) per mole of soluble phosphorus in the influent (P_{ini}). At relatively high effluent P concentrations (above 1 mg/L), there is a one to one relationship between the amount of metal salt added and phosphorus precipitated. When lower effluent phosphorus concentrations are needed, the molar ratio of metal salt required increases exponentially. Competing chemical reactions in the wastewater begin to consume the metal salt as phosphorus concentrations decrease. Ferric doses between 1.5 and 2.0 Me_{dose}/P_{ini} Ratio remove 80 to 98% of influent P. Me_{dose}/P_{ini} Ratios of 6 to 7 are needed to achieve effluent phosphorus concentrations below 0.10 mg/L (USEPA August 2010).

An unfortunate side effect of chemical precipitation is an increase in overall sludge production. Table 8-20 compares the increase in sludge production expected depending on where chemical precipitation is done in the treatment train and on the effluent phosphorus concentrations achieved. Chemical precipitation at the SWRF would be done after secondary clarification and prior to filtration. As a result, sludge production is expected to increase between 10 and 40 percent.

Chemical phosphorus removal will require a heated chemical storage building, a mixing tank, in-line probes for pH and temperature monitoring, and tertiary filtration.

Table 8-20 Estimated Increase in Sludge Production from Chemical Phosphorus Removal

Treatment Location	Increase in Sludge Production	
	Process	Total
Metal salts to primary clarifier	50 – 100%	60 – 70%
Metal salts to secondary treatment to achieve effluent P in the range of 0.5 – 1.0 mg/L	35 to 45%	5 – 25%
Tertiary application of metal salts to achieve effluent P less than 0.1 mg/L	45 to 60%	10 to 40%

Source: (USEPA August 2010)

An alternative proprietary system for chemical phosphorus removal is Blue Water's Blue PRO reactive filtration process. The Blue PRO filtration system absorbs phosphorus onto a hydrous ferric oxide coated sand media rather than relying on chemical precipitation alone. The Blue PRO process was successfully pilot tested at the Georgetown WWTP in 2009. This system has reliably produced effluent phosphorus concentrations below 0.05 mg/L using a chemical dose of only 10 mg/L as Fe which is a significant reduction in chemical use over traditional precipitation chemistry. Published costs for the BluePro system equipment are $178,300 for a 1 mgd system and $494,000 for a 3 mgd system. A heated building, monitoring equipment, piping, and SCADA interface would also be required for a complete system. The manufacturer estimates annual operations and maintenance costs, including power and chemicals, at $30,000 per year for a 1 mgd unit and $84,000 per year for a 3 mgd unit.

REUSE WATER STORAGE

The SWRF location has two, unused lagoon pond cells from the original treatment plant. The first cell has a volume of 2.5 million gallons and the second pond has a volume of 2.1 million gallons. The ponds have been off-line since the original SWRF mechanical facility was constructed in 1998. The ponds could be repurposed for reuse water storage. The ponds will need some earthwork to remove vegetation, reshape the slopes, and prepare the surface for a synthetic liner. Construction cost estimates for preparing the ponds for reuse water are presented in Table 8-21.

Table 8-21 Construction Cost Estimate for Preparing Ponds for Reuse Water Storage

Item	Quantity	Unit	Unit Cost	Est. Cost
1. Lagoon Dimensions = 200'x 337.5'; Volume = 2.5 MG*				
a. Dewatering	1	ls	$20,000	$20,000
b. Surface Excavation	2,000	cy	$10	$20,000
c. Lagoon Surface Preparation**	1	ls	$35,000	$30,000
d. New Liner – Material***	1	ls	$170,000	$170,000
e. New Liner – Labor***	1	ls	$45,000	$45,000
2. Lagoon Dimensions = 400'x 137.5'; Volume = 2.1 MG*				
a. Dewatering	1	ls	$20,000	$20,000
b. Surface Excavation	2,500	cy	$10	$25,000
c. Lagoon Surface Preparation**	1	ls	$30,000	$30,000
d. New Liner – Material***	1	ls	$160,000	$160,000
e. New Liner – Labor***	40	cy	$40,000	$40,000
3. Mobilization	1	ls	$15,000	$15,000
Subtotal Estimated Cost				$575,000
Contingency (20%)				$115,000
Subtotal				$690,000
Engineering (15%)				$103,500
Estimated Total				$793,500

* A 1:1 side slope was assumed for both lagoons.
** Surface preparation includes fill and compaction.
*** Material and Labor costs were provided by a lagoon lining company (Flexi-Liner Corp.)
Contractor's overhead, profit, insurance and bond costs are built in to each line item.

SOLIDS THICKENING

Solids destruction within the aerobic digesters is primarily a function of basin liquid temperature and solids retention time. To meet EPA requirements for well-stabilized, Class B biosolids, sludge must be digested aerobically for a minimum of 40 days at 20 degrees C or for 60 days at 15 degrees C. The basin volume required to achieve a solids retention time (SRT) of 60 days was calculated using typical WAS flow rate and concentrations for 2009 operating conditions. The estimated capacity of the aerobic digesters is equivalent to 1.2 mgd of influent flow when the digester temperature is at 15 degrees C and the digester total solids content is 1.5 percent. Digester capacity can be increased simply by operating with a higher solids concentration. To treat 1.6 mgd of influent flow, the digester solids concentration needs to be near 3 percent.

Operations staff currently thicken digested solids by adding solids to the digesters followed by multiple rounds of settling and decanting. Because of concerns over generating odors, settling and decant periods are kept as short as possible. Solids concentrations of 1.5 to 2.0 percent have been achieved through this method over the past several years. Longer settling periods or the use of chemical coagulants may increase the sludge concentration to 3 or 4 percent, but would likely generate intolerable odors. There are single-family homes within 600 feet of the SWRF property line. Installing solids thickening alongside the digesters would allow staff to either thicken solids prior to placing them in the digester, continuously remove solids from the digester for thickening during digestion, or to thicken post-digestion. Thickening the solids would have multiple benefits including increased digester capacity, minimization of odors, and decreased sludge hauling costs.

The only technology evaluated for thickening was a rotary drum thickener although the solids screw press selected for the NWRF could also be applied at the SWRF. Belt filter presses and centrifuges were not considered because they are typically used for dewatering rather than thickening, require constant operator supervision, and have larger footprints than a rotary drum thickener.

Rotary Drum Thickener. Rotary drum thickeners, shown in Figure 8-10, consist of a thickening system (including polymer feed system) and rotating cylindrical screens. Polymer is mixed with dilute sludge in the mixing and conditioning zone. The conditioned sludge is then passed to rotating screen drums which separate the flocculated solids from the water. Capture rates of up to 98% are typical. A variable speed drive unit rotates the drum at approximately 5 to 20 rpm. An internal rotating helical screw conveys the thickened solids along the interior surface to the discharge port. Thickened sludge discharges into a sump and is pumped to the sludge stabilization process. The liquid fraction (filtrate) is recycled to the liquid treatment stream.

Figure 8-10 Rotary Drum Thickener

Rotary Drum Thickeners are used primarily to thicken biological sludge from secondary treatment processes. Rotary drum thickeners are typically not used to thicken primary sludge alone because the material can be handled with less sophisticated, lower cost equipment. In addition, abrasive grit (sand) in primary sludge increases wear on the equipment. The SWRF does not have primary clarifiers so the drum thickener would be used to thicken WAS or digested sludge only.

Drum thickener success with municipal WAS is variable and dependent on solids characteristics. Polymer requirements are a concern because of floc sensitivity and shear potential in the rotating drum. The thickener uses a rotating drum with wedge wires, perforations, stainless steel fabric, polyester fabric or a combination of stainless steel and polyester fabric as the porous media. Rotary drum thickeners can thicken waste activated sludge from 0.5 to 1.0 percent solids to between 4 and 9 percent solids (Metcalf and Eddy 2003). Rotary drum thickeners are available in capacities up to 400 gpm.

Rotary drum thickeners work well for smaller facilities because they are relatively simple to operate and maintain. In Colorado, they have been used at Carbondale, Telluride, and Durango. Rotary drum thickeners offer the flexibility of varying process performance with sludge feed rate, polymer feed rate, and drum speed adjustments.

Design criteria and costs for a rotary drum thickener are summarized in Tables 8-22 and Table 8-23, respectively. The drum thickener could be placed next to the existing digesters within a small building. Thickening may be done either continuously as WAS enters the digester or periodically to thicken digester contents. In the second scenario, digested solids would be removed from the digester and thickened with the rotary drum thickener. Thickened solids would be returned to the digester while the filtrate or liquid is returned to the influent pump station wet well.

Table 8-22 Rotary Drum Thickener Design Criteria

Parameter	Criteria
Number	1
Flow Each, gpm	264
Feed Solids, %	1.0
Dry Solids per Hour	1320
Dry Solids Content, %	4 – 8

Table 8-23 Estimated Construction Cost for Rotary Drum Thickener

Item	Quantity	Units	Unit Price, $	Cost, $
Excavation	100	cy	10	2,000
Buried Piping	1	ls	20,000	20,000
Concrete	10	cy	500	5,000
Building	120	sf	200	24,000
Rotary Drum Thickener	1	ls	140,000	168,000
Polymer System	1	ea	50,000	55,000
Thickened Sludge Pump	2	ea	30,000	72,000
Valves	1	ls	50,000	60,000
HVAC	15%	Percent		61,000
Electrical	20%	Percent		81,000
Subtotal				**548,000**
Mobilization/Demobilization	6%	Percent		33,000
Contractor OH&P	10%	Percent		55,000
Insurance and Bonds	3%	Percent		16,000
Subtotal				**652,000**
Contingency	25%	Percent		163,000
Total Estimated Construction Cost				**815,000**
Engineering	15%	Percent		122,300
Total Estimated Project Cost				**937,300**

The rotary drum thickener could be located in a small building immediately north of the aerobic digesters. Thickened sludge pumps will be needed to return sludge to the digesters.

LEVELS OF TREATMENT

The SWRF received its final permit in September 2011 and the NWRF final permit was issued in January 2011. Both permits contain low effluent ammonia limits for both facilities, but do not include limits for either nitrate or phosphorus. The NWRF is capable of removing ammonia to the permit limits, nitrate, and phosphorus. The SWRF cannot consistently meet an effluent ammonia concentration below 1 mg/L as it is currently configured and operated. Effluent ammonia concentrations for this facility range between 1.9 and 4.4 mg/L as N.

Due to a myriad of factors related to classifying the South Platte River as a drinking water supply, it is possible in the near future that the State of Colorado may establish a total inorganic nitrogen (TIN) allocation on the South Platte River. A TIN allocation accounts for ammonia nitrogen and nitrate nitrogen levels. This allocation would attempt to ensure that at the drinking water intakes, the nitrate levels would be below the drinking water standard.

The recommended alternative for the SWRF is an integrated fixed film activated sludge (IFAS) process. IFAS is capable of meeting the permit effluent ammonia concentrations and can remove nitrate to below 10 mg/L as N. The process is compatible with both biological and chemical phosphorus removal. Biological phosphorus removal can reduce effluent P to approximately 1 mg/L. Further reductions require chemical addition followed by tertiary filtration. Several different effluent filters were evaluated for the SWRF.

PUBLIC PARTICIPATION

The Town of Erie maintains an informational website where upcoming and current project information is posted for public review and comment. Draft and final reports of planning efforts are posted.

The Town has an open house for capital improvement projects (CIP) in January of each year. Representatives from Indigo Water Group and Farnsworth Group attended on CIP open house to answer questions. Major CIPs are presented at an open house for the specific major project to invite public comment. Landowners around the major project are mailed an invitation to the meeting and the meeting notice is posted on the Town website.

The Town of Erie holds Board of Trustee meetings on the 2nd and 4th Tuesdays of each month with minor variations to accommodate major holidays. All meetings are open to the public with meeting agendas available online at least 24 hours prior to each meeting. Meeting packets provided to the Board are also available to the public and are posted on the Town of Erie's website at least 24 hours prior to each meeting.

SECTION 9
MANAGEMENT AND FINANCIAL PLANS

This section presents the management structure for the Town of Erie as well as its financial planning. In January 2009, Red Oak Consulting completed a Water and Wastewater Rates and Fees study for the Town of Erie (Red Oak Consulting January 2009). Selected portions of the rate study can be found in Appendix K. The rate study included financial planning for construction of the North Water Reclamation Facility (NWRF) and the NWRF interceptor.

MANAGEMENT STRUCTURE AND AGREEMENTS

The Town of Erie is a Statutory Town. Colorado Statutory Towns operate under Title 31, Article 1, Section 203 and Article 4, Part 3 of the Colorado Revised Statutes. Statutory Towns have an elected Mayor and a Board of Trustees composed of the Mayor and four or six additional members elected at large. Erie's Board of Trustees consists of the Mayor, the Mayor Pro Tem, and five trustees. All are elected positions. Trustees serve four year terms with three being eligible for reelection every two years. Staggering terms provides for overlap and continuity.

The board maintains a complete set of rules and regulations that are used to uniformly enforce rates, construction codes, and other policies. Board meetings are held twice per month on the second and fourth Tuesdays. Board meetings are open to the public.

The Town of Erie owns, operates, and manages its own wastewater reclamation facilities and collection system. The Town of Erie includes land in both Weld and Boulder Counties. Each county falls under the jurisdiction of its own water quality management agency. Because both of Erie's Water Reclamation Facilities are within Weld County, North Front Range Water Quality Planning Association is the primary water planning agency for the Town. Portions of the Town's planning area are within Boulder County which were previously managed by Denver Regional Council of Governments. The water quality planning portion of DRCOG was disbanded in 2010 with all review responsibilities returning to the State level.

The Town accepts wastewater from within its planning area boundary. There are no IGAs in place that obligate Erie to provide wastewater service to areas outside of its planning boundary.

PRELIMINARY OPERATING PLAN

Both the NWRF and SWRF utilize activated sludge for secondary treatment. Regulation 100 requires the operator in responsible charge (ORC) for activated sludge facilities with permitted capacities between 1.01 and 4.0 mgd to hold a Class B operator's license. Jon Mays, the ORC for both the NWRF and SWRF, holds a Class A license. The Town of Erie plans to use existing staff to operate both WRFs. The Town currently has three full-time and one part-time operators. The NWRF became operational in early 2011. At that time, flows will be diverted from the SWRF completely to the NWRF. This will allow staff to take the SWRF off-line for repairs. There are two diversion structures at the SWRF which allow

the Town to divert some or all of the wastewater generated upstream of the SWRF to the NWRF. The Town may elect to convert the SWRF to an IFAS process while the NWRF is treating all of the flow generated in the service area. This will simplify construction since the SWRF can be taken completely out of service for the entire construction period.

WASTEWATER MANAGEMENT PLAN

The wastewater utility is a self-supporting enterprise fund with funding for annual operations and maintenance expenses, capital projects, and debt service met primarily through wastewater service rates and tap fees. The Town maintains two separate utility operating funds for water and wastewater. The planning objectives of each of these funds are similar.

- To provide service to the Town's customers in a manner that will protect the environment.
- Develop infrastructure capacity to meet growth demands.
- Emphasis on renovation and replacement of infrastructure and overall system reliability through programmed renovation and ongoing maintenance.
- Charge rates that cover costs of O&M, renovation and replacement and debt service.
- Charge tap fees that cover growth related capital improvements.
- Planned, regular rate and tap fee adjustments for financial stability and customer acceptance.

The wastewater fund is separate from the Town's General Fund and is not dependent on any taxes or other revenue sources.

FINANCIAL MANAGEMENT PLAN

The Town completed a Water and Wastewater Rate Study in January 2009 (Red Oak Consulting January 2009). The primary goal of the study was to ensure that an adequate level of revenue from wastewater service revenue is maintained to finance Erie's daily operations as well as future capital improvements and expansions. In December 2008, the Board of Trustees adopted the wastewater rates recommended in the study. Red Oak used many assumptions in the development of their wastewater rate study. Changes made to any of the following assumptions could materially affect these results:

- Wastewater utility debt service coverage requirement equal to 1.10x net revenues
- Annual growth of 300 accounts per year.

The rate study categorized wastewater funds into one of three categories: Capital Expansion, Capital Replacement, or Operating Fund. Separate financial forecasts were made for each category to better determine sufficiency of revenues collected using existing rates to meet revenue requirements during the five-year study period; 2009 through 2013.

The capital expansion fund is associated with growth related capital improvements, which, for the study period totaled $9.8 million for improvements including the north side interceptor. The $20.6 million cost of the NWRF construction was assigned to year 2008 for planning purposes. Sources of funding included tap fee revenue, developer contributions, bond proceeds, transfers from the operating fund, and interest income. The average revenue from tap fees was $1.3 million annually. The averaged existing and

proposed debt service payment was $2.1 million, which included an annual inflation allowance of 4.5 percent.

The capital repair and replacement fund is associated with repair and replacement of existing infrastructure. For the study period, repairs and replacement totaled $3.4 million including slip lining the collection system in Old Town, master plan updates, and a water reclamation facility land buffer. Sources of funding included transfers from the operating fund, bond proceeds, and interest income. Beginning in 2009, annual transfers from the operating fund will average $700,000. The annual inflation allowance of 4.5 percent is included.

The operating fund is associated with annual operating revenues and revenue requirements, which include operation and maintenance expense, transfers to the capital expansion fund as well as the capital repair and replacement fund, and debt payments. The operating fund had an existing balance of $3.7 million at the beginning of 2009. The source of revenue is from service rates, averaging $2.2 million annually, and from interest income, which is calculated at a 2.0 percent annual interest rate on the average fund balance. Debt payments average $123,286 annually over the study period.

Reliance on plant investment fees or system development charges to cover capital costs is not prudent due to the cyclical nature of development and the corresponding cash flow irregularities. Operating reserves provide flexibility to moderate impacts from cash flow irregularities.

TWENTY-YEAR PLANNING PERIOD FINANCIALS

Table 9-1 presents planned capital improvement projects (CIPs) funded by the Town of Erie between 2011 and 2029. Additional interceptors, temporary lift stations, and permanent lift stations will be funded through developer contributions in support of the specific geographic areas they will serve. For budget purposes, the SWRF conversion to IFAS to meet more stringent effluent limits for ammonia and nitrate is shown taking place over three years from 2012 to 2014. Design of improvements would take place in 2012 with construction being done over the following two years. The first major physical expansion of the NWRF is shown taking place in 2025 with design in 2024. Other significant capital expenditures include the North Side Interceptor in 2018, Interstate Interceptor in 2019, extension of sanitary sewer to the Airpark residential areas in 2022, Arapahoe Ridge capacity improvements in 2023, and Old Town capacity improvements in 2023. A list of anticipated CIPs is included in Appendix L.

Tables 9-2 and 9-3 on the following page show the 20-year projected cash flow and expenses including capital projects. Cash flow projections for taps, tap fees, and service revenue are based on the population growth percentages discussed in Section 4. Tap and service fee revenues are assumed to increase with time in accordance with the recommendations in the Town's *Water and Wastewater Rate Study* completed in January 2009 (Red Oak Consulting). Expenditures include debt services, operations, and capital improvement projects. Tap and service fees are adequate to fund projected maintenance and CIP costs.

Table 9-1 Town of Erie
2012 Budget Worksheet
Five Year CIP Summary

Project Title	Budgeted 2011	Projected 2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	Total
GIS Development Project	30,100	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	30,100
North Water Reclamation Facility	3,295,900	300,000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	3,595,900
North WRF Influent Pipeline	521,500	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	521,500
Wastewater Utility Plan	19,400	-	-	-	-	100,000	-	-	-	-	130,000	-	-	-	-	169,000	-	-	-	418,400
Collection System Rehabilitation	312,900	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	312,900
Flow Monitoring Equipment (2 units)	20,000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	20,000
NWRF Reuse Pump Station	1,039,300	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1,039,300
Extension of Sewer Service to Com'l Contingency	425,000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	425,000
Decommisioning of Lift Stations	75,000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	75,000
SWRF Modifications	-	200,000	2,000,000	2,500,000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4,700,000
Wastewater Discharge Pad	-	36,300	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	36,300
Sanitary Sewer to Airpark - Line A1 & D	-	-	-	-	-	-	-	-	-	280,000	840,000	-	-	-	-	-	-	-	-	1,120,000
Sanitary Sewer to Airpark - Line C	-	-	-	-	-	-	-	-	-	112,000	560,000	-	-	-	-	-	-	-	-	672,000
Airpark Sewer Ext to West of Coal Creek	-	-	-	-	-	-	-	-	-	84,000	490,000	-	-	-	-	-	-	-	-	574,000
North Side Interceptor	-	-	-	-	-	-	901,000	6,625,000	-	-	-	-	-	-	-	-	-	-	-	7,526,000
Sanitary Sewer to Airpark - Residential Areas	-	-	-	-	-	-	-	-	-	-	302,400	2,520,000	-	-	-	-	-	-	-	2,822,400
Northside Lift Station & Force Main P1	-	-	-	-	-	-	450,000	-	-	-	-	-	-	-	-	-	-	-	-	450,000
Northside Lift Station & Force Main P2	-	-	-	-	-		-	3,600,000	-	-	-	-	-	-	-	-	-	-	-	3,600,000
Interstate Interceptor	-	-	-	-	-	-	-	-	5,625,000	-	-	-	-	-	-	-	-	-	-	5,625,000
Arapahoe Ridge Capacity Improvements	-	-	-	-	-	-	-	-	-	-	-	-	1,416,000	-	-	-	-	-	-	1,416,000
Westside Capacity Improvements	-	-	-	-	-	104,000	-	-	-	-	-	-	-	-	-	-	-	-	-	104,000
Old Town Capacity Improvements	-	-	-	-	-	-	-	-	-	-	-	-	1,635,000	-	-	-	-	-	-	1,635,000
Austin Industrial Line	-	-	-	-	-	-	-	-	-	-	-	-	356,000	-	-	-	-	-	-	356,000
NWRF Expansion	-	-	-	-	-	-	-	-	-	-	-	-	-	300,000	6,000,000	-	-	-	-	6,300,000
	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	5,739,100	536,300	2,000,000	2,500,000	-	204,000	1,351,000	10,225,000	5,625,000	476,000	2,322,400	2,520,000	3,407,000	300,000	6,000,000	169,000	-	-	-	43,374,800

Table 9-2 Town of Erie Wastewater Cash Flow 20 Year Projection

Year	Tap per Year[1]	Cumulative Taps[2]	Total Tap Fees Collected[3]	Usage Fees[4]	Projected Income	Projected Expenditures[5]	Earnings on Reserves
2010		6,216		2,764,373	8,017,177		8,017,177
2011	100	6,316	429,000	3,132,035	3,561,035	9,159,200	2,419,012
2012	100	6,416	429,000	3,548,595	3,977,595	4,248,149	2,148,458
2013	320	6,736	1,372,800	4,020,558	5,393,358	5,819,391	1,722,425
2014	339	7,075	1,455,168	4,555,293	6,010,461	6,431,683	1,301,202
2015	360	7,435	1,653,939	4,919,716	6,573,655	4,053,651	3,821,207
2016	374	7,809	1,720,097	5,313,293	7,033,390	4,378,231	6,476,366
2017	389	8,198	1,788,901	5,738,357	7,527,257	5,656,567	8,347,056
2018	404	8,602	1,860,457	6,197,425	8,057,882	14,665,652	1,739,286
2019	421	9,023	1,934,875	6,693,219	8,628,094	10,207,148	160,233
2020	437	9,460	2,143,505	7,228,677	9,372,182	5,210,435	4,321,979
2021	455	9,915	2,229,245	7,806,971	10,036,216	7,215,305	7,142,891
2022	473	10,388	2,318,415	8,431,529	10,749,944	7,577,607	10,315,228
2023	492	10,880	2,411,152	9,106,051	11,517,203	8,641,511	13,190,919
2024	512	11,392	2,507,598	9,834,535	12,342,133	5,717,506	19,815,546
2025	532	11,924	2,788,858	10,621,298	13,410,156	11,608,275	21,617,427
2026	554	12,478	2,900,412	11,471,002	14,371,414	5,979,846	30,008,996
2027	576	13,053	3,016,429	12,388,682	15,405,111	6,021,831	39,392,276
2028	599	13,652	3,137,086	13,379,776	16,516,862	6,241,734	49,667,404
2029	623	14,275	3,262,569	14,450,159	17,712,728	6,421,084	60,959,048

[1]Taps sold per year. Estimates are based on growth projections. 2014 - 2015 assume 6% increase. 2017 - 2029 assume 4% increase.

[2]Total taps in the system. Previous year + current year.

[3]Tap Fees Collected is the product of tap sales and the associated tap fee; no annual adjustment is assumed. Tap Fee = $4,290 through 2014; $4,600 through 2019; $4,900 through 2019; $5,240 through 2029

[4]2010 is actual revenue. 2011 to 2014 assumes a 13.3% increase/ year. 2015 - 2029 assumes an 8% increase/ year.

[5]Expenditures include debt services, operations, and capital

Table 9-3 Town of Erie Projected Wastewater Expenses

Year	Debt Service[1]	WW Admin[2]	Collection[2]	WW Treatment[2]	WW Capital[3]	Total
2010	1,016,230	625,372	98,444	564,487	17,393,703	19,698,236
2011	1,578,700	805,800	148,400	887,200	5,739,100	9,159,200
2012	1,574,299	805,800	169,600	1,162,150	536,300	4,248,149
2013	1,574,963	846,090	178,080	1,220,258	2,000,000	5,819,391
2014	1,575,034	888,395	186,984	1,281,270	2,500,000	6,431,683
2015	1,579,169	932,814	196,333	1,345,334	0	4,053,651
2016	1,576,025	979,455	206,150	1,412,601	204,000	4,378,231
2017	1,577,451	1,028,428	216,457	1,483,231	1,351,000	5,656,568
2018	1,576,130	1,079,849	227,280	1,557,392	10,225,000	14,665,652
2019	1,574,400	1,133,842	238,644	1,635,262	5,625,000	10,207,148
2020	1,576,300	1,190,534	250,576	1,717,025	476,000	5,210,435
2021	1,576,862	1,250,060	263,105	1,802,876	2,322,400	7,215,305
2022	1,575,763	1,312,563	276,261	1,893,020	2,520,000	7,577,607
2023	1,578,575	1,378,191	290,074	1,987,671	3,407,000	8,641,511
2024	1,578,773	1,447,101	304,577	2,087,054	300,000	5,717,506
2025	1,577,605	1,519,456	319,806	2,191,407	6,000,000	11,608,275
2026	1,578,643	1,595,429	335,796	2,300,978	169,000.00	5,979,846
2027	1,578,018	1,675,200	352,586	2,416,026	-	6,021,831
2028	1,575,730	1,758,960	370,216	2,536,828	-	6,241,734
2029	1,521,780	1,846,908	388,726	2,663,669	-	6,421,084
Total		24,100,248	5,018,096	34,145,738	60,768,503.00	154,953,042

[1]Debt services scheduled received from Finance

[2]2013 - 2029 assumes a 5% increase each year.

[3]2010 - 2016 is the approved CIPs budget. 2017 is collective of remaining projects per WW Utility Plan

INTEREST IN STATE FINANCIAL ASSISTANCE

The rate study developed for the Town to project future user and tap fee charges assumes that all the projected capital improvements will be funded from the cash reserve generated by user and tap fees and by debt financing. Debt financing will come from the issuance of bonds or loans such as the State Revolving Loan Fund Program. The North Water Reclamation Facility (NWRF) debt was financed through revenue bonds. At this time, it is anticipated that the debt financing for projects at the South Water Reclamation Facility (SWRF) will also be financed through revenue bonds.

The Town anticipates the majority of new interceptors, sewer lines, and lift stations will be financed by private developers as development occurs. Developer constructed infrastructure is dedicated to the Town upon completion. These assets become the assets of the Town and are considered in-kind contributions. The Town may assist with financing and planning of larger interceptors where new developments are located remotely relative to existing infrastructure. In these situations, it may not be possible for a developer to absorb all of the costs associated with a new interceptor and/or lift station.

USER CHARGE STUDY SUMMARY

Sewer connection fees are one-time payments intended to cover the capital cost of future capacity. Future users are charged sewer connection fees (also known as tap fees or system development charges) when they connect to the system to pay their share of system construction costs. Sewer fees are based on an "equity buy-in" concept or "growth related cost allocation" which are accepted industry standards supported by legal precedent. Sewer connection fees must 1) be based on clearly defined needs and costs and 2) apportion costs equitably among all customers in proportion with the benefits provided.

Sewer connection fees developed by Red Oak Consulting are presented in Table 9-4. The proposed fees were adopted by the Board of Trustees in December 2008. Tap fees for single family residences increased slightly from $4,000 to $4,290 while tap fees for larger diameter taps decreased. The reallocation better apportions cost in accordance with the benefits received. Lowering tap fees for larger diameter services may encourage commercial development within the service area.

Table 9-4 2008 and 2009 Sewer Connection Fees

Meter Size, inches	2008 Tap Fee	Adopted 2009
3/4	4,000	4,290
1	8,000	7,150
1 ½	16,000	14,300
2	32,000	22,880
3	72,000	42,900
4	144,000	71,500
6	376,000	143,000

Source: Table 9-2 (Red Oak Consulting January 2009)

Monthly service charges fund operation and maintenance of the wastewater treatment facility, capital improvements to replace aging infrastructure, and periodic upgrades to existing facilities to comply with new regulatory requirements. Operation and maintenance costs are paid in cash from monthly service revenues. Capital costs for infrastructure replacement and upgrades can be paid by a combination of cash funding, grants, and loans. Monthly service charges must be sufficient to cover cash operating expenses and maintain operating reserves.

The 2009 Rate Study determined that 2008 service rates were not sufficient to cover future expenditures on maintenance and replacement activities. Red Oak Consulting recommended that service rates be increased 13.3 percent per year each year from 2009 through 2013. The proposed changes were adopted by the Board of Trustees in December 2008. Table 9-5 summarizes service rates from 2008 through 2013. A single family home generating 6,000 gallons of wastewater in a month, can expect to pay $39 per month in 2009 and $65 per month in 2013. Wastewater generation rates for residential customers are calculated based on average winter water use. Generation rates for commercial, industrial, and other non-residential accounts are calculated based on actual water use each month.

Table 9-5 Existing and Future Sewer Service Fees

Account Type	Previous 2008	Adopted 2009				
		2009	2010	2011	2012	2013
Service charge, $ per bill						
Residential	$8.30	$8.79	$10.28	$11.84	$13.56	$15.68
Commercial	$20.00	$8.79	$10.28	$11.84	$13.56	$15.68
Volume charge, $ per 1,000 gallons						
Residential	$4.39	$5.15	$6.76	$6.48	$7.31	$8.20
Commercial	$4.21	$5.15	$5.76	$6.48	$7.31	$8.20

The Town of Erie has evaluated the impact that future project costs will have on user rates and have concluded that regularly scheduled increases will cover the anticipated debt service for the capital projects. The Town's rate study predicts that the average user charge per residential customer per year will increase by approximately 6 to 7 percent every two years for the near term in order to provide sufficient revenue to cover the anticipated capital and operating costs for the system. This represents an annualized user charge increase of 3 to 4 % per year. The tap fees are expected to also increase at around 3 to 4% per SC per year (Burns and McDonnell, Inc. April 29, 2009).

Erie's 2011 budget predicted wastewater revenues of $4.14 million. Roughly 87 percent of all revenues will be generated through sewer service fees while 13 percent will be generated from tap fees. Interest income will contribute another 1 percent to the total budget (Town of Erie 2010). Budgeted expenditures total $3.5 million and include wastewater debt service (46%), collection system operation and maintenance (4%), wastewater treatment (25%), wastewater repairs and maintenance (2%), and wastewater administration (23%).

RATE COMPARISON

Plant investment and service fees for Erie were compared to similar sized municipalities. Data collected from municipal web sites is presented in Table 9-6. Municipalities smaller than 10,000 residents and larger than 40,000 residents were excluded from the survey. They were excluded to remove distortions in cost of service that result from economy of scale or lack thereof. Population data was taken from the 2010 Census Data posted on the State Demographer's web site at http://dola.colorado.gov/demog/ (State of Colorado 2010).

The current plant investment fee of $4,290 for a residential unit for the Town of Erie is almost exactly the average for Colorado municipalities listed in Table 9-3.

The average monthly service fee for the municipalities surveyed is $24.77. Erie's current monthly service charge of $50.84 is forty-nine percent higher than the average for Colorado municipalities listed in Table 9-3. According to Colorado Department of Local Affairs, the statewide average monthly sewer service fee was $15.55 in 2006 (DOLA December 2006). The statewide survey includes many smaller municipalities with lagoon wastewater treatment which have low operating costs and low monthly service fees. Erie's higher monthly service fee reflects a significantly higher level of treatment.

EPA has proposed that, if a community's average monthly residential user fee for sewer service is at or above 1 to 2 percent of the median household income, the community will not be expected to make

improvements to meet nutrient criteria. Erie's mean household income is $87,284 (Town of Erie 2010). Before EPA would consider a variance, monthly sewer fees would need to be between $72 and $144 per month. Erie's 2010 monthly service fee of $54.84 is 24 percent lower than the minimum sewer fees proposed by EPA.

In summary, the expected costs per customer for both usage and tap fees should remain within the anticipated range for other wastewater utilities in the metro area that provide a comparable level of treatment. Therefore, the improvements presented in the Utility Plan will not place an unreasonable burden on the average customer.

Table 9-6 Comparison of 2010-2011 Residential Wastewater Utility Service Charges.

Municipality	Population Served	Tap Fee	Base Rate	Usage Rate per 1,000 gallons	Monthly Service Fee at 6,000 gallons
Castle Pines North[1,2]	10,360	3,845	7.30	3.78	29.98
Fort Morgan[3]	11,315	1,200	18.67	1.88	29.95
Federal Heights[4]	11,467	5,058	Denver Metro	Denver Metro	Denver Metro
Steamboat Springs[5,6]	12,088	2,500	29.95	1.80	40.75
Superior[7]	12,483	Unknown	15.06	None	15.06
Sterling[8]	14,777	2,700	15.63	None	15.63
Erie in 2010	**18,135**	**4,290**	**10.28**	**6.76**	**50.84**
Louisville[9]	18,376	3,221	15.06	None	15.06
Evans[10]	18,537	11,600	9.89	None	9.89
Windsor[11]	18,644	3,700	20.00	1.22	27.32
Golden[12]	18,867	3,486	11.48	1.79	22.22
Freemont Sanitation District	20,000	2,260	19.38	none	19.38
Lafayette[13]	24,453	6,000	8.31	0.86	13.47
Englewood[14]	30,255	1,900	18.58	2.65	34.48
Brighton[15]	33,352	Unknown	2.15	3.77	24.77
Northglenn[16]	35,789	Built-Out	0	3.31	19.86
Broomfield[17]	38,232	8,427	11.04	2.76	27.60
Average	20,419	4,299	13.30	2.78	24.77

[1] (Town of Castle Rock Development Services 2011), [2] (Castle Pines North Metro District July 2007),
[3] (City of Fort Morgan 2010), [4] (City of Federal Heights 2009), [5] (Mount Werner Water District 2007)
[6]Calculated using spreadsheet that accounts for square footage, sinks, dishwasher, etc.
http://steamboatsprings.net/sites/default/files/2009/04/06/20090407171624427.pdf
[7] (Town of Superior 2011), [8] (City of Sterling, Colorado 2011), [9] (City of Louisville, Colorado n.d.) 2006 tap fee used.
[10]Tap fee is for water and wastewater services combined. (City of Evans, Colorado 2010)
[11] (Town of Windsor, Colorado 2009), [12] (City of Golden, Colorado 2010),
[13] (City of Lafayette, Colorado n.d.), [14] (City of Englewood, Colorado 2010),
[15] (City of Brighton, Colorado 2011), [16] (City of Northglenn, Colorado 2009). A minimum sewer charge of $9.93 per month applies to all accounts.
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