City of Evans

Wastewater Utility Plan Update

October 2013

City of Evans, Colorado

HDR

City of Evans

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Acronyms

AC	Acre
ADWF	Average Dry Weather Flow
AER	Aerobic Zone
ANR	Anaerobic Zone
BNR	Biological Nutrient Removal
BOD	Biological Oxygen Demand
BW	Backwash
CAS	Conventional Activated Sludge
CFS	Cubic Feet Per Second
CIP	Clean in Place
DRCOG	Denver Regional Council of Governments
DO	Dissolve Oxygen
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
GPM	Gallons Per Minute
HP	Horse Power
HRT	Hydraulic Retention Time
IFAS	Integrated Fixed Film Activated Sludge
IGA	Intergovernmental Agreement
I&I	Infiltration and Inflow
LTSA	Long Term Service Area
MBBR	Moving Bed Bioreactor
MBR	Membrane Bioreactor
MF	Membrane Filtration
MG	Million Gallons
MGD	Million Gallons Per Day
MHI	Median Household Income
MLSS	Mixed Liquor Suspended Solids
MPN	Most Probable Number
NFRWQPA	North Front Range Water Quality Planning

	Association
OUR	Oxygen Uptake Rate
PCL	Primary Clarifier
PEL	Preliminary Effluent Limit
PSL	Primary Sludge
RAS	Return Activated Sludge
RST	Rotating Screen Thickener
sBOD	soluble Biological Oxygen Demand
SC	Solids Contact
SCFM	Standard Cubic Feet Per Minute
SCL	Secondary Clarifier
SDWA	Safe Drinking Water Act
SF	Square Feet
SRF	State Revolving Fund
SRT	Solids Retention Time
TAZ	Transportation Analysis Zone
THS	Thickened Sludge
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
TVS	Table Value Standard
UV	Ultraviolet
VAR	Vector Attraction Reduction
VSS	Volatile Suspended Solids
WAS	Waste Secondary Sludge
WQCC	Water Quality Control Commission
WQCD	Water Quality Control Division
WET	Whole Effluent Toxicity

WTF	Water Treatment Facility
WUSA	Wastewater Utility Service Area
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant

1.0 Executive Summary

1.1. Introduction

In 2009 the City of Evans undertook the task of planning for the future of the wastewater collection and treatment system. The goals of the planning effort were to define the conditions of the existing infrastructure, evaluate growth trends and estimate future impact, describe improvements necessary to meet discharge permit requirements at the City's two wastewater treatment facilities, protect water resources, accommodate existing users and future growth, and develop a capital improvements plan and financial program that will ensure the goals of the utility are accomplished. The results of the planning effort were documented in the 2011 Wastewater Utility Plan. Due to the continued increase in flows to the Evans WWTF over the past few years, the facility required rerating to increase the capacity. This Wastewater Utility Plan Update addresses the requirements for rerating the Evans WWTF. The following executive summary briefly describes the chapter contents, conclusions and recommendations arising from this document.

1.2. Objectives of Utility Planning

The purpose of a utility plan is to provide utility managers with a vision of the fullest extent of wastewater treatment and collection possible, as well as potential diversion strategies, so that utility service can be sustained into the future. If utility managers track the progression of discharge standard development and the growth in plant loadings, a facility plan provides a road map of actions necessary to sustain the viability of the wastewater system. Tracking discharge standard development includes participation in the receiving water quality planning process and a continuing dialogue with regulatory agencies. Flow and loading development takes on an important meaning, beyond the routine collection of data, as key threshold levels are approached and planned responses implemented.

Utility managers can plan proactively for the strategies necessary for implementation of facilities. Such activities include briefing the City of Evans Council, coordinating with other agencies (CDPHE), coordinating or planning with other utility services (water service/reclaimed water production), preparing the rate payers in the community for change, financial planning (capitalization fee collection, outside grant/loan support, service offers to key developments), and so on. Utility managers can also avoid pitfalls, such as extending services beyond levels that the utility can support or allowing new development into the system without contemplating the full cost impact of offering service.

1.3. Basis of Planning

1.3.1. Population Forecasts

Determination of the demand on the wastewater collection and treatment system is dependent on land use, population density, the magnitude and type of commercial and industrial activity to be served, the condition of the existing system and regulatory

requirements. The area studied in this document was established through meetings with City Staff, North Front Range Water Quality Planning Association (NFRQWPA), Colorado Department of Health and Environment (CDPHE), surrounding communities, and by examination of property ownership, zoning, planning jurisdiction and environmental conditions. The study area boundary, as decided by the planning team, follows the Evans Urban Growth Area.

Historical growth trends for the Evans area were analyzed and meetings were held with City staff to evaluate growth trends, development densities and resulting population projections. Two data sources were used to evaluate future population projections: the Evans Comprehensive Plan and the North Front Range Metropolitan Planning Organization's Transportation Analysis Zone (TAZ) model results. Table 1-1 summarizes the development densities and unit population values used in the projections. Table 1-2 summarizes the population projections within the planning boundary. Developable acres were used in projections to account for parks and other undevelopable areas.

Table 1-1. Comprehensive Plan Development Densities and Unit Population Values

	Density (units/acre)	Unit Population (people/household)
Within the Planning Boundary		
Residential - High Density	8	1.8
Residential - Urban Neighborhood	4	2.9 (existing) / 2.5 (future)
Residential - Rural Neighborhood	1	2.9 (existing) / 2.5 (future)
Mixed Use	1.2	1.2
Outside the Planning Boundary		
Residential - High Density	8	1.8
Residential - Urban Neighborhood	2.5	2.9 (existing) / 2.5 (future)
Residential - Rural Neighborhood	0.4	2.9 (existing) / 2.5 (future)
Mixed Use	1.2	1.2

Table 1-2. Theoretical Buildout Population Projection Summary within the Planning Boundary

Land Use Category	Area (acres)	Population
Residential - High Density	293	4,219
Residential - Urban Neighborhood	3,148	31,480
Residential - Rural Neighborhood	2,276	5,690
Mixed Use	276	596
Commercial	671	0
Industrial	862	0
Employment	313	0
Public/Institutional	140	0
Parks/Open Space (Existing)	354	0
Totals	**8,333**	**41,985**

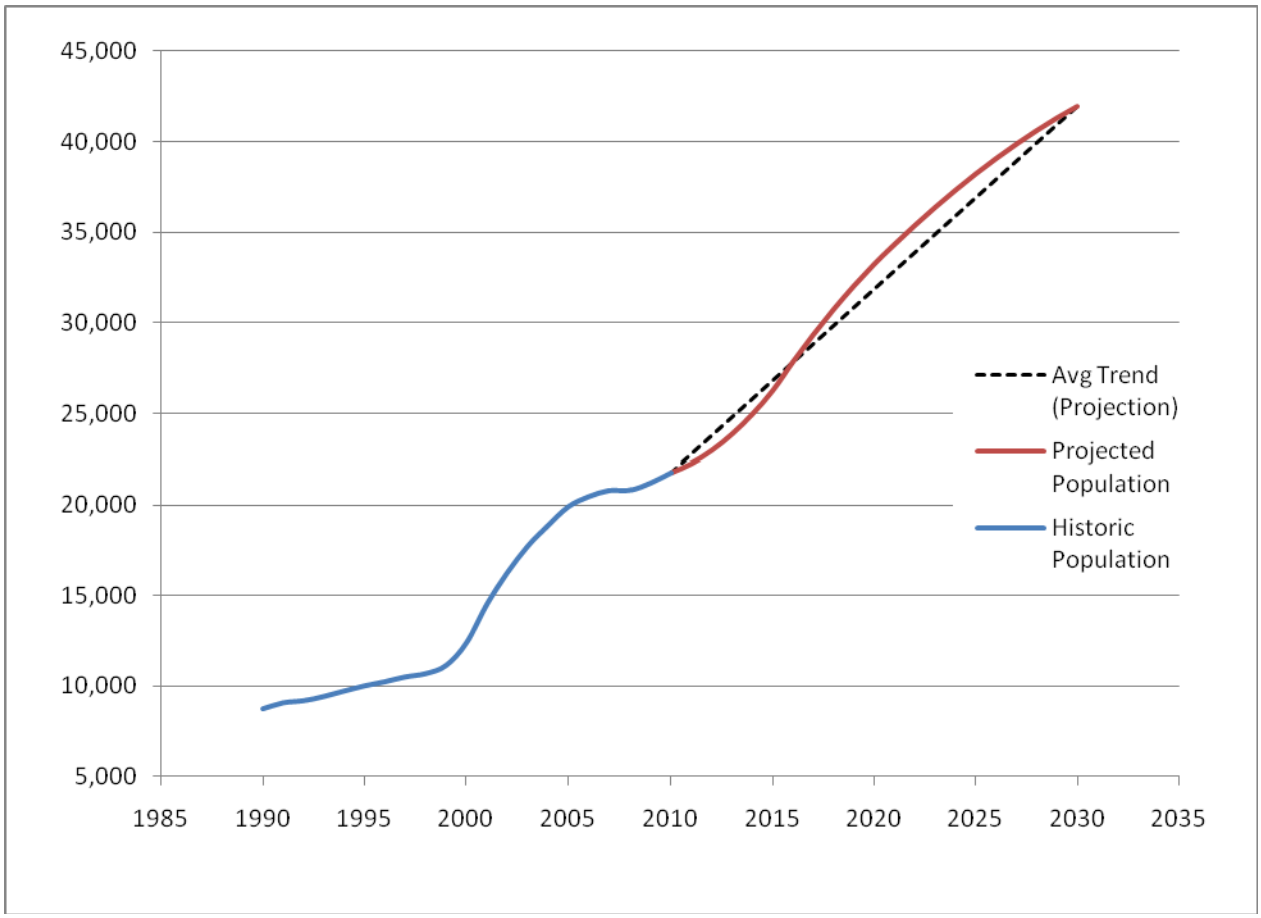
The City of Evans experienced rapid growth over the past 10 years. It is estimated the population of the City of Evans increased at an average annual rate of nearly 13 percent since the time of the 2000 census. Between 1990 and 2000, the population increase was at a Census Bureau estimated rate of 6 percent. Since 2004, the population increase has slowed to an average annual rate of approximately 5 percent. The current population estimate within the city limits is approximately 18,888. It is expected the current economic slow down being experienced across the nation has also slowed growth rates within the City of Evans. Average annual growth rates within the city for 2008 and through 2009 have slowed to less than an estimated 0.5 percent. Table 1-3 summarizes the current and predicted study area population within the planning boundary.

Table 1-3. Evans Predicted Study Area Population within the Planning Boundary

Year	Population	Added Population	Annual Percentage Growth
2010	21,746	150	1.0%
2020	33,246	11,500	5.0%
2030	41,985	8,739	2.4%

Chart 1-1 below provides the total planning boundary population forecasts.

Chart 1-1. Study Area Population Projections within the Planning Boundary



1.3.2. Flow and Loading

The City of Evans sewer collection system is comprised of two separate service basins, one for the Evans Wastewater Treatment Facility (WWTF) and one for the Hill-n-Park WWTF. Table 1-4 summarizes the service basins area, existing (2010) population and average daily flow. The existing area served by the Evans WWTF is approximately 1,132 acres and by the Hill-n-Park WWTF is approximately 488 acres. Based on a total existing (2010) population of approximately 21,746 inside the planning boundary, the estimated population being served by the Evans WWTF is approximately 12,380, while the estimated population served by the Hill-n-Park WWTF is approximately 9,366. The population split was established by delineating the service basins based on the service to parcels by each collection system and intersecting them with the landuse-based population projections.

Table 1-4. WWTF Service Basins

Service Basin	Total Served Area (Acres)	Existing Population	Average Daily Flow, MGD
Evans	1,132	12,380	1,187,000
Hill-n-Park	488	9,366	484,000
Total	**1,620**	**21,746**	**1,671,000**

To appropriately plan for necessary improvements to the wastewater collection and treatment systems and to adequately meet the demands of existing and future population, historical flow and loading data was first examined and unit loadings and peaking factors established. These factors were then used in conjunction with estimated population to arrive at projected flow and loading to each wastewater treatment facility. Total annual average dry-weather flow (ADWF) or base flow projections in the collection system for future land use conditions are comprised of residential, commercial, industrial, and mixed used contributions. In addition to existing (2010) and future (2030) conditions, flow projections were developed for the intermediate year of 2020 for planning purposes. The flow projections utilize the unit flow and I&I factors established in Chapter 2. Mixed use projections are based on half residential and half commercial area.

1.3.2.1. Evans WWTF

Based on the uncapped scenario, the per capita unit loadings are combined with population and peaking factors to generate projections of future loading. Table 1-5 summarizes existing and projected flow and loading.

Table 1-5. Evans WWTF Existing and Projected Wastewater Loading in Planning Boundary

Criteria	Year 2010	Year 2020	Year 2030
Sewered Population	12,380	12,941	13,502
Average Dry-Weather Flow, MGD	1.230	1.277	1.432
Peak Hour Dry-Weather Flow, MGD[1]	1.968	2.043	2.291
Peak Hour Wet-Weather Flow, MGD[2]	3.284	3.516	4.087
Influent BOD_5, lb/d[3]	2,874	2,984	3,346
Influent TSS, lb/d[3]	2,638	2,739	3,071
Influent Total Ammonia, lb/d[3]	328	341	382

[1]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[2]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).
[3]Calculated by average dry-weather flow multiplied by average influent concentration.

1.3.2.2. Hill-n-Park WWTF

Based on unit flow factor recommendations presented above, flow projections for the Hill-n-Park WWTF have been developed. At buildout, the existing developments with current septic system service are assumed to be served by a collection system and influent would be conveyed to the Hill-n-Park WWTF. The unit flow factors were combined with existing and future population and landuse coverage and peaking factors to generate projections of future flow. Table 1-6 summarizes existing and projected flow and loading.

Table 1-6. Hill-n-Park WWTF Existing and Projected Wastewater Flow and Loading in Planning Boundary (Maintain Current Flow Split Between WWTFs)

Criteria	2010	Year 2020	Year 2030
Population	9,366	20,305	28,483
Average Dry-Weather Flow, MGD	0.491	1.949	2.902
Peak Hour Dry-Weather Flow, MGD[1]	0.687	2.729	4.063
Peak Hour Wet-Weather Flow, MGD[2]	1.660	7.968	13.982
Influent BOD_5, lb/d[3]	1,134	4,554	6,780
Influent TSS, lb/d[3]	1,012	4,180	6,223
Influent Total Ammonia, lb/d[3]	122	520	775

[1]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[2]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).
[3]Calculated by average dry-weather flow multiplied by average influent concentration.

1.4. Regulatory Drivers

Prior to selecting and sizing wastewater treatment facilities, a thorough understanding of current and future regulatory drivers is required. Chief among the potential permitting requirements are the following:

- 2013 PEL #200089, which provides the updated effluent discharge standards for the Evans Wastewater Treatment Facility (WWTF) based on a rerating of 1.9 MGD.
- 2009 draft PEL #200090, which set updated effluent discharge standards for the City of Evans Hill-n-Park WWTF.
- Reassessment of the South Platte River as a potential water supply.
- Colorado Department of Health and Environment (CDPHE) Regulations 85 and 31.
- EPA Revised Federal Ambient Water Quality Criteria for Ammonia.

These permit issues govern effluent quality and have implications for the wastewater management choices available to the City.

The City of Evans currently owns and operates two wastewater treatment facilities; the Evans WWTF and the Hill-n-Park WWTF. Both facilities discharge to the main stem of the South Platte River, Middle South Platte River Sub-Basin, with the Hill-n-Park WWTF discharging just downstream of the confluence with the Big Thompson River and the Evans WWTF approximately 4 miles further downstream. Additionally, the La Salle WWTF discharges approximately 2.5 miles downstream of the Hill-n-Park WWTF.

The South Platte River along this stretch is designated for primary contact recreation, agriculture, and water supply. At this time, no water is drawn off for use as potable water downstream of the discharges. However, a number of cities and towns between the City of Evans and the border with Nebraska pull drinking water from wells located adjacent to the river. Conversations with Colorado Department of Health and Environment (CDPHE) indicate they expect a direct potable water draw from the South Platte River within the next 10 years. This would result in significant changes to all discharge permits upstream and would include at a minimum, a requirement for nitrate removal. The site specific in-stream water quality standards for this stretch of river are provided in Table 1-7.

Table 1-7. Water Quality Standards for Stream Segment COSPMS01b

Parameter	Value	Units
Physical and Biological		
Dissolved Oxygen (DO)	7	mg/L
pH	6.5-9	s.u.
E. Coli	126	Colonies/100mL
Temperature	30	degC
Inorganic		
Ammonia	TVS	
Un-Ionized Ammonia Acute and Chronic	TVS	
Chlorine Acute	0.019	mg/L
Chlorine Chronic	0.011	mg/L
Free Cyanide Acute	0.005	mg/L
Sulfide Chronic	0.002	mg/L
Boron Chronic	0.75	mg/L
Nitrite	0.5	mg/L
Nitrate	10	mg/L
Chloride Chronic	250	mg/L
Sulfate Chronic	Water Standard	mg/L
Metals		
Total Recoverable Arsenic – acute	50	µg /L
Dissolved Cadmium – acute/chronic	9.2/1.2	µg /L
Total Recoverable Trivalent Chromium – acute	50	µg /L
Dissolved Hexavalent Chromium – acute/chronic	16/11	µg /L
Dissolved Copper – acute/chronic	49.6/29.3	µg /L
Dissolved Iron – chronic	Water Standard	µg /L
Total Recoverable Iron – chronic	1,000	µg /L
Dissolved Lead – acute/chronic	281/10.9	µg /L
Dissolved Manganese – acute/chronic	4,738/2,618	µg /L
Total Mercury – chronic	0.01	µg /L
Dissolved Nickel – acute/chronic	1,513/168	µg /L
Dissolved Selenium – acute/chronic	18.4/4.6	µg /L
Dissolved Silver – acute/chronic	22/3.5	µg /L
Dissolved Zinc – acute/chronic	467/405	µg /L

The CDPHE Regulation 31 and Revised Federal Ammonia Criteria will have both short and long term impacts on the discharge requirements for the two facilities.

For the purposes of this Wastewater Utility Plan Update, the following effluent nutrient limits will be assumed to be in the Evans WWTF and Hill-n-Park WWTF discharge permits.

- Near term – next 5 years
  - Ammonia < 1 mg/L
  - BOD < 30 mg/L
  - TSS < 30 mg/L
  - Regulation 85 nutrient monitoring
- Long term – outside 10 years
  - Ammonia <1 mg/L
  - BOD < 25 mg/L
  - TSS < 25 mg/L
  - TN < 5 mg/L
  - TP < 0.5 mg/L

The goals of the phase 1 improvements are as follows:

- Increase flow and load capacity
- Remove ammonia below 1 mg/L
- Plan for expansion to meet future total nitrogen and phosphorus effluent limits

1.5. Wastewater Treatment Facilities Evaluation and Alternatives Development

The City of Evans is served by a gravity sanitary sewer system and two wastewater treatment facilities- Evans WWTF and the Hill-n-Park WWTF. Demands on these wastewater facilities have expanded especially on the Hill-n-Park WWTF due to development. Also, it is expected CDPHE will require higher levels of wastewater treatment to protect receiving water quality in the future. Additionally, flow and loading to the Evans WWTF is greater than 95 percent of permitted capacity.

1.5.1. Evans WWTF

The Evans WWTF consists of a three lagoon system rated for a 30-day average flow of 1.2 MGD and an organic loading of 2,772 pounds of biochemical oxygen demand (BOD) per day. The facility was originally constructed in the early 1960's. Recent improvements to the plant include outfall improvements in 1997, a raw water irrigation system in 2000, and a new grit removal unit in 2003. The basic processes in the Evans WWTF lagoon system are as follows;

- Influent 9 IN Parshall Flume with flow transmitter.
- Headworks Building with one 0.25 IN opening cylindrical fine screen and screenings screw washer/compactor/conveyor and dumpster.
- One vortex grit chamber with grit pump, concentrator and conveyor.
- Influent pumping with three 1,540 gpm, 40 hp, centrifugal pumps.
- One 12.30 million gallon partially mixed Aeration Lagoon No. 1, equipped with (8) 15 HP Aqua-Aerobics aerators.

- One 8.25 million gallon partially mixed Aeration Lagoon No. 2, equipped with (5) 15 HP Aqua-Aerobics aerators.
- One 1.63 million gallon Polishing Cell.
- One 29,000 gallon Chlorine Contact Channel with liquid sodium hypochlorite disinfection and sodium bisulfate dechlorination.
- Effluent 6 IN Parshall Flume with flow transmitter.
- 24 IN outfall pipe to the South Platte River.

The capacity of the existing treatment facility was evaluated against current and predicted flow and loading conditions. Table 1-8 summarizes the capacity of each process unit.

Table 1-8: Evans WWTF Capacity Rating Evaluation Summary

Unit Process	Capacity Criteria	Rated Capacity	Year 2020 Ave/Peak Anticipated Flows or Loading	Year 2030 Ave/Peak Anticipated Flows or Loading	Comment
Influent Pipeline	Hydraulic	4.14 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future.
Influent Flow Measurement	Hydraulic	3.69 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity through 2020.
Influent Screen	Hydraulic	4.8 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future. Need to add second unit for redundancy
Grit Removal	Hydraulic	4.0 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future, however, a second unit is required for redundancy.
Influent Pumping	Hydraulic	4.4 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future. Firm capacity=4.4 MGD.
Treatment Lagoons	Solids Loading (BOD/TSS)	2,772 lb/day	2,984 lb/day	3,346 lb/day	Improvements required to meet future flow and permit requirements.
Chlorine Disinfection Equipment and Channel	Hydraulic	1.4 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	System should be expanded or replaced to meet future flow requirements based on proposed treatment alternatives.
Outfall	Hydraulic	2 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for current conditions. Parallel outfall should be constructed to meet future flow requirements.

Evaluation of alternatives to improve and expand the existing treatment facility to accommodate current and predicted flow and loading was performed. Changing water quality regulations may dictate improvements to treated wastewater effluent quality. These regulations were taken into consideration when evaluating treatment alternatives. Alternatives were identified and evaluated through an interactive process involving City and consultant staff. Driving forces considered in the evaluation process included permit revisions, age and condition of existing facilities, service area and population growth,

process improvement, and impact to neighboring properties. Table 1-9 summarizes the processes evaluated, the alternatives considered, and the alternatives selected for detailed analysis.

Table 1-9. Evans WWTF – Alternative Development Ideas and Initial Screening Results

Idea		Initial Screening Result
Biological Process Improvements – Short Term to Meet Future Ammonia Limits:		
Alternative EB1	Construct submerged attached growth reactor after lagoon treatment system	Retain for evaluation.
Alternative EB2	Construct MBBR treatment system	Retain for evaluation.
Alternative EB3	Convert Polishing Pond to an aerated complete mix reactor	Retain for evaluation.
Biological Process Improvements – Long Term to Meet Nitrate and/or Total Nitrogen and Phosphorus Limits:		
Alternative EB4	Add denitrification filter to Alternative EB1	Retain for evaluation.
Alternative EB5	Convert Alternative EB2 MBBR to IFAS through addition of anaerobic and anoxic cells and the construction of secondary clarification	Retain for evaluation.
Alternative EB6	Expand aerated complete mix reactor from Alternative EB3	Retain for evaluation.
Alternative EB7	Demolish existing lagoon treatment system and construct conventional activated sludge system	Fail. Too expensive.
Alternative EB8	Demolish existing lagoon treatment system and construct membrane bioreactor system	Fail. Too expensive.
Alternative EB9	Decommission the Hill-n-Park WWTF, construct pump station and forcemain to convey all flow to the Evans WWTF, demolish existing Evans WWTF lagoon treatment system and construct Conventional Activated Sludge System	Fail. Too expensive.
Alternative EB10	Phosphorus removal through chemical addition and effluent filtration – Addition to all alternatives above	Retain for evaluation.
Disinfection Improvements		
Alternative ED1	Maintain existing liquid sodium hypochlorite disinfection system	Retain for evaluation.
Alternative ED2	Ultraviolet light disinfection	Retain for evaluation.
Alternative ED3	Irradiation	Fail. Not proven technology.
Alternative ED4	Ozone	Fail. Too expensive.
Biosolids Management		
Alternative EBM1	No Action - Maintain liquid application to agricultural land	Retain for evaluation. Existing system.
Alternative EBM2	Dewater and create Class A product	Retain for evaluation.

Idea		Initial Screening Result
Alternative EBM3	Landfill	Fail. Too expensive and does not promote use of valuable product.
Support Facilities		
Alternative ESF1	Replacement of standby power with larger unit	Retain for evaluation.

1.5.1.1. Evans WWTF Phase 1 Improvements Recommendations

The existing Evans WWTF is currently above 95 percent of rated capacity and has had difficulty at times removing ammonia during the winter and spring months. Additionally, the facility will not be able to meet future permit requirements for ammonia and nitrate.

Biological Process Improvements – Alternative EB2/EB5- Construct MBBR treatment system

Based on capital cost, long term energy use, and overall constructability, flow scenario 1 (maintain current flow split) with treatment Alternative EB2/EB5 –MBBR/IFAS is recommended for implementation to meet the short and long term requirements. To meet future phosphorus limits, Alternative EB10 will be implemented.

This alternative requires a greater initial investment to meet the short term ammonia limits, however, it provides a much lower overall operations cost than the other alternatives and can be constructed without taking the existing treatment facility offline. Additionally, Alternative EB2/EB5 provides a higher degree of operational control and can be retrofitted in the future to meet lower effluent total nitrogen and phosphorus limitations, if and when required. The alternative consists of the following (Phase 1):

- Existing Polishing Pond will be dewatered, dredged, excavated and lined with a geomembrane liner.
- A portion of Aeration Lagoon No. 2 will filled in and an MBBR tank constructed.
- Polishing Pond will be used as the secondary clarifier.
- Construct blower building and air distribution header to new MBBR.
- Construct air delivery piping in new MBBR.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Replace standby generator with larger unit.
- Construct parallel outfall to allow for passage of the peak hour flow.

The estimated project cost for constructing the Phase 1 improvements to the biological process is approximately $4,040,000.

If the future regulations require the removal of total nitrogen and phosphorus, the MBBR system will be upgraded to and IFAS through the following (Phase 2):

- MBBR will be expanded through the construction of an anaerobic zone and anoxic zone upstream of the MBBR tanks and additional aerobic zones downstream. The MBBR tanks will be converted to IFAS tanks through the addition of suspended growth bacteria through return activated sludge. The anaerobic zone will provide for biological phosphorus removal while the anoxic tank will provide denitrification to convert nitrate to nitrogen gas.
- A new secondary clarifier will be constructed in the Polishing Pond to allow for the recycle of suspended growth bacteria to the IFAS tank.
- New effluent filtration will be constructed to remove additional phosphorus.

The estimated project cost for constructing Phase 2 is approximately $4,224,000 which results in a total biological improvements cost of $8,264,000.

Disinfection Improvements – Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection

The existing sodium hypochlorite disinfection system is not sized to treat flows above 1.2 MGD and will require expansion and/or replacement to treat peak hour flows once the biological improvements have been completed (currently the treatment lagoons act as flow equalization). Although capital costs for installation of UV light disinfection are greater than upgrading the chemical disinfection system, there are considerable advantages for the City of Evans to consider installation of UV light disinfection. UV light is less susceptible to market fluctuations for chemicals, carries less risk for toxic chemical spill and provides greater assurance that future regulatory requirements are met. Since capacity to the existing chlorination system will need to be increased, it is recommended the City construct *Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection System* to alleviate capacity limitations and to meet future, more stringent, permit requirements. The alternative consists of the following:

- Two parallel UV channels will be installed in the existing chlorine contact channel.
- Level control facilities, lamp removal hoists and monorails, and lamp cleaning systems will be installed.
- A portion of the existing chlorine storage room will be used for UV lamp maintenance and cleaning purposes, and for installation of the UV electrical service equipment.
- A building will be constructed over the UV channels to provide for weather protection and equipment maintenance.

The estimated project cost for constructing the improvements to the disinfection system is approximately $643,000.

Overall Evans WWTF Improvements

Based on the recommended alternatives, the overall project cost for the Evans WWTF Phase 1 improvements is approximately $4,683,000. Table 1-10 provides a summary of the annual cost expenditures.

Table 1-10. Evans WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 15-16
Phase 1 Improvements			
Biological Process Improvements[2]	-Capacity above 90% -Ammonia limits	Design $673,000	Construction $3,367,000
UV Disinfection	-Capacity above 90%	Design $107,000	Construction $536,000
	Totals	**$780,000**	**$3,903,000**
	Overall Total	**$4,683,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, construction of parallel outfall, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

1.5.2. Hill-n-Park WWTF

Similar to the Evans WWTF, the Hill-n-Park WWTF also employs a lagoon treatment system and is currently permitted for a 30-day average flow of 0.9 MGD and an organic loading of 1,000 pounds of BOD per day.

The facility was originally constructed as part of the Hill-n-Park residential development in the late 1970's. Recent improvements to the treatment facility include the following:

- General improvements in 2000 including construction of headworks building, chlorine contact channel, installation of two new mixers in Aeration Lagoon No. 1 and three new mixers in Aeration Lagoon No. 2, and construction of Polishing Cell. Lining of Aeration Lagoon No. 2 and Polishing Cell.
- Headworks building improvements in 2003 including new step screen, screenings washer compactor, new influent flow building, and new non-potable wash water system.
- Construction of new outfall in 2004.
- Installation of two additional mixers in Aeration Lagoon No. 1 in 2009.

The basic processes for the Hill-n-Park WWTF lagoon system are as follows;

- Influent 9 IN Parshall flume with flow transmitter.
- Influent Flow Building with one 6 MM opening step screen and screenings screw washer/compactor/conveyor and dumpster, manual bar screen with 1/2 IN opening.
- One 9.07 million gallon partially mixed Aeration Lagoon No. 1 with (6) 15 HP Aqua-Aerobics aerators.
- One 5.06 million gallon partially mixed Aeration Lagoon No. 2 with (3) 15 HP Aqua-Aerobics aerators.
- One 2.03 million gallon Polishing Cell.
- One 40,000 gallon Chlorine Contact Basin.
- Effluent 9 IN Parshall flume with flow transmitter.

- 21 IN outfall to South Platte River.

The capacity of the existing treatment facility was evaluated against current and predicted flow and loading conditions. Table 1-11 summarizes the capacity of each process unit.

Table 1-11: Hill-n-Park WWTF Capacity Rating Evaluation Summary

Unit Process	Capacity Criteria	Rated Capacity	Year 2020 Ave/Peak Anticipated Flows or Loading	Year 2030 Ave/Peak Anticipated Flows or Loading	Comment
Influent Pipeline	Hydraulic	2.35 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD.
Influent Flow Measurement	Hydraulic	5.73 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD.
Influent Screen	Hydraulic	1.75 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. Expansion required to meet future flow requirements.
Treatment Lagoons	Solids Loading (BOD/TSS)	1,335 lb/day	4,554 lb/day	6,780 lb/day	Adequate capacity for rerated 0.99 MGD. Expansion required to meet future flow and permit requirements.
Chlorine Disinfection Equipment and Channel	Hydraulic	1.92 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. System should be expanded or replaced to meet future flow requirements.
Outfall	Hydraulic	3.5 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. Parallel outfall should be constructed to meet future flow requirements.

Similar to the Evans WWTF, an evaluation of alternatives to improve and expand the Hill-n-Park WWTF to accommodate current and predicted flow and loading was performed. Table 1-12 summarizes the processes evaluated, the alternatives considered, and the alternatives selected for detailed analysis.

Table 1-12. Hill-n-Park WWTF – Alternative Development Ideas and Initial Screening Results

Idea		Initial Screening Result
Biological Process Improvements – Short Term to Increase Capacity and Meet Future Ammonia Limits:		
Alternative HB1	Convert Aeration Lagoons No. 1 to complete mix through addition of mixers	Retain for evaluation.
Alternative HB2	Construct MBBR nitrification cell into Aeration Lagoon No. 1	Retain for evaluation.
Biological Process Improvements – Long Term to Increase Capacity and Meet Nitrate and/or Total Nitrogen and Phosphorus Limits:		
Alternative HB3	Compartmentalize Aeration Lagoon No. 1, convert to complete mix through addition of mixers, add clarification and return flow	Retain for evaluation.
Alternative HB4	Construct MBBR nitrification and denitrification cells into Aeration Lagoon No. 1	Retain for evaluation.
Alternative HB5	Convert portion of Aeration Lagoon No. 2 to an aerated and complete mix reactor, add clarification and return flow	Retain for evaluation.
Alternative HB6	Demolish existing lagoon treatment system and construct Medium Rate Activated Sludge (MRAS) system	Retain for evaluation.
Alternative HB7	Demolish existing lagoon treatment system and construct Conventional Activated Sludge System	Retain for evaluation.
Alternative HB8	Decommission the Evans WWTF, construct pump station and forcemain to convey all flow to the Hill-n-Park WWTF, demolish existing Hill-n-Park WWTF lagoon treatment system and construct Conventional Activated Sludge System	Retain for evaluation.
Alternative HB9	Demolish existing lagoon treatment system and construct membrane bioreactor system	Fail. Too expensive.
Alternative HB10	Phosphorus removal through chemical addition and effluent filtration – Addition to all alternatives above	Retain for evaluation.
Disinfection Improvements		
Alternative HD1	Maintain existing liquid sodium hypochlorite disinfection system	Retain for evaluation.
Alternative HD2	Ultraviolet light disinfection	Retain for evaluation.
Alternative HD3	Irradiation	Fail. Not proven technology.
Alternative HD4	Ozone	Fail. Too expensive.
Biosolids Management		
Alternative HBM1	No Action - Maintain liquid application to agricultural land	Retain for evaluation. Existing system.
Alternative HBM2	Dewater and create Class A product	Retain for evaluation.

Idea		Initial Screening Result
Alternative HBM3	Landfill	Fail. Too expensive and does not promote use of valuable product.
Support Facilities		
Alternative HSF1	Installation of standby power	Retain for evaluation.
Alternative HSF2	Construction of administration/operations building	Retain for evaluation.

1.5.2.1. Hill-n-Park WWTF Phase 1 Improvements Recommendations

Due to the lower flow at the Hill-n-Park WWTF and the recent rerating to 0.99 MGD, only a few improvements are required for Phase 1 including replacement of the existing headworks equipment and addition of standby power. The overall project cost for the Hill-n-Park WWTF Phase 1 improvements is approximately $424,000. Table 1-12 provides a summary of the annual cost expenditures.

Table 1-12. Hill-n-Park WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 14-15
Phase 1 Improvements			
Hill-n-Park WWTF Headworks Screening Improvements	-Existing equipment is at end of useful life	Construction $200,000	
Hill-n-Park WWTF Standby Power	-Required to meet regulatory requirements		Construction $224,000
	Totals	**$200,000**	**$224,000**
	Overall Total	**$424,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.

1.5.2.2. Hill-n-Park WWTF Phase 2 Improvement Recommendations

The Phase 1 improvements are expected to address issues at the Hill-n-Park WWTF for the next 10 to 15 years. To meet capacity and regulatory requirements beyond this time frame, the Hill-n-Park WWTF will require expansion and conversion to a more robust treatment system to bring the average dry-weather capacity to 3 MGD. Phase 2 improvements will include the following:

- Improvements to headworks including grit removal.
- New IFAS reactor, secondary clarifier, and blower building.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Construct new effluent filtration to meet effluent total phosphorus limit.
- Construct parallel outfall to allow for passage of the peak hour flow.

- UV disinfection.

The project cost of constructing the Phase 2 biological process improvements is approximately $15,020,000.

Support Facilities

Additionally, it is expected the construction of the new operations/laboratory/maintenance building will be required to meet the additional operational needs. Key support facilities include the following:

- Maintenance facilities
- Operations personnel offices
- Laboratory facilities

To provide the appropriate space for the above facilities, it is estimated a minimum of 6,000 sf is required. Estimated project cost for this alternative is approximately $1,800,000.

Hill-n-Park WWTF Phase 2 Improvements Summary

Based on the recommended alternatives, the overall project cost for the Hill-n-Park WWTF Phase 2 improvements is approximately $17,717,000. Table 1-13 provides a summary of the Phase 2 annual cost expenditures.

Table 1-13. Hill-n-Park WWTF Phase 2 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 18-23	FY 23-28
Phase 2 Improvements			
Biological Process Improvements[2]	-Capacity above 80% (i.e. increase of flows above 0.8 MGD) -Reg 31 TN and TP effluent limits	Design $2,758,000 Construction $12,516,000	
UV Disinfection	-Revised Federal Ammonia Criteria- ammonia limit lowered below 1 mg/L -No longer use chloramine disinfection	Design- $107,000 Construction- $536,000	
Operations Building	-Required for process control		Design- $360,000 Construction- $1,440,000
	Totals	**$15,917,000**	**$1,800,000**
	Overall Total	**$17,717,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

1.5.3. Opportunities for Consolidation of Treatment

1.5.3.1. City of Greeley

Greeley and Evans have discussed opportunities for shared facilities or consolidation of treatment. The Greeley Water Pollution Control Facility (WPCF) is located north of the Evans WWTF and discharges to the Poudre River. For both cities, flow to the treatment facilities is primarily by gravity. For Evans to send flow to the Greeley WPCF would require construction and operation of a large lift station. The same holds true for Greeley to send flow to Evans. As such, at this time, consolidation of treatment facilities does not provide benefit. However, the cities will continue to work together and make use of opportunities to share facilities similar to the Saint Michaels Subdivision diversion.

1.5.3.2. Town of Milliken

Evans currently has an agreement in place with Milliken to provide wastewater service to a development area referred as Homestead at Ashton located approximately west of 65th Avenue and south of 29th Street. The agreement states Evans will serve the area until 50 units are constructed at which time a pipeline from the development to Milliken will be constructed. Discussions included revisiting the IGA once the development actually begins building and selling homes as it may make more sense to continue sending the wastewater flow to the Hill-n-Park WWTF.

Additionally, the two parties discussed other opportunities for Evans to assist Milliken in wastewater service to areas north of town. The existing Milliken WWTF was upgraded in 2004 and has a rated capacity of 0.7 MGD. Current flow to the facility is approximately 0.4 MGD. Milliken indicated a desire to look at opportunities to gravity flow wastewater from their northern service area to Evans.

Finally, the parties discussed opportunities for coordinating a facility located to the south of the South Platte River. It was agreed that if a facility is needed in this location, there is an opportunity for Evans, Milliken, La Salle, Platteville, and Gilchrest to coordinate and develop a regional wastewater treatment plant together. This opportunity appears to be more than 20 years off; however, the parties have agreed to continue discussions.

1.5.3.3. Town of La Salle

The Town of La Salle is located to the south of Evans across the South Platte River. La Salle currently owns and operates a lagoon wastewater treatment system with a discharge between the Hill-n-Park WWTF and the Evans WWTF. The La Salle WWTF has a rated capacity of approximately 0.46 MGD and is currently operating at half capacity. The La Salle WWTF is facing many of the same permitting issues as Evans and will need to add nitrogen/phosphorus removal at some point in the future. Due to the proximity of La Salle to Evans, there exists an opportunity for the two communities to work together in the future when looking at collection and treatment options on the south side of the South Platte River. This opportunity appears to be more than 20 years off. Currently, Evans and La Salle are working on a revision to a joint planning IGA.

1.6. Conveyance Facilities Evaluation and Alternatives

The existing sewer collection system consists entirely of gravity sewer pipes that convey flow to the two WWTFs. There are several diversion manholes which direct flow through various collector systems. The existing Evans collection and conveyance system consists of gravity sewers ranging in size from 6-inch to 30-inch. Generally, sewers located east of 29th Ave flow to the Evans WWTF with the remaining portion of the western section of the City flowing to the Hill-n-Park WWTF. Currently, the system does not have any lift stations or force mains. Gravity piping generally follows the roads within the City limits. Table 1-14 summarizes the major features found in the sanitary sewer collection system contained in the planning boundary by sewer basin.

Table 1-14. City of Evans Collection System

Gravity Sewer Size	Length (Lineal feet of pipe, number of manholes)
Evans Sewer Basin	
6-inch	*1,000*
8-inch	*180,790*
10-inch	*8,380*
12-inch	*17,370*
15-inch	*11,500*
24-inch	*1,690*
Manholes (Standard)	*889*
Manholes (Diversion)	*3*
Hill-n-Park Sewer Basin	
6-inch	*6,260*
8-inch	*112,500*
10-inch	*14,130*
12-inch	*1,950*
15-inch	*19,500*
24-inch	*13,240*
30-inch	*6,130*
Manholes (Standard)	*761*
Manholes (Diversion)	*0*

The gravity mains vary in material throughout the system and generally based on size and age. Most of the pipe materials are known, however a portion of the system, mostly local, is of unknown material. The known pipe materials in the City of Evans' sewer collection system include A2000, clay, ductile iron (DIP), PVC, and various materials that have been repaired by sliplining. The majority of the system, 48.9 miles is PVC pipe.

Evans' sewer system contains three active diversion or flow split manholes in the analyzed system which are used to direct flows from the incoming sewers to two

outgoing sewers. These manholes are used to optimize sewer system capacity by directing flow into either a parallel sewer or into an adjacent sewer system that has additional capacity.

A hydraulic analysis using the calibrated collection system model was used to locate problem areas during dry-weather and wet-weather scenarios both under existing and buildout conditions. The collection system was evaluated based on the following criteria:

1. Capacity: Local Collector / Collector System (8” - 18”)
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of one-half of the full pipe (d/D = 50 percent)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100 percent)
2. Capacity: Interceptor (> 18”) System
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of 70 percent of the full pipe (d/D = 70 percent)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100 percent)
3. Velocities: Entire Analyzed System
    a. Peak-hour dry-weather – Minimum velocity equal to 2 fps.
    b. Peak-hour wet-weather – Maximum velocity equal to 10 fps.

The base buildout scenario in the InfoSewer model was used to properly size improvements based on only the proposed p and the Northern portion of the LTSA. The modeling did not include the South and East LTSAs as these are outside the 20-year planning horizon. The North LTSA includes an IGA with Greeley and it is anticipated this transfer of service area from City of Greeley to City of Evans will occur in the near future. The WWTF basin boundaries were not modified from the original locations. Flow was not reduced to the Evans WWTF so no Central Lift Station is required. The Southern Lift Station and forcemain are still required because of the buildout area in the South planning boundary that can not be delivered to the Hill-n-Park WWTF by gravity. The North LTSA flow is conveyed by gravity to the Hill-n-Park WWTF via the Ashcroft Draw interceptor.

The potential problem pipes established from the model results were examined to identify likely hydraulic issues under the various flow scenarios. There are a total of 60 pipes in the existing system with hydraulic deficiencies identified including 18 problem pipes under existing flow conditions. The hydraulic problems were separated into two categories; Type A and Type B. Type A problems consisted of a series of problem pipes and that were hydraulically connected to one another. Type B problems are isolated hydraulic restrictions that are not hydraulically connected to other problem locations or series of problem pipes. Type A problems accounted for 93 percent of the problem pipes or a total of 55 pipes with a cumulative length of approximately 3.3 miles of pipe. Type B problems accounted for the remaining 7 percent of problem pipes or a total of 5 pipes with a cumulative length of approximately 0.2 miles of pipe. Both Type A and Type B problems have system improvement recommendations developed and estimates of capital

cost prepared. It is envisioned that Type A problems are identified as recommendations for capital improvements while the Type B problems are addressed through coordination with development activities, system rehabilitation and replacement (R&R) work, or replacement with other department CIP projects.

Problem areas are grouped in three tiers to establish implementation priority: Tier 1 problem areas are Type A improvements and have the highest priority; Tier 2 projects are also Type A improvements but have lower priority compared to Tier 1; and Tier 3 projects are Type B improvements which have the lowest priority. The priorities were assigned based on a number of qualitative factors including the extent of the problem and the flow conditions in which they occur, potential for SSOs, ease of constructability, and data confidence. It is anticipated that Tier 1 projects would be scheduled for construction within the next 10 years. Tier 2 projects would be completed in the following 10-years within the 20-year planning horizon or before as opportunities arise associated with other utility or roadway projects in the project vicinity. It is anticipated that Tier 3 projects could be addressed as minor capital projects or with system rehabilitation work in the project vicinity. Future infrastructure expansion due to growth is expected to be constructed as development occurs.

1.6.1.1. Conveyance Phase 1 Improvements

40th and Pueblo Street

Problem area located in Evans WWTF basin along Pueblo Street from 37th Street to 40th Street, and 40th Street from Pueblo Street to Central Street, and Central Street from 40th Street to 42nd Street. Due to small pipe slopes limited by shallow pipe depths there are capacity problems in existing wet-weather and buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow is approximately 1,700 gpm. Improvements include replacement of 4,397 ft of 15” collector with 21” along Pueblo Street, 40th Street, and Central Street. Estimated project cost for this alternative is approximately $1,502,000.

43rd Street

Problem area located in Evans WWTF basin along 42nd Street from Central Street to US Hwy 85, and 43rd Street from US Hwy 85 to Industrial Parkway, and 42nd Street from Industrial Parkway to the Evans Town Ditch. Due to shallow pipe slopes limited by shallow pipe depths there are capacity problems in existing and buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 1,200 to 1,600 gpm. Improvements include replacement of 1,427 ft of 12” collector with 21”, 2,496 ft of 15” collector with 21” and re-grade 144 ft of 15” collector along 42nd Street and 43rd Street. Estimated project cost for this alternative is approximately $2,080,000.

1.7. Recommendations and Capital Improvement Plan

The City of Evans has limited resources to invest in wastewater infrastructure, making prioritization of capital improvement projects a necessity. Multiple criteria govern the

prioritization of capital improvement projects. The following list highlights the criteria that dictate capital improvements priorities for wastewater treatment and conveyance:

- Surface Water Protection
- Protection of Public Health
- Collection and Treatment System Reliability
- Growth
- Coordination and Compatibility With Other Capital Programs, i.e. Street Overlays
- Renewal and Replacement
- Regulatory Compliance
- Permit Changes

Table 1-15 summarizes the recommended capital improvements by year.

Table 1-15. Capital Improvements Summary

Project Description	Estimated Cost[1]							
	Trigger	FY 13-14	FY 14-15	FY 15-16	FY 16-17	FY 18-23	FY 23-28	FY 28-32
Hill-n-Park WWTF Headworks Screening Improvements	-Existing equipment is at end of useful life	Construction $200,000						
Evans WWTF Biological Process Improvements – Phase 1	-Capacity above 90% -Ammonia limits	Design $673,000	Construction $3,367,000					
Evans WWTF UV Disinfection	-Capacity above 90%	Design $107,000	Construction $536,000					
Hill-n-Park WWTF Standby Power	-Required to meet regulatory requirements		Construction $224,000					
Hill-n-Park WWTF Biological Process Improvements	-Capacity above 80% (i.e. increase of flows above 0.8 MGD) -Reg 31 TN and TP effluent limits					Design $2,758,000 Construction $12,516,000		
Hill-n-Park WWTF UV Disinfection	-Revised Federal Ammonia Criteria-ammonia limit lowered below 1 mg/L -No longer use chloramine disinfection					Design $107,000 Construction $536,000		
40th and Pueblo Street	-Flow ranges from 300 gpm in P1219 to 1,500 gpm in P1262						Design $300,000 Construction $1,202,000	
43rd Street	-Flow ranges from 430 gpm in P1202 to 1,450 gpm in P1216						Design $416,000 Construction $1,664,000	
Evans WWTF Biological Process Improvements – Phase 2	-Reg 31 TN and TP effluent limits						Design $704,000	

Project Description	Estimated Cost[1]							
	Trigger	FY 13-14	FY 14-15	FY 15-16	FY 16-17	FY 18-23	FY 23-28	FY 28-32
							Construction $3,520,000	
Operations Building	-Required for process control						Design $360,000 Construction $1,440,000	
37th Street Area	-Flow ranges from 300 gpm (in 12" collector) and 1,400 gpm in (15" collector)							Design $123,000 Construction $494,000
49th Street	-Flow ranges from 400 gpm in P740 to 11,300 gpm in P208 when growth occurs in West Hill-n-Park Planning Boundary area							Design $737,000 Construction $2,947,000
	Yearly Totals	**$980,000**	**$4,127,000**	**-**	**-**	**$15,917,000**	**$9,606,000**	**$4,301,000**
Total Project Costs		**$34,931,000**						

1.8. Implementation Schedule

The intended implementation schedule for the planning, design, and construction for the Evans WWTF Improvements is provided Table 1-16 below.

Table 1-16. Evans WWTF Phase 1 Improvements Implementation Schedule

Item	Start Date	Completion Date	Notes
Preliminary Engineering Report (PER)	June 20, 2014	July 15, 2014	
Submit Site Application and PER	July 15, 2014		
Site Application Review	July 15, 2014	September 15, 2014	6-8 week review time
Preliminary Design Report (PDR)	September 15, 2014	October 14, 2014	Suggest that Site Application be approved prior to PDR Submittal
Submit for SRF Funding	October 14, 2014	November 18, 2014	30 day review
Project Funding Approved	November 18, 2014		
Final Design	January 9, 2015	April 27, 2015	
Construction	June 4, 2015	June 3, 2016	

1.9. Financial Evaluation

The existing City of Evans wastewater system is funded through user charges which pay for on-going operations and maintenance expenses and modest facility improvements. Both sanitary sewer and municipal water utilities are funded in this manner. The following sections discuss the current wastewater budgets and user charges.

1.9.1. Existing Budgets

The annual operations and maintenance budget for the City of Evans sewer utility totals approximately $1,129,325, based on the 2012 budget. Of the total budget, personnel expenses for operations staff salaries and benefits amount to approximately $293,711. Operations, maintenance, and supplies total approximately $310,765. An annual debt service payment of $111,708 principle and interest is scheduled on a debt balance of $115,454 as of 2012. A transfer to the City's general fund in the amount of $413,141 is made for administrative services provided by general City services. The City has also instituted an Asset Management Plan beginning with the 2010 budget which is intended to provide funds for a portion of the City's renewal and replacements necessary to maintain reasonable operating condition of its infrastructure. For the sewer utility, this amount is budgeted at $10,763 for 2010. Net revenues for the 2012 fiscal year amount to approximately $1,127,280. The net result of income to expenditures results in a net operating loss of $2,044.

Cash reserves for capital projects are currently $297,181. In addition, $90,984 in sewer system development fee revenue is projected for connection to the sanitary sewer system in 2012. These fees are designed to pay for the investment the City has in excess

capacity in the existing system and to fund expansion of the system to serve growth. These fees, combined with capital project cash reserves, total $388,165.

1.9.2. Existing User Charges

User charges to customers provide the needed revenue to operate the City of Evans wastewater utility. User charges are comprised of monthly rates and sewer development fees (connection fees). The current customer base consists of approximately 8,280 sewer connections.

Residential customers are charged a sewer rate based on the following:

- Service Tap Fee per month = $11.15

For commercial sewer service, the rates are based on the following:

- Service Tap Fee per month = $4.31
- Volume Charge = $1.08 per 1,000 gallons of potable water metered
- Total Sewer Rate=Base Service Charge + Volume Charge

The sewer rate results in an average charge of approximately $220.85 per month per commercial connection.

1.9.2.1. Sewer System Development Fees

The City Council has adopted by Resolution 45-2009, sewer system development fees. The resolution calls for a sewer system development fees that vary based upon the size of the water tap size. Table 1-17 presents the existing sewer system development fees for various tap sizes.

Table 1-17. Existing Sewer System Development Fees

Water Tap Size	Meter Equivalent	Inside City Sewer System Development Fee	Outside City Sewer System Development Fee
¾”	1	$3,636.50	$5,454.75
1”	1.67	6,072.96	9,109.43
1 ½”	3.33	12,109.55	18,164.32
2”	5.33	19,382.55	29,073.82
3”	10.67	38,801.46	58,202.18
4”	16.67	60,620.46	90,930.68
6”	33.33	121,204.55	181,806.82
8”	53.33	193,934.55	290,901.82
10”	126.67	460,635.46	690,953.18
12”	166.67	606,095.46	909,143.18

1.9.3. Financial Evaluation

A brief financial analysis has been undertaken in order to estimate the user charges necessary to support the capital improvement program (CIP) outlined in Chapter 6. Capital improvements of any significant magnitude will require additional funding as described below. System development fees are set at a level to provide the City with growth based funding for expansion of the wastewater collection and treatment system.

1.9.3.1. Historical Financial Analysis – Fiscal Years 2005 through 2012

The City of Evans provided historical financial documents detailing revenues and expenses for the five year period from 2005 through 2012. The results of the City's historical financial analysis are provided in Table 1-18. From this analysis, it appears as though rate revenues were not sufficient to cover current operations and maintenance expenses, debt service and administrative transfers in the period from 2005 through 2008. However, in 2009, the City showed a balance of funds after O&M, debt service and transfers. This is due to the adjustment of rates accomplished in Resolution 45-2009.

Table 1-18. Historical Financial Analysis

Account Name	FY 2005	FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011	FY 2012
REVENUE								
Rate Revenues	$844,478	$876,369	$898,149	$918,467	$1,094,250	$1,022,094	$1,027,800	$1,125,063
Non-Operating Revenues	$65,852	$113,993	$162,961	$85,723	$17,442	$11,029	$2,994	$2,218
Total Source of Funds	$910,330	$990,362	$1,061,110	$1,004,190	$1,111,692	$1,033,123	$1,030,794	$1,127,281
EXPENSES								
Personnel	$0	$0	$0	$0	$0	$289,261	$284,713	$293,711
Operations	$1,113,429	$1,001,070	$964,399	$557,334	$672,257	$300,002	$310,895	$310,765
Total Operations & Maintenance	$1,113,429	$1,001,070	$964,399	$557,334	$672,257	$589,263	$595,608	$604,476
TOTAL DEBT SERVICE	$108,486	$184,643	$184,602	$184,167	$109,687	$115,005	$113,361	$111,708
CAPITAL IMPROVEMENTS FUNDED THROUGH RATES	$0	$0	$0	$0	$0	$0	$0	$0
TRANSFER TO OTHER FUNDS	$0	$245,206	$264,288	$334,002	$289,528	$363,517	$412,824	$413,141
TOTAL CHANGE IN RESERVE FUNDS	$0	$0	$0	$0	$0	$0	$0	$0
TOTAL REVENUE REQUIREMENTS	$1,221,915	$1,430,919	$1,413,289	$1,075,503	$1,071,472	$1,067,785	$1,121,793	$1,129,325
Balance/(Deficiency) of Funds	$(311,585)	($440,557)	($352,179)	($71,313)	$40,220	($34,662)	($90,999)	($2,044)

1.9.3.2. Projected Future Financial Analysis– Fiscal Years 2013 through 2030

The impact of financing the recommended capital improvements summarized in Table 6-10 was evaluated against current user rates and fees. The largest impact to rates is directly related to the projected cost and timing of the Evans and Hill-n-Park WWTF improvement projects. In the evaluation, it was assumed that growth would continue at a rate of 5 percent per year as predicted in Chapter 2, and the cost of operating and maintaining the system would also increase at an average rate of 3 percent per year.

Table 1-19. Summary Revenue Requirements Analysis (2013 Through 2030)

CITY OF EVANS
SEWER- EXHIBIT 1
REVENUE REQUIREMENT SUMMARY

SEWER REVENUE REQUIREMENT SUMMARY
PROJECTED 2013 - 2030

Account Name	FY 2013	Projected FY 2014	FY 2015	FY 2016	FY 2017	FY 2018	FY 2019	FY 2020	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030
REVENUE																		
Rate Revenues	$1,216,525	$1,222,608	$1,228,721	$1,234,864	$1,241,039	$1,250,346	$1,259,724	$1,272,321	$1,285,044	$1,297,895	$1,310,874	$1,323,983	$1,337,222	$1,350,595	$1,364,101	$1,377,742	$1,391,519	$1,405,434
Non-Operating Revenues	4,955	6,465	8,287	9,910	12,170	12,305	12,524	13,481	14,805	16,066	17,094	16,300	16,625	17,384	18,016	18,712	19,315	19,699
Total Sources of Funds	**$1,221,480**	**$1,229,073**	**$1,237,008**	**$1,244,774**	**$1,253,208**	**$1,262,652**	**$1,272,248**	**$1,285,802**	**$1,299,849**	**$1,313,961**	**$1,327,968**	**$1,340,282**	**$1,353,847**	**$1,367,978**	**$1,382,117**	**$1,396,454**	**$1,410,834**	**$1,425,134**
EXPENSES																		
Personnel	328,280	339,770	351,662	363,970	376,709	389,894	403,540	417,664	432,282	447,412	463,071	479,279	496,054	513,415	531,385	549,984	569,233	589,156
Operations	336,256	346,344	356,734	367,436	378,459	389,813	401,507	413,552	425,959	438,738	451,900	465,457	479,421	493,803	508,617	523,876	539,592	555,780
Total Operations & Maintenance	**$664,536**	**$686,113**	**$708,396**	**$731,406**	**$755,168**	**$779,707**	**$805,047**	**$831,216**	**$858,241**	**$886,150**	**$914,971**	**$944,736**	**$975,474**	**$1,007,219**	**$1,040,002**	**$1,073,859**	**$1,108,825**	**$1,144,936**
TOTAL DEBT SERVICE	**$115,454**	**$164,915**	**$430,709**	**$434,423**	**$430,279**	**$434,196**	**$338,860**	**$323,661**	**$605,105**	**$886,550**	**$1,231,319**	**$1,301,680**	**$1,372,041**	**$1,463,510**	**$1,509,245**	**$1,649,967**	**$1,678,112**	**$1,706,256**
RATE FUNDED CAPITAL	**$0**	**$50,000**	**$100,000**	**$150,000**	**$200,000**	**$250,000**	**$300,000**	**$350,000**	**$400,000**	**$450,000**	**$500,000**	**$550,000**	**$600,000**	**$650,000**	**$700,000**	**$750,000**	**$800,000**	**$850,000**
TRANSFER TO OTHER FUNDS	**$430,156**	**$440,910**	**$451,933**	**$463,231**	**$474,812**	**$486,682**	**$498,849**	**$511,320**	**$524,103**	**$537,206**	**$550,636**	**$564,402**	**$578,512**	**$592,975**	**$607,799**	**$622,994**	**$638,569**	**$654,533**
TOTAL CHANGE IN RESERVE FUNDS	$11,335	$105,981	($13,892)	$129,443	$281,335	$430,237	$539,567	$579,088	$326,245	$172,851	($27,765)	($1,751)	$35,602	$63,157	$147,895	$69,490	$108,557	$152,605
TOTAL REVENUE REQUIREMENTS	**$1,221,480**	**$1,447,919**	**$1,677,146**	**$1,908,503**	**$2,141,594**	**$2,380,821**	**$2,482,323**	**$2,595,286**	**$2,713,695**	**$2,932,757**	**$3,169,161**	**$3,359,067**	**$3,561,629**	**$3,776,861**	**$4,004,942**	**$4,166,311**	**$4,334,062**	**$4,508,331**
Balance/(Deficiency) of Funds	$0	($218,847)	($440,138)	($663,729)	($888,386)	($1,118,170)	($1,210,076)	($1,309,484)	($1,413,845)	($1,618,795)	($1,841,193)	($2,018,785)	($2,207,782)	($2,408,883)	($2,622,825)	($2,769,857)	($2,923,228)	($3,083,197)
Balance/(Deficiency) as a % of Rate Rev.	**0.0%**	**17.9%**	**35.8%**	**53.7%**	**71.6%**	**89.4%**	**96.1%**	**102.9%**	**110.0%**	**124.7%**	**140.5%**	**152.5%**	**165.1%**	**178.4%**	**192.3%**	**201.0%**	**210.1%**	**219.4%**
Proposed Rate Adjustment	**0.0%**	**17.9%**	**15.2%**	**13.2%**	**11.6%**	**10.4%**	**3.5%**	**3.5%**	**3.5%**	**7.0%**	**7.0%**	**5.0%**	**5.0%**	**5.0%**	**5.0%**	**3.0%**	**3.0%**	**3.0%**
Cumulative Proposed Rate Adjustment	**0.0%**	**17.9%**	**35.8%**	**53.7%**	**71.6%**	**89.4%**	**96.1%**	**102.9%**	**110.0%**	**124.7%**	**140.5%**	**152.5%**	**165.1%**	**178.4%**	**192.3%**	**201.0%**	**210.1%**	**219.4%**
Debt Service Coverage Ratio																		
Before Rate Adjustment	1.10	0.62	0.18	0.12	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
After Proposed Rate Adjustment	1.10	1.95	1.20	1.64	2.12	2.57	3.48	3.87	2.20	1.70	1.38	1.42	1.46	1.49	1.56	1.50	1.54	1.59
Residential Monthly Average Bill																		
After Proposed Rate Adjustment	**$11.15**	**$13.15**	**$15.14**	**$17.14**	**$19.13**	**$21.12**	**$21.86**	**$22.63**	**$23.42**	**$25.06**	**$26.81**	**$28.15**	**$29.56**	**$31.04**	**$32.59**	**$33.57**	**$34.57**	**$35.61**
Monthly $ Change	**$0.00**	**$2.00**	**$2.00**	**$2.00**	**$1.99**	**$1.99**	**$0.74**	**$0.77**	**$0.79**	**$1.64**	**$1.75**	**$1.34**	**$1.41**	**$1.48**	**$1.55**	**$0.98**	**$1.01**	**$1.04**
Commercial Monthly Average Bill																		
After Proposed Rate Adjustment	**$220.85**	**$260.38**	**$299.96**	**$339.55**	**$378.94**	**$418.35**	**$433.00**	**$448.15**	**$463.84**	**$496.30**	**$531.05**	**$557.60**	**$585.48**	**$614.75**	**$645.49**	**$664.85**	**$684.80**	**$705.34**
Monthly $ Change	**$0.00**	**$39.53**	**$39.58**	**$39.59**	**$39.39**	**$39.41**	**$14.64**	**$15.15**	**$15.69**	**$32.47**	**$34.74**	**$26.55**	**$27.88**	**$29.27**	**$30.74**	**$19.36**	**$19.95**	**$20.54**
Ending Fund Balance																		
Operating Reserve	$308,516	$414,497	$400,605	$505,049	$511,383	$516,620	$531,187	$610,276	$661,520	$734,372	$706,606	$704,856	$740,457	$778,615	$801,510	$846,001	$859,557	$882,163
Capital Reserve	724,274	621,774	425,024	633,299	1,144,902	1,856,780	2,718,934	3,606,460	3,013,695	2,296,309	1,235,305	1,030,354	875,794	882,628	989,861	963,829	866,204	853,989

As can be seen in Table 1-19, the timing of the capital improvement plan has a significant impact on the rates of the utility. This is due primarily to the lack of any outside funding sources other than revenue bonds to fund these projects. With near-term revenue adjustments and bond funding to accomplish the capital improvement schedule set forth in Chapter 6, the City's primary impact is in the first three years of this plan. After this initial period, the need for rate adjustments based upon the planning assumptions used in this analysis is reduced to slightly more than an inflationary adjustment. This is due to the building and use of reserve funding in conjunction with the collection of system development fees based upon calculated growth.

Another important item to note in Table 1-19 is the line item "Capital Improvements From Rates". A general rule of thumb for funding utility renewal and replacement projects is to fund a minimum level of annual depreciation expense. The 2009 depreciation expense is $472,000. As shown in the summary table above, the utility gradually increases the funding of capital from rates increasing this level above current annual depreciation expense over the 6-year review period shown above. While the capital improvements funded through rates amount begins to exceed current depreciation, the current depreciation figures should be adjusted as the plant improvements are brought on line.

This type of capital funding also improves debt service coverage requirements that are required by bonding agencies if revenue bond funding is used to pay for the project costs. Most bond covenants require a minimum of 1.25 coverage. This means the utility has the capacity to pay the debt service obligations, plus an additional 25 percent of those obligations, each year. As seen in the Table 7-4, the utility is not currently meeting debt service coverage requirements. However, with the proposed rate adjustments, that situation is quickly remedied.

It is recommended that the City of Evans pursue available grants and low-interest loans to offset the cost of the wastewater treatment plant improvement projects. Following is a detailed discussion of outside sources of funding available for projects of this type.

1.9.4. Recommended Financial Plan

Chapter 6 presented a summary of estimated capital costs for a variety of alternative wastewater collection and treatment plant management programs and summarized the costs for the recommended plan. These recommended program elements and estimated capital costs were presented in Table 6-10. Implementation of the recommended plan will require a coordinated effort on the part of the City of Evans to see that the various elements of the plan are implemented within the community and the larger study area.

The recommended capital improvement plan suggests these be implemented in multiple phases. As previously stated, it is recommended the City of Evans pursue grant and low-interest loan funding to reduce the impact of the Phase 1 improvements to rate payers. To meet the long term financial needs of the sewer utility, it is recommended the City raise sewer rates $2/month for the next five years to a final rate of $21.12 by 2018. This will provide the required funding to meet the upcoming Phase 1 improvements as well as the long term operations and maintenance requirements. After this point, rates can be raised on an inflationary path until the Hill-n-Park WWTF improvements are required either

from a capacity or regulatory need. At which time, the rates will again need to be raised to accommodate the required improvements.

2.0 Basis of Planning

2.1. Introduction

To plan for future wastewater facility needs, it is necessary to project wastewater flows to Evans WWTF and Hill-n-Park WWTF. Determination of the flow to the treatment facilities is dependent on the following:

- Land use,
- Topography,
- Population density,
- Magnitude and type of commercial and industrial activity in the area to be served,
- Visiting population and employment impact,
- Condition of the existing system, and
- Regulatory requirements.

The purpose of this chapter is to identify current flow and project future conditions.

2.2. Study Area Definition

Wastewater flow is determined by the size and the land use of the area to be served in combination with the infiltration and inflow (I/I). The physical characteristics of the area to be served, such as topography, geology, and geographical location greatly influence the type of land use and in turn the population density as well as commercial and industrial activity within the area. The planning investigation examines the physical characteristics of the Study Area, population densities, and land use that dictate wastewater service requirements in the future.

The City of Evans is located within Weld County and bordered by the communities of Greeley and Garden City to the North, Kersey to the East, La Salle to the South, and Milliken to the southwest. The current City limits are bounded by the City of Greeley to the North, 1st Avenue to the East, 77th to the West, and Weld County Road 46 to the South.

2.2.1. Planning Boundary

Evans' existing planning boundary is defined by the existing 208 Boundary as shown on Figure 2-1. The City of Evans is in the process of updating their Comprehensive Land Use Plan (Comp Plan) which has redefined Evans' Urban Growth Boundary (UGB) and Long-term Growth Areas. These new service area boundaries extend beyond the current 208 Boundary. As a result, this Wastewater Utility Plan seeks to expand the Evans 208 Boundary to the south as shown in Figure 2-1. The proposed 208 Boundary expansion would extend into an Undesignated Service Area. The proposed planning boundary includes the area within the existing 208 Boundary, the 208 Boundary extension and an overlap into Milliken's 208 Boundary. The planning boundary overlay is permitted under an existing Inter-governmental Agreement (IGA) between Evans and Milliken. For reference, the Evans/Milliken IGA is included in Appendix E.

It should be noted that Evans' proposed planning boundary extends into Milliken's currently defined service area. Per the Evans/Milliken IGA, depending on how lands develop and the

associated sewer systems are extended, this area could be served by either entity.

2.2.2. Long-Term Service Area

The proposed Long-Term Service Area (LTSA) (Figure 2-1) generally follows the Long-Term Growth Area as defined by the Evans Comp Plan Update. This area is defined as being outside the 20 year planning window of this study and is not included in the population and flow projections. Additionally, no 208 WUSA boundary change is requested for areas in the LTSA at this time. A summary of the LTSA extensions are as follows:

- The east and south extensions are in an Undesignated Service Area.
- The west extension is into Milliken's 208 Boundary but is permitted under the previously referenced IGA and has been identified as an Overlapping Service Area (see Appendix E).
- The north extension is into the City of Greeley's currently defined service area. The Evans/Greeley IGA notes that Evans may provide service to these areas when the existing T-Bone Lift Station reaches its design/useful life or when additional growth exceeds the lift station capacity (this will require a plan amendment in the future). A copy of this IGA is included in Appendix D.

2.2.3. Stakeholder Coordination

HDR and Evans staff met with staff from Milliken, Greeley, and La Salle to discuss the proposed planning boundary and Long-Term Service Area boundaries and opportunities for shared facilities and/or consolidation of treatment. The following sections describe these meetings.

2.2.3.1. City of Greeley

The City of Greeley (Greeley) bounds Evans along the north edge. The two cities have a long standing relationship to assist each other on water and wastewater issues. Greeley currently provides treatment of Evans' potable water and delivers it through master meters. Concurrently, Evans provides wastewater service to Greeley in the area bounded by the Highway 34 Bypass to the north, 37th Street to the south, 65th Avenue to the east, and 71st Avenue to the west, referred to as the Saint Michaels Subdivision. A portion of this subdivision currently flows by gravity to the south into the Evans Ashcroft Draw trunk sewer. The majority of the remaining North LTSA is currently served by the Greeley T-Bone lift station.

Greeley recently completed a loading evaluation of the area and determined flows could increase at the T-Bone lift station from approximately 0.4 MGD currently to approximately 1 MGD in the next 10 to 15 years. Due to pipeline constrictions downstream of the T-Bone lift station, Greeley and Evans discussed increasing the amount of flow diverted to Evans. This would allow for gravity flow of all wastewater in the area and the removal of the lift station. The diverted flow would be directed to Ashcroft Draw and on to the Hill-n-Park WWTF. With the diversion of flow away from the lift station, the load to the Hill-n-Park WWTF would immediately increase by 0.4 MGD. However, due to the costs for increasing the capacity of the Hill-n-Park WWTF, Greeley decided to continue use of the T-Bone lift station for the foreseeable future. Recently, Greeley and Evans have agreed to allow for the construction of a bypass from the T-Bone lift station in the event of a failure. However, the bypass will only be used during emergency situations and as such, flow from this area is not included in the future planning for the Hill-n-Park WWTF.

Additionally, Greeley indicated a desire to redirect wastewater flows from the Auto Plaza to the Evans WWTF. However, the conveyance of this flow to the Evans WWTF has not yet been fully developed and is assumed to be more than 10 years out. Flows for this area are not included in any part of the projections as they are undefined at this time.

Finally, Greeley and Evans discussed opportunities for shared facilities or consolidation of treatment. The Greeley Water Pollution Control Facility (WPCF) is located north of the Evans WWTF and discharges to the Poudre River. For both cities, flow to the treatment facilities is primarily by gravity. For Evans to send flow to the Greeley WPCF would require construction and operation of a large lift station. The same holds true for Greeley to send flow to Evans. As such, at this time, consolidation of treatment facilities does not provide benefit. However, the cities will continue to work together and make use of opportunities to share facilities similar to the Saint Michaels Subdivision diversion.

2.2.3.2. Town of Milliken

The Town of Milliken (Milliken) is located along the southwest boundary of Evans. A meeting was held with Milliken to discuss service areas boundaries and opportunities for coordination. Evans currently has an agreement in place with Milliken to provide wastewater service to a development area referred as Homestead at Ashton located approximately west of 65th Avenue and south of 29th Street. The agreement states Evans will serve the area until 50 units are constructed at which time a pipeline from the development to Milliken will be constructed. Discussions included revisiting the IGA once the development actually begins building and selling homes as it may make more sense to continue sending the wastewater flow to the Hill-n-Park WWTF.

Additionally, the two parties discussed other opportunities for Evans to assist Milliken in wastewater service to areas north of town. The existing Milliken WWTF was upgraded in 2004 and has a rated capacity of 0.7 MGD. Current flow to the facility is approximately 0.4 MGD. Milliken indicated a desire to look at opportunities to gravity flow wastewater from their northern service area to Evans.

Finally, the parties discussed opportunities for coordinating a facility located to the south of the South Platte River. It was agreed that if a facility is needed in this location, there is an opportunity for Evans, Milliken, La Salle, Platteville, and Gilchrest to coordinate and develop a regional wastewater treatment plant together. This opportunity appears to be more than 20 years off; however, the parties have agreed to continue discussions.

2.2.3.3. Town of La Salle

The Town of La Salle is located to the south of Evans across the South Platte River. La Salle currently owns and operates a lagoon wastewater treatment system with a discharge between the Hill-n-Park WWTF and the Evans WWTF. The La Salle WWTF has a rated capacity of approximately 0.46 MGD and is currently operating at half capacity. The La Salle WWTF is facing many of the same permitting issues as Evans and will need to add nitrogen/phosphorus removal at some point in the future. Due to the proximity of La Salle to Evans, there exists an opportunity for the two communities to work together in the future when looking at collection and treatment options on the south side of the South Platte River. Currently, Evans and La Salle are working on a revision to a joint planning IGA.

2.2.4. Study Area Recommendations

The proposed planning boundary and LTSA are shown on Figure 2-1. Table 2-1 below

summarizes the areas of the proposed service areas and the existing 208 Boundary area.

Table 2-1. Service Area Summary

Planning Boundary	Total Area
Existing 208 Boundary	7,490 acres
Overlapping Service Area (Milliken)	2,180 acres
Undesignated Service Area (Weld Co.)	4,190 acres
TOTAL PLANNING BOUNDARY	*13,860 acres*
LTSA	**Total Area**
Ashcroft Draw Sewer Basin (Greeley)	1,650 acres
Overlapping Service Area (Milliken)	727 acres
Undesignated Service Area (Weld Co.)	7,561 acres
TOTAL LTSA	*9,938 acres*

The proposed planning boundary and LTSA boundaries were reviewed with North Front Range Water Quality Planning Association (NFRWQPA) staff. Typically, NFRWQPA only requires population/flow projections and collection system extension analysis to be performed within the planning boundary. Given the significant additional area encompassed by the LTSA, population/flow projections and collection system analysis have also applied to the LTSA. However, the LTSA is outside of the 20 year planning window and are not included in the final population and flow projection numbers.

2.3. Study Area Description

The Evans Study Area is primarily within Township 5N, Ranges 65W and 66W, with portions of the Study Area also in Township 4N, Ranges 65W and 66W. Currently, the developed portion of the City of Evans lies between the City of Greeley to the North, the Town of LaSalle to the South, Weld County Road 46 to the South, the South Platte River to the East, and 77th Avenue to the West. The majority of the developable land in the Study Area lies to the west of the current limits. This area is comprised mainly of agricultural land with some wetlands and riparian vegetation. Figure 2-2 depicts many of the Study Area's physical characteristics discussed in the following sections.

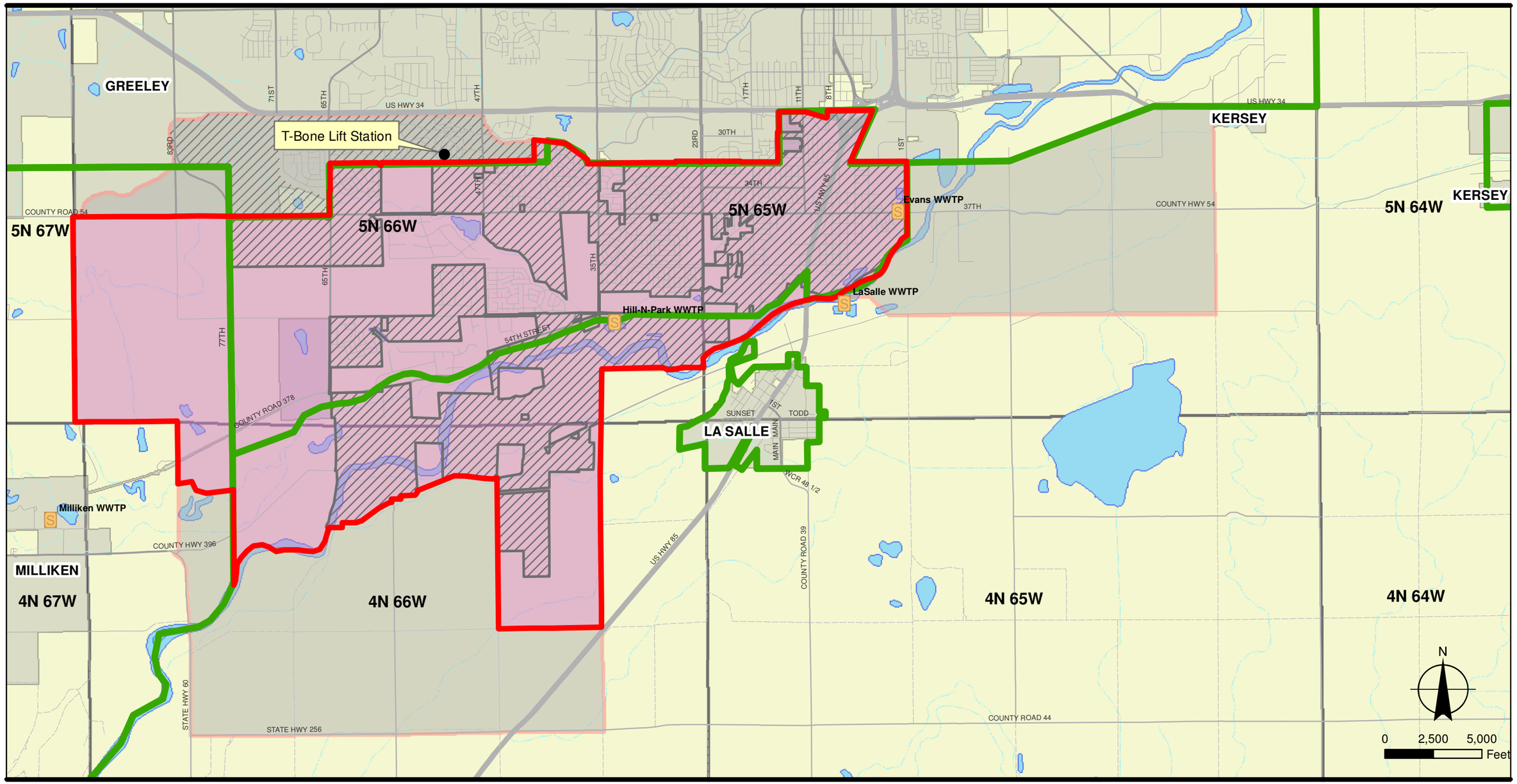
2.3.1. Geology and Soils

The geology of the Study Area is comprised mainly of alluvium sediments consisting of sand, silt, clay, gravel, and alluvial fan materials. These materials may be found in the level to gently rolling terrain that exists across much of the Study Area.

2.3.2. Surface Water and Topography

The Study Area is located in the South Platte River Basin. The South Platte River runs on the South and East side of Evans, flowing Southwest to Northeast. The Cache la Poudre River approaches Evans from the North and combines with the South Platte River downstream of the Evans WWTF discharge. Chapter 3 provides a detailed description of the receiving water.

Topography in the area is generally rolling and varies in elevation from 4,650 FT near the South Platte River to 4,850 FT along the Northwestern edge.



Legend

- Treatment Plants
- Existing 208 Boundaries
- Evans City Limits
- Ashcroft Draw sewer basin per Greeley IGA

Service Area

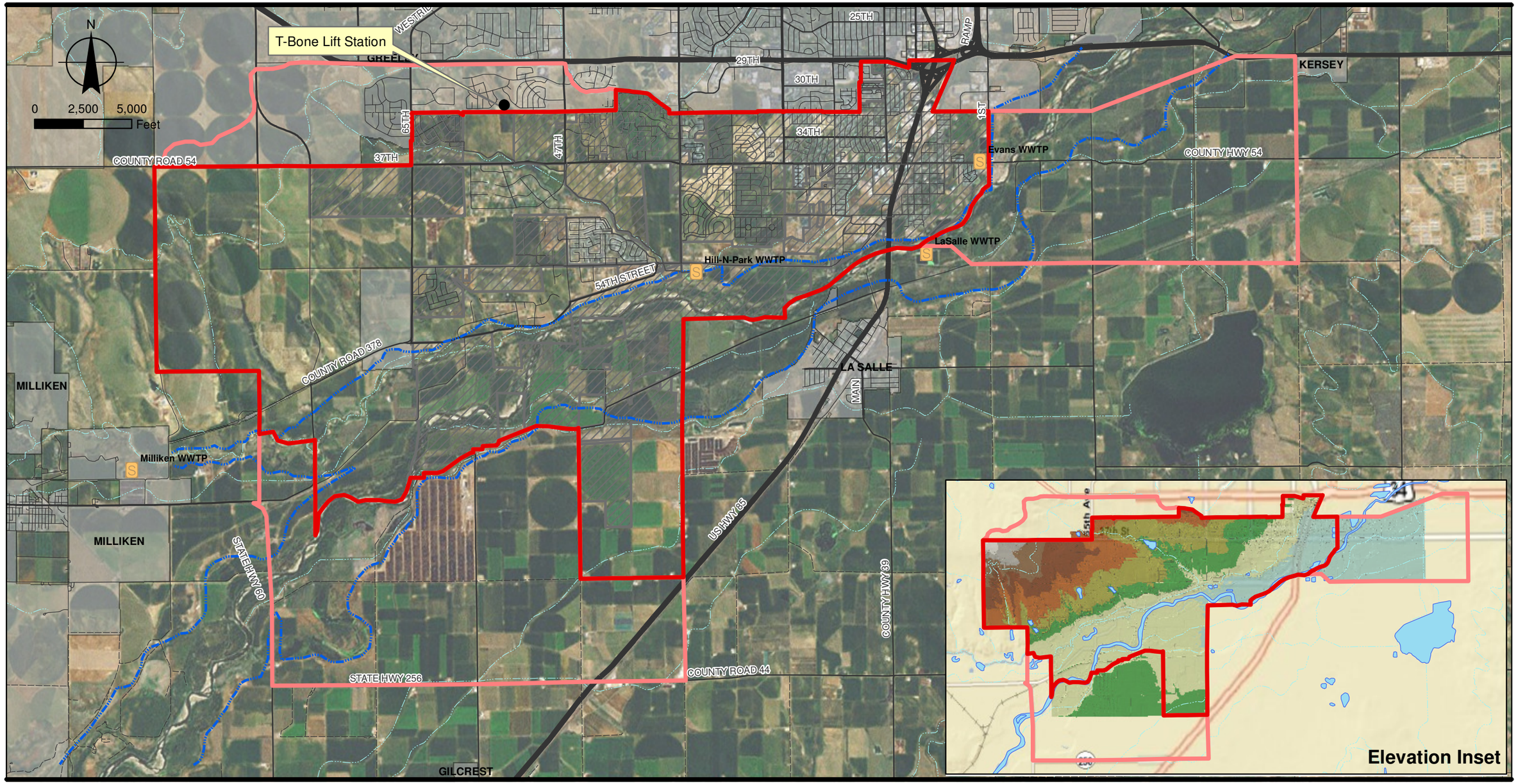
- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Existing 208 Planning Boundaries and Proposed Service Area

HDR
ONE COMPANY | *Many solutions*

Figure 2.1
July 18th, 2011



Legend

- Treatment Plants
- Floodplain (Approx.)
- City Limits (Neighboring Communities)

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Elevation (Inset)

- 5003.009 - 5050.83
- 4955.188 - 5003.009
- 4907.367 - 4955.188
- 4859.546 - 4907.367
- 4811.724 - 4859.546
- 4763.903 - 4811.724
- 4716.082 - 4763.903
- 4668.261 - 4716.082
- 4620.44 - 4668.261

Evans Wastewater Utility Plan

Study Area Conditions

HDR
ONE COMPANY | *Many solutions*

Figure 2.2
July 18th, 2011

2.3.3. Groundwater

Groundwater depths are approximately between five and twenty feet across the Study Area.

2.3.4. Water Supply

The City of Evans owns and operates the water distribution system within the current service area. Additionally, the city owns the water rights necessary to meet the current and long term demand. However, an intergovernmental agreement provides the raw water to the City of Greeley for treatment and return to the Evans distribution system. A series of master meters delivers the treated water to the City of Evans and allows the City of Greeley to accurately bill Evans. Once the water is delivered, it becomes the responsibility of the City of Evans to supply customers, meet return flow obligations, or utilize the excess return flows for other purposes. The City of Evans distribution system does not include pump stations for treated water storage as the City of Greeley provides the service pressure. See Chapter 3 for a detailed description of Evans' water rights.

2.3.5. Floodplain

Federal Emergency Management Agency (FEMA) floodplain maps show within the Study Area the existence of 100-year floodplain along the South Platte River. This floodplain parallels the river channel and is shown on Figure 2-2. The floodplain maps indicate a 100-year flood elevation of approximately 4,642 FT and 4,654 FT near the Evans WWTF and Hill-n-Park WWTF, respectively.

A constructed levee protects the Evans WWTF from flooding as the existing ground elevation is approximately 4,638 FT. However, the lagoons and treatment systems are surrounded by a berm with a top elevation of approximately 4,643.5 FT. The Hill-n-Park WWTF is located outside the 100-year flood plain with a minimum ground surface elevation of approximately 4,666 FT.

2.3.6. Biological Environment

The Study Area is located in the High Plains Eco-Region and vegetation is categorized by agriculture, deciduous woodlands, and riparian zone vegetation. Agricultural lands are used to grow primarily alfalfa, wheat, and hay. They are also used for pasture. Plants associated with pasture land are various clovers, timothy, fescue and bluegrass. Vegetation in riparian zones along the creeks and rivers and in wetlands typically consists of cottonwoods, willows, alder, and dogwood with an understory of numerous forbs and grasses. Deciduous woodlands may be found in upland and riparian areas and often contain vegetation similar to that found in riparian zones.

2.3.7. Wildlife and Important Habitat

The Study Area supports a variety of wildlife species. Increased human development has placed considerable pressure on habitat in the Study Area. Table 2-2 summarizes common wildlife resources and associated habitats in the Study Area.

Table 2-2. Common Wildlife Resources

Wildlife Group	Common Representative Species	Associated Habitats
Large Mammals	White-tailed deer Mule Deer Elk	Deciduous Woodlands Riparian Agricultural Lands
Small Mammals	Deer Mouse Eastern Cottontail Striped Skunk Meadow Vole Raccoon Prairie Vole House Mouse Black Tailed Prairie Dog	Deciduous Woodlands Riparian Agricultural Lands Urban/developed Lands
Furbearers	Coyote Beaver Muskrat Badger Red Fox	Coniferous forest Deciduous Woodlands Riparian Agricultural Lands Urban/developed Lands
Waterfowl	Canada Goose Great Blue Heron Pintail Mallard American White Pelican	Riparian Wetlands Aquatic
Upland Game Birds	Turkeys	Coniferous forest Riparian Agricultural Lands
Raptors	Red-tailed Hawk Great Horned Owl American Kestrel Bald Eagle	Deciduous Woodlands Riparian Agricultural Lands
Songbirds/passerine	Common Grackle American Robin House Finch Barn Swallow Mourning Dove Killdeer Vesper Sparrow House Sparrow Western Meadowlark European Starling Horned Lark Red Winged Black Bird Black-billed Magpie	Deciduous Woodlands Riparian Agricultural Lands Urban/developed Lands Wetlands
Reptiles/Amphibians	Common Garter Snake Bull Snake Western Rattlesnake Painted Turtle Lesser Earless Lizard Bull Frog Plains Spadefoot Western Chorus From Tiger Salamander	Deciduous Woodlands Riparian Agricultural Lands Wetlands Urban/developed Lands

The South Platte River and riparian areas found in the planning area provide potential nesting habitat for a wide variety of waterfowl.

The South Platte River contains a number species of fish including carp, creek chub, fathead minnow, Johnny darter, longnose sucker, longnose dace, green sunfish, and white sucker.

Threatened or endangered species that would be expected to be encountered in the Study Area include the Bald Eagle.

2.3.8. Wetlands

Wetlands are protected by Section 404 of the Clean Water Act and work in wetlands may require coordination with both Federal and state water quality agencies and the issuance of a permit by the U.S. Army Corps of Engineers. Wetlands are important and sensitive environmental areas that serve many beneficial functions including ground water recharge, flood control, filtering of surface water runoff, and providing essential wildlife habitat.

2.4. Land Use

Land use in and around the City of Evans over the past 15 years is steadily changing from predominately agricultural to residential and commercial general land use types. The purpose of this section is to document the existing condition land use and estimate future condition land use to form the basis for population and wastewater flow projections.

2.4.1. Existing Conditions

Current condition land use was mapped for the Study Area using the "Account Type" field within Weld County parcel GIS data (see Figure 2-3). A large majority of the Study Area is currently served by private septic systems or consists of agricultural use. To assist in evaluating the wastewater unit flow characteristics for the Evans sewer system, parcels that are connected to the Evans system were identified and mapped (see Figure 2-4). In addition, there are several subdivisions within the planning boundary that are currently served by septic systems which are also shown on Figure 2-4. It is anticipated that as these septic systems fail, these properties will connect to the Evans sewer system. A summary of the developed lands, within the Study Area, and their sewer service type is provided in Table 2-3. These developed land areas provide a reference point for comparison to future land use conditions. Wastewater flows used for existing treatment facility analysis and allocation to the collection system model were based on recorded flow meter data and sewer tap information.

Table 2-3. Existing Developed Lands and Existing Sewer Service Summary

	Developed Area (acres)		
	Served by Evans	Septic Systems within Planning Boundary[1]	Served by Greeley[2]
Residential	1,290	2,530	460
Commercial	311	2.11	35
Industrial	19.2	4.92	0
TOTALS	**1,620.2**	**2,537**	**495**

[1]Includes Arrowhead Estates, Dos Rios Estates and Indian Hills subdivisions

[2]Area identified in Evans/Greeley IGA

2.4.2. Future Conditions

For the purposes of this Wastewater Utility Plan, the following planning horizons shall be used:

- Current- 2012 (estimated from Census)
- Intermediate- 2020
- Future- 2030
- Buildout- 2035 (used by North Front Range Metropolitan Planning Organization (NFRMPO) for their Traffic Analysis Zones (TAZ)

Future condition land use for this study is defined by the *Draft Land Use Plan* as developed for the on-going Evans Comp Plan Update and is shown on Figure 2-5. The *Draft Land Use Plan* was modified for purposes of projecting future wastewater flows by removing undevelopable areas. The resulting developed area can then be applied to development densities (units per acre) to estimate the number of residential units or commercial/industrial building areas. The undevelopable areas were estimated with GIS using the following data and assumptions:

1. Lake Buffers – Removed the lake surface and an additional 100 FT buffer around all lake boundaries.
2. Streams – a 50 FT buffer was applied to stream and irrigation ditch on each side of the centerline.
3. FEMA Floodplains – the FEMA floodplain boundary for the South Platte River and the Little Thompson were assumed as undevelopable areas.
4. Road Right-of-Way – Existing rights-of-way within the Evans city limits were removed using parcel data. Existing rights-of-way for County roads were estimated using a 60 FT buffer applied to road on each side of the centerline.
5. Railroad Right-of-Way – Existing right-of-way for railroads were estimated using a 100 FT buffer applied to the railroad on each side of the centerline.
6. Slopes and Miscellaneous Drainages – A slope map was developed using the digital elevation model to identify minor drainages. This resulted in five drainages with side slopes unsuitable for development located between Spomer and Rehmer lakes located toward the western side of the service area.

The resulting developable areas for the planning boundary and LTSA are summarized in Table 2-4 and Table 2-5, respectively. Chart 2-1 provides a summary of the future developable area within the planning boundary. It should be noted the Draft Land Use Plan provides additional detail with respect to land use classifications that will allow for a refined estimate of future population and associated wastewater flows.

Table 2-4. Future Developable Area Summary within the Planning Boundary

Land Use Category	Area (acres)
Residential - High Density	293
Residential - Urban Neighborhood	3,148
Residential - Rural Neighborhood	2,276
Mixed Use	276
Commercial	671
Industrial	862
Employment	313
Public/Institutional	140
Parks/Open Space (Existing)	354
Totals	**8,333**

Chart 2-1 Future Developable Area within the Planning Boundary (Acres)

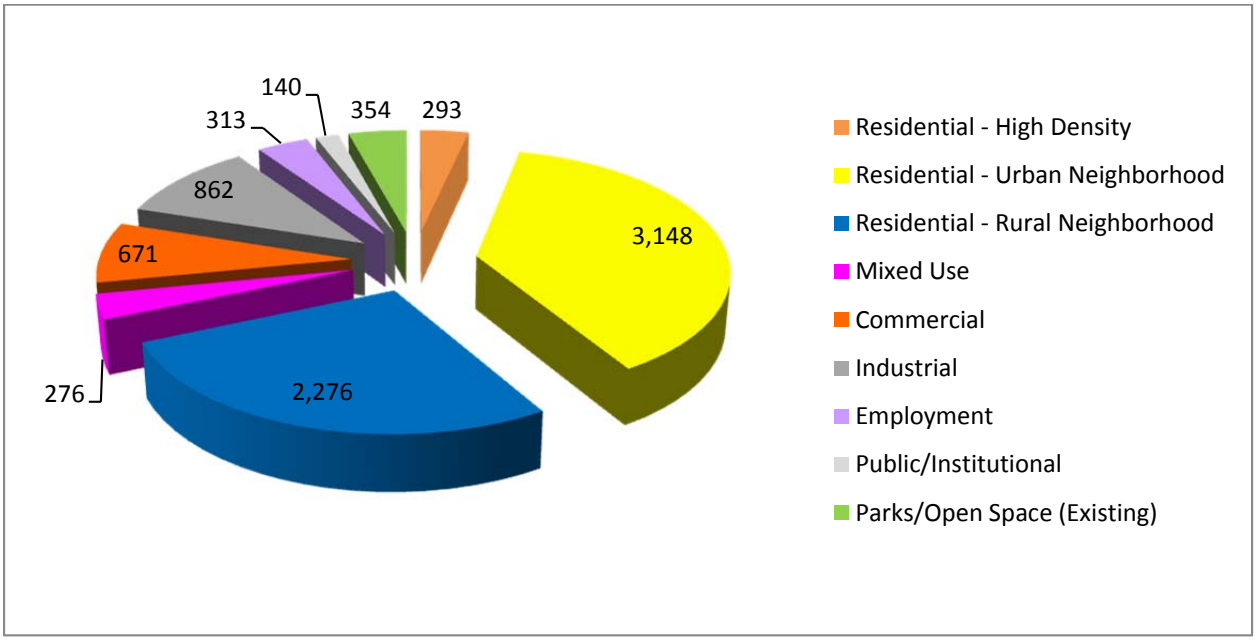
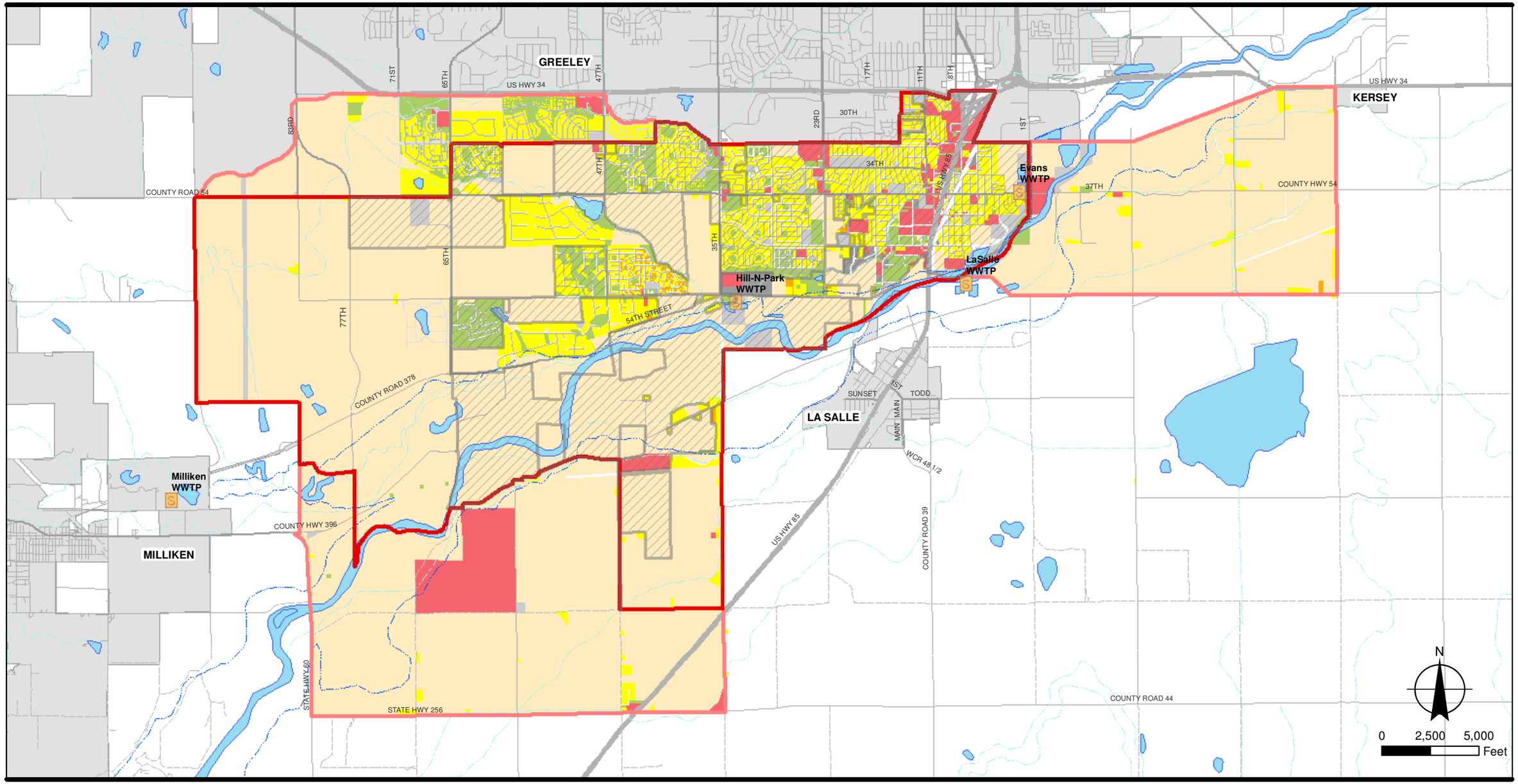


Table 2-5. Future Developable Area Summary within the LTSA

Land Use Category	East LTSA (acres)	South LTSA (acres)	North LTSA (acres)
Residential - High Density	53	0	93
Residential - Urban Neighborhood	1,098	652	383
Residential - Rural Neighborhood	695	1,342	802
Mixed Use	33	0	0
Commercial	174	442	117
Industrial	0	744	0
Employment	0	0	0
Public/Institutional	0	0	0
Parks/Open Space	2	0	8
Totals	**2,055**	**3,180**	**1,403**



Legend

- Treatment Plants
- Streams
- Floodplain (Approx.)
- Evans City Limits
- City Limits (Neighboring Communities)

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Existing Land Use Category

- Agricultural
- Residential
- Mobile Home
- Industrial
- Exempt; Public/Utility
- Commercial
- Natural Resource; Vacant Land

Evans Wastewater Utility Plan

Existing Condition Land Use

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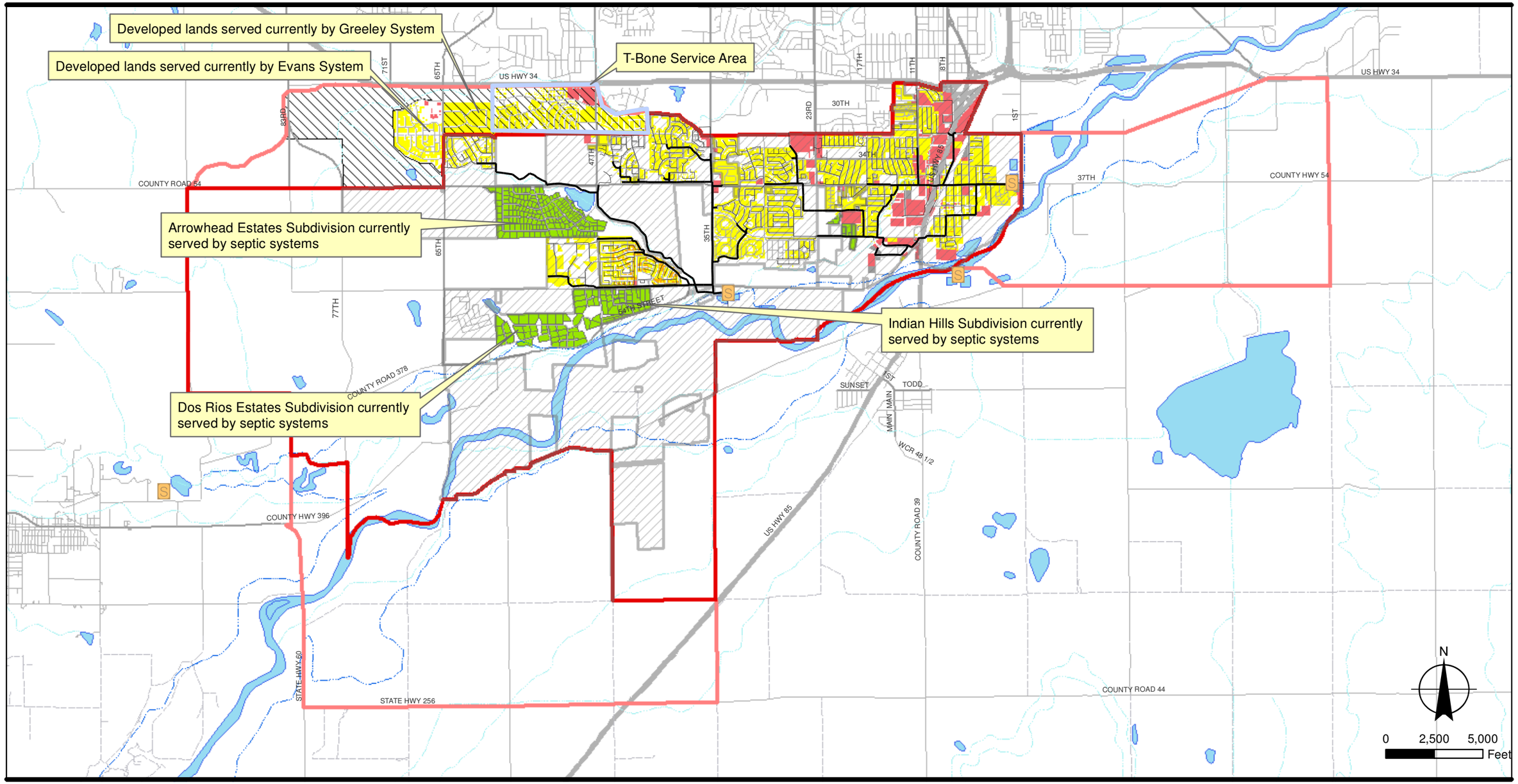
Figure 2.3
July 18th, 2011



Legend

- Treatment Plants
- Exist. Local Sewer
- Exist. Collector/Interceptor Sewer
- Streams
- Floodplain (Approx.)
- Evans City Limits
- Developed Area Served by Septic
- Ashcroft Draw sewer basin per Greeley IGA

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Existing Land Use Served by Evans

- Residential
- Mobile Home
- Industrial
- Commercial

Evans Wastewater Utility Plan

Existing Developed Land Use Served by Evans Sewer System or Septic System

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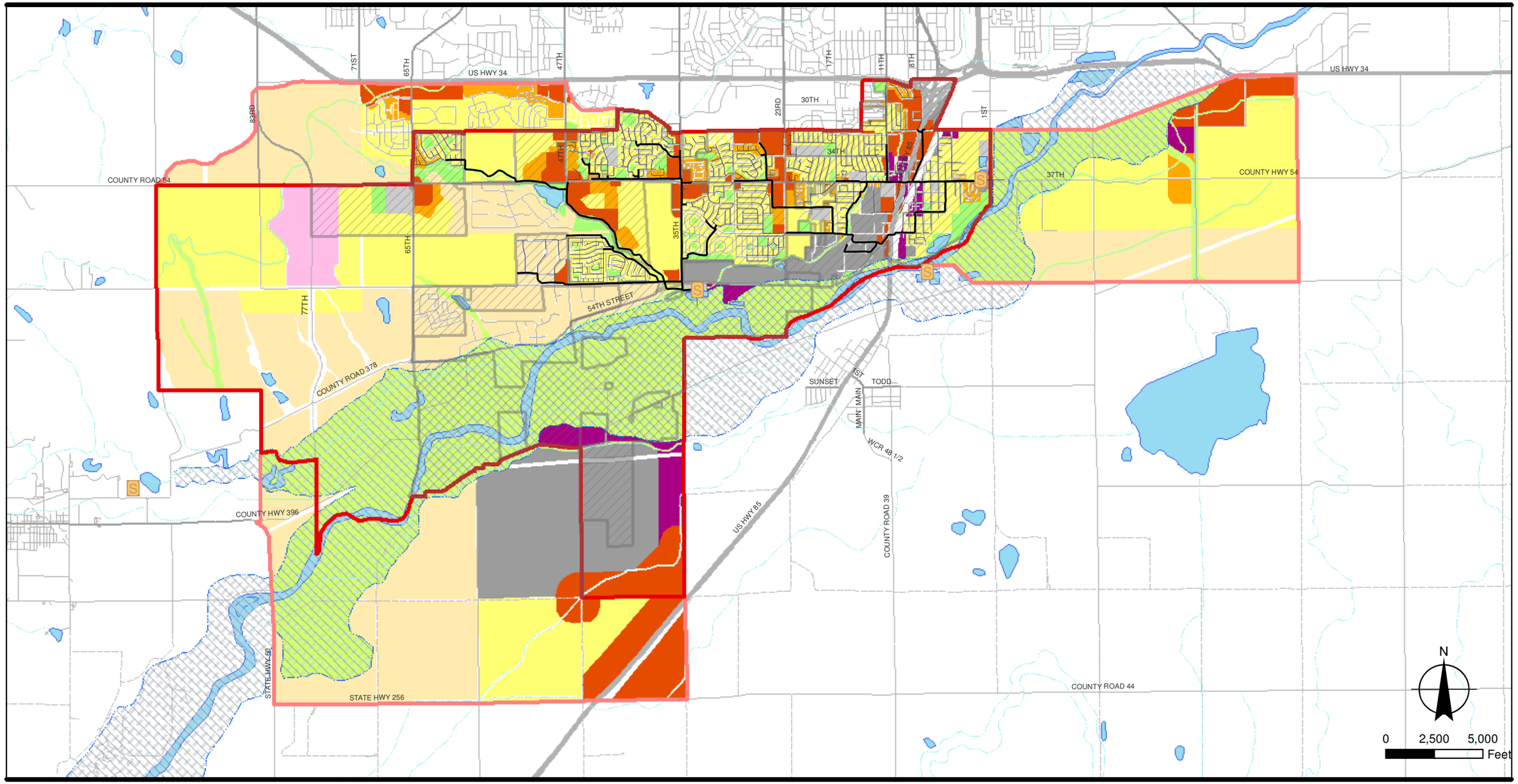
Figure 2.4
July 18th, 2011



Legend

- Treatment Plants
- Exist. Local Sewer
- Exist. Collector/Interceptor Sewer
- Evans City Limits

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Future Condition Landuse

- Residential - Rural Neighborhood
- Residential - Urban Neighborhood
- Residential - High Density
- Industrial
- Public/Institutional
- Mixed Use
- Commercial
- Employment
- Parks/Open Space (Existing)
- Undevelopable Lands

Population Summary by Service Area Region

Description	Developable Acres	Buildout Population
Planning Boundary	8,333	41,985
East UPA	2,055	8,392
South UPA	3,180	5,417
North UPA	1,403	4,535
Total	**14,971**	**60,329**

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Evans Wastewater Utility Plan

Future Condition Land Use

Figure 2.5
July 18th, 2011

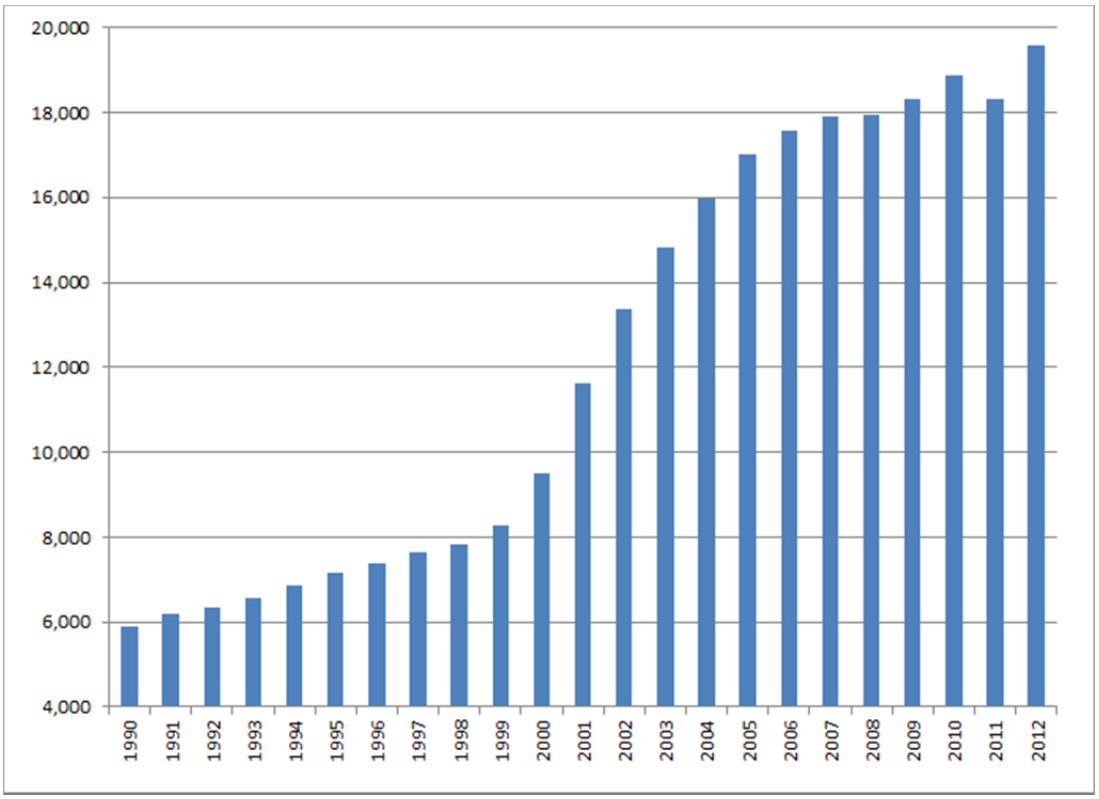
2.5. Population and Growth

Historical growth trends for the Evans area were analyzed and meetings were held with City staff to evaluate growth trends, development densities and resulting population projections. Two data sources were used to evaluate future population projections: the Evans Comprehensive Plan and the North Front Range Metropolitan Planning Organization's Transportation Analysis Zone (TAZ) model results.

2.5.1. Existing Population and Trends

In wastewater facility planning it is important to understand trends in population for the Study Area in order to predict future population and its impact to the sewer collection system and wastewater treatment facilities. The City of Evans experienced rapid growth over the past 10 years. It is estimated the population of the City of Evans increased at an average annual rate of nearly 13 percent since the time of the 2000 census. Between 1990 and 2000, the population increase was at a Census Bureau estimated rate of 6 percent. Since 2004, the population increase has slowed to an average annual rate of approximately 5 percent. The estimated 2012 population estimate within the city limits is approximately 19,576. This information is shown graphically on Chart 2-2. It is expected the current economic slow down being experienced across the nation has also slowed growth rates within the City of Evans. Within the planning boundary but outside the City of Evans limits there are approximately 2,858 people in Greeley and unincorporated Weld County with wastewater service provided by the City.

Chart 2-2. City of Evans Existing Population and Trends



2.5.2. Future Population Forecast

Accurate projection of future population is essential in the analysis of growth impacts on the

City's infrastructure. Population projections are used as the basis to develop plans for future utilities to serve growth and analyze the impact of growth on the existing sewer collection system and wastewater treatment facilities. The basis for the population projection was the Evans Comprehensive Plan land use and supporting development densities. The TAZ data were used to validate and refine the existing and projected population for the Study Area.

2.5.2.1. Comprehensive Plan Population Results

The Draft Land Use Plan and development densities from the Evans Comprehensive Plan were used to develop population projections for the Study Area. Table 2-6 summarizes the development densities and unit population values used in the projections. Table 2-7 and 2-8 summarize the population projections within the planning boundary and the LTSA. Developable area in acres was used in projections to account for parks and other undevelopable areas.

Table 2-6. Comprehensive Plan Development Densities and Unit Population Values

	Density (units/acre)	Unit Population (people/household)
Within the Planning Boundary		
Residential - High Density	8	1.8
Residential - Urban Neighborhood	4	2.9 (existing) / 2.5 (future)
Residential - Rural Neighborhood	1	2.9 (existing) / 2.5 (future)
Mixed Use	1.2	1.8
Outside the Planning Boundary		
Residential - High Density	8	1.8
Residential - Urban Neighborhood	2.5	2.9 (existing) / 2.5 (future)
Residential - Rural Neighborhood	0.4	2.9 (existing) / 2.5 (future)
Mixed Use	1.2	1.8

Table 2-7. Theoretical Future Population Projection Summary within the Planning Boundary

Land Use Category	Area (acres)	Population
Residential - High Density	293	4,219
Residential - Urban Neighborhood	3,148	31,480
Residential - Rural Neighborhood	2,276	5,690
Mixed Use	276	596
Commercial	671	0
Industrial	862	0
Employment	313	0
Public/Institutional	140	0
Parks/Open Space (Existing)	354	0
Totals	**8,333**	**41,985**

Table 2-8. Theoretical Future Population Projection Summary within the LTSA

Land Use Category	East LTSA		South LTSA		North LTSA	
	Area (acres)	Population	Area (acres)	Population	Area (acres)	Population
Residential - High Density	53	763	0	0	93	1,339
Residential - Urban Neighborhood	1,098	6,863	652	4,075	383	2,394
Residential - Rural Neighborhood	695	695	1,342	1,342	802	802
Mixed Use	33	71	0	0	0	0
Commercial	174	0	442	0	117	0
Industrial	0	0	744	0	0	0
Employment	0	0	0	0	0	0
Public/Institutional	0	0	0	0	0	0
Parks/Open Space (Existing)	2	0	0	0	8	0
Totals	**2,055**	**8,392**	**3,180**	**5,417**	**1,403**	**4,535**

2.5.2.2. TAZ Population Results

NFRMPO TAZ model results were obtained in ArcGIS format. The data was intersected with the Planning Boundary and LTSA boundaries to proportion the TAZ model results to each of the individual service areas. These population results are summarized in Table 2-9 and presented on Figure 2-6.

Table 2-9. TAZ Population Projections

Land Use Category	2005 Population	2035 (Buildout) Population
Proposed Planning Boundary	19,978	27,042
East LTSA	329	469
South LTSA	224	204
North LTSA	7,247	15,839
Totals	**27,778**	**43,554**

Primary observations from the TAZ data results are as follows:

- The TAZ model uses 2.9 people per household for current conditions and decreases to 2.5 people per household for 2035 conditions. This is consistent with US Census Bureau (2010) findings of 2.9 people per household and is anticipated to decrease in the future.
- 2005 TAZ population results for the planning boundary agree well with City of Evans information (2010).
- Future TAZ population projections within the planning boundary are less than previously predicted City of Evans populations per the 2002 Comprehensive Plan. The TAZ resulting growth rate of 1.2 percent is also less than recent population trends experienced.
- TAZ population projections in both the East and South LTSA show essentially no growth and in the instance of the South LTSA negative growth.

- Current and future TAZ population projections in the North LTSA are significantly higher than City of Evans estimates.

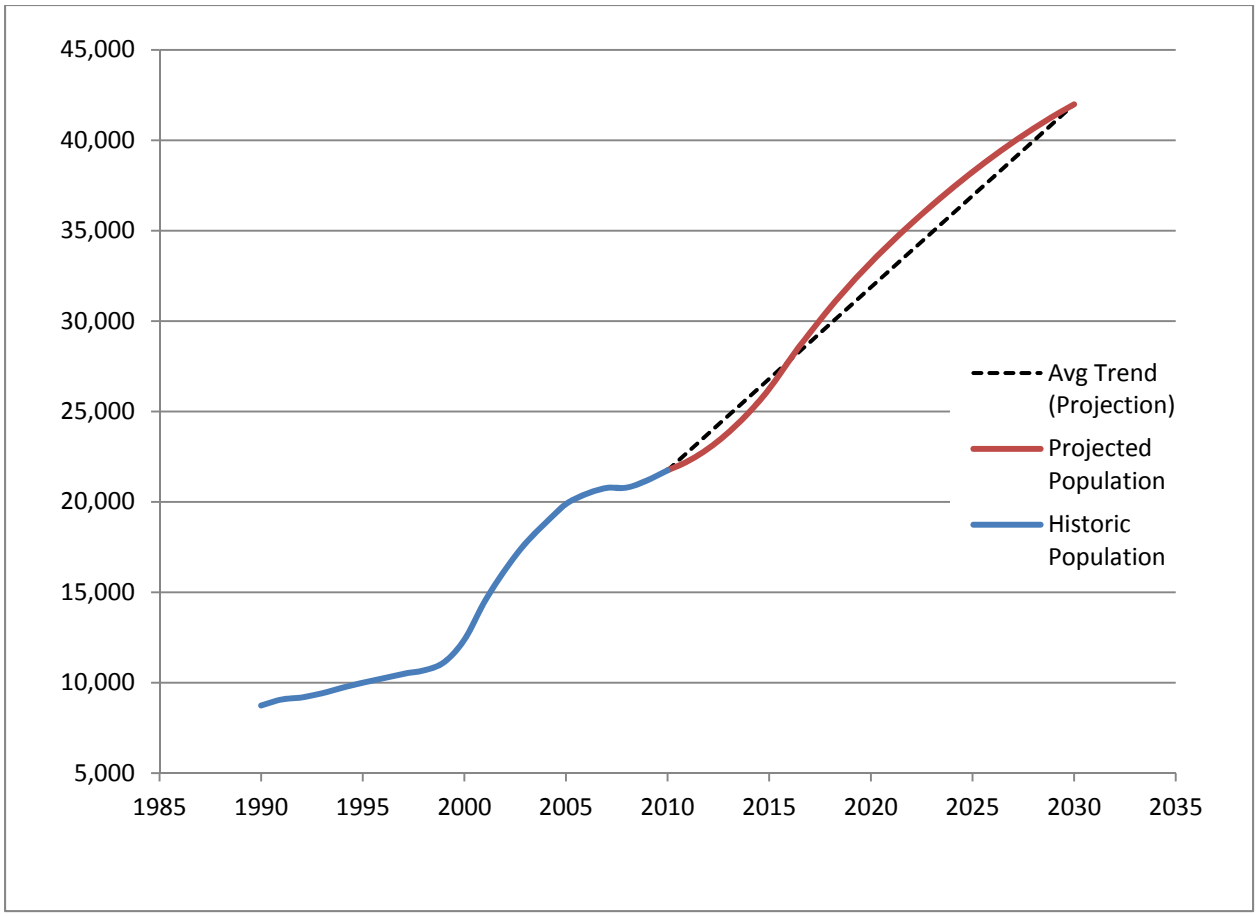
2.5.2.3. Conclusions

The *Draft Land Use Plan* and development densities noted above result in population forecasts consistent with previous City of Evans planning documents. These population forecasts in the planning boundary are presented in Table 2-10 and Chart 2-3. The single family population per household values used in the TAZ data was applied to the *Draft Land Use Plan* to refine the population projections used in this study. In addition, the current condition TAZ data validated the population estimates derived from the existing condition land use plan and city records. The *Draft Land Use Plan* results in an average annual growth rate of approximately 5 percent which compares well with recent development and growth trends. Even though the recent economic slow down makes growth rate and population forecasting uncertain, these land use analyses still provide the best available tool for projecting wastewater flows.

Table 2-10. Population Projections within the Planning Boundary

Year	Population	Added Population	Annual Percentage Growth
2010	21,746	555	2.6%
2011	22,246	500	2.3%
2012	22,946	700	3.1%
2013	23,846	900	3.9%
2014	24,946	1,100	4.6%
2015	26,246	1,300	5.2%
2016	27,846	1,600	6.1%
2017	29,346	1,500	5.4%
2018	30,746	1,400	4.8%
2019	32,046	1,300	4.2%
2020	33,246	1,200	3.7%
2021	34,346	1,100	3.3%
2022	35,396	1,050	3.1%
2023	36,396	1,000	2.8%
2024	37,346	950	2.6%
2025	38,246	900	2.4%
2026	39,096	850	2.2%
2027	39,896	800	2.0%
2028	40,646	750	1.9%
2029	41,346	700	1.7%
2030	41,985	639	1.5%

Chart 2-3. Study Area Population Projections within the Planning Boundary



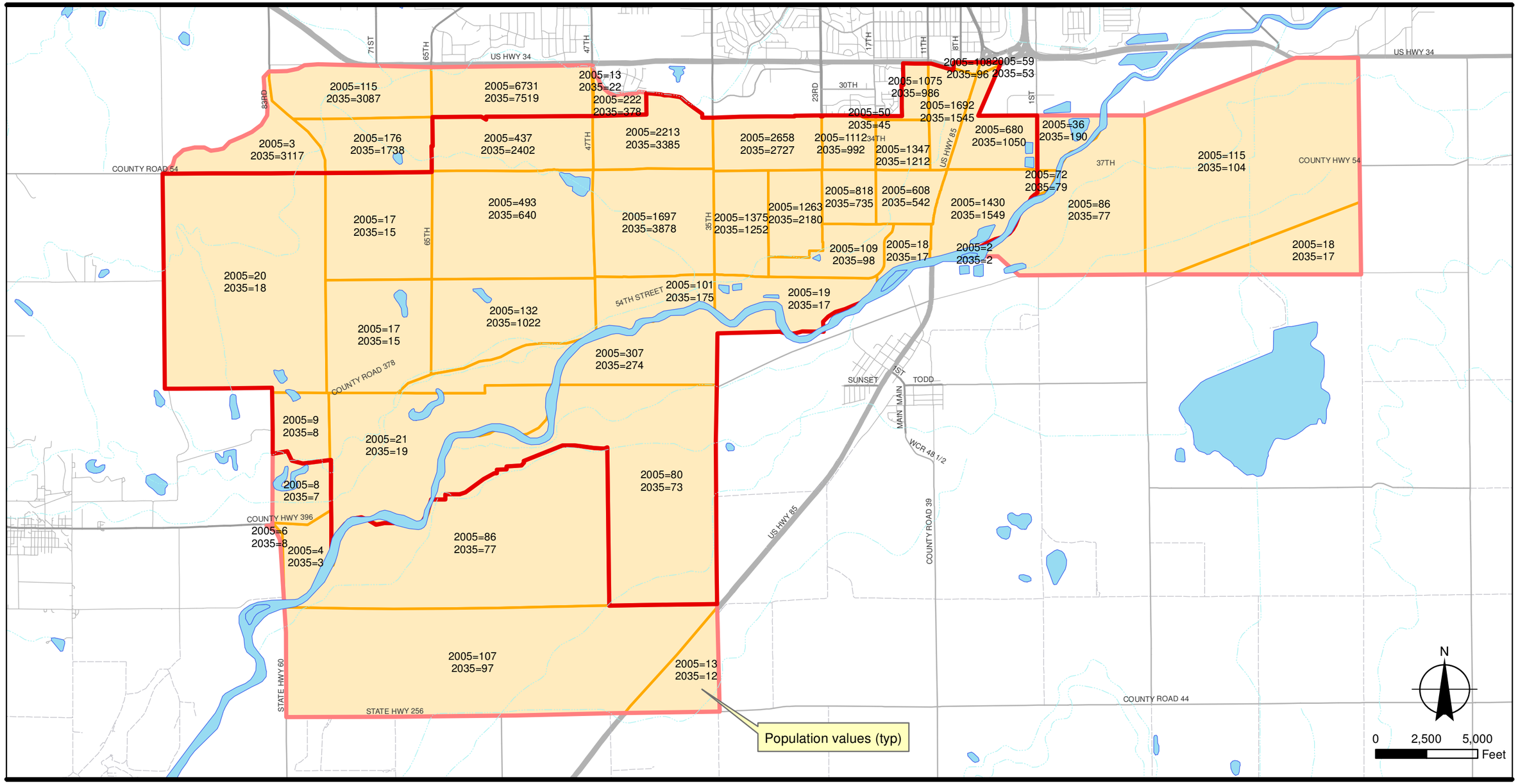
2.6. Analysis of Wastewater Flows

2.6.1. Wastewater Treatment Facility Hydraulic Capacity

The current rated hydraulic capacities of the Evans WWTF and Hill-n-Park WWTF are average daily flows of 1.2 and 0.5 million gallons per day (mgd), respectively. The WWTF hydraulic capacity includes sanitary flows associated with existing conditions, land use and increased flow due to the system infiltration and inflow (I/I) response as discussed in detail in the following sections.

2.6.2. Wastewater Treatment Facility Flow Recording

The Evans WWTF and Hill-n-Park WWTF both make use of influent and effluent Parshall Flumes for plant flow recording. Influent flow to each facility is measured with a 9-inch Parshall flume and ultrasonic transmitter installed on the channel upstream of the influent screen. Plant effluent flow is measured downstream of the chlorine contact channel by a 6-inch and 9-inch Parshall flume, for the Evans WWTF and Hill-n-Park WWTF, respectively. Measuring devices at each WWTF provide recordings on daily sewer charts.



Legend

- Streams
- TAZ Boundaries
- LAKES
- **Service Area**
  - Planning Boundary
  - Long-term Growth/Service Area

TAZ Summary by Service Area

Description	2005 Population	2035 Population
Planning Boundary	19,978	27,042
East UPA	329	469
South UPA	224	204
North UPA	7,247	15,839
Total	**27,778**	**43,554**

Evans Wastewater Utility Plan

TAZ Population Distribution

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Figure 2.6
July 18th, 2011

2.6.3. Historical Flow Data

2.6.3.1. Evans WWTF

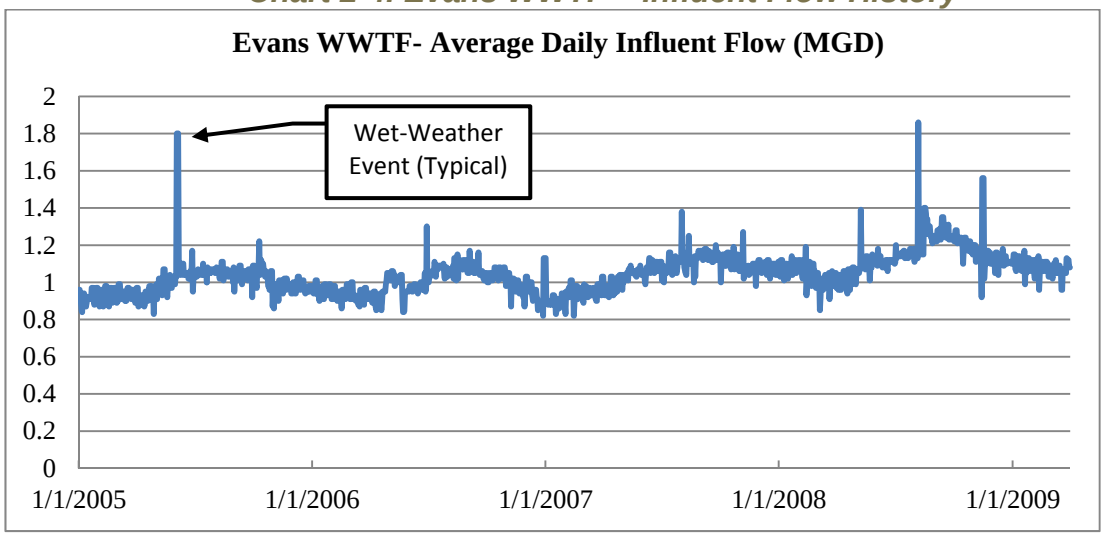
Historical flow data are required to accurately size treatment facilities and determine whether existing facilities are at their design capacity. Average monthly flow data from January 2005 through December 2009 for the Evans WWTF can be found in Table 2-11. Chart 2-4 provides the influent flow readings for the following data set.

Table 2-11. Historical Recorded Flow Data for the Evans WWTF

Month	2005 Flow, MGD	2006 Flow, MGD	2007 Flow, MGD	2008 Flow, MGD	2009 Flow, MGD
January	0.922	0.957	0.905	1.080	1.101
February	0.933	0.942	0.946	1.073	1.084
March	0.938	0.932	0.956	1.005	1.072
April	0.927	0.936	0.979	1.022	1.100
May	0.989	0.992	1.048	1.102	1.140
June	1.129	0.998	1.081	1.109	1.310
July	1.051	1.063	1.090	1.149	1.410
August	1.059	1.084	1.126	1.264	1.240
September	1.039	1.067	1.153	1.267	1.230
October	1.046	1.043	1.118	1.215	1.210
November	0.978	0.983	1.083	1.173	1.170
December	0.979	0.951	1.079	1.120	1.180
Yearly Average	0.999	0.996	1.047	1.132	1.187
Summer Average[1]	1.032	1.022	1.080	1.153	1.238
Winter Average[2]	0.966	0.968	1.015	1.111	1.136
Max Monthly Average	1.129	1.084	1.153	1.267	1.410

[1] Average of April through September
[2] Average of January, February, March, October, November, and December

Chart 2-4. Evans WWTF – Influent Flow History



The data indicates flows have been fairly consistent over the time period reported (January 2005 to December 2009) with only a slight increase over the past couple of years. July 2009 has the highest monthly average flow (1.41 MGD) found in the reporting period and the lowest average monthly flow (0.905 MGD) was reported in January 2007. Summer and winter flows vary by approximately 5 percent over the four year period indicating the collection system is minimally influenced by wet and dry months. However, the system does appear to be influenced by wet weather rain events as show on Chart 2-4 indicating a problem with inflow. The annual average influent flow for the last full year of data, 2009, was 1.187 MGD. The peak hour flow for the period was approximately 3 MGD recorded on August 6, 2008.

2.6.3.2. Hill-n-Park WWTF

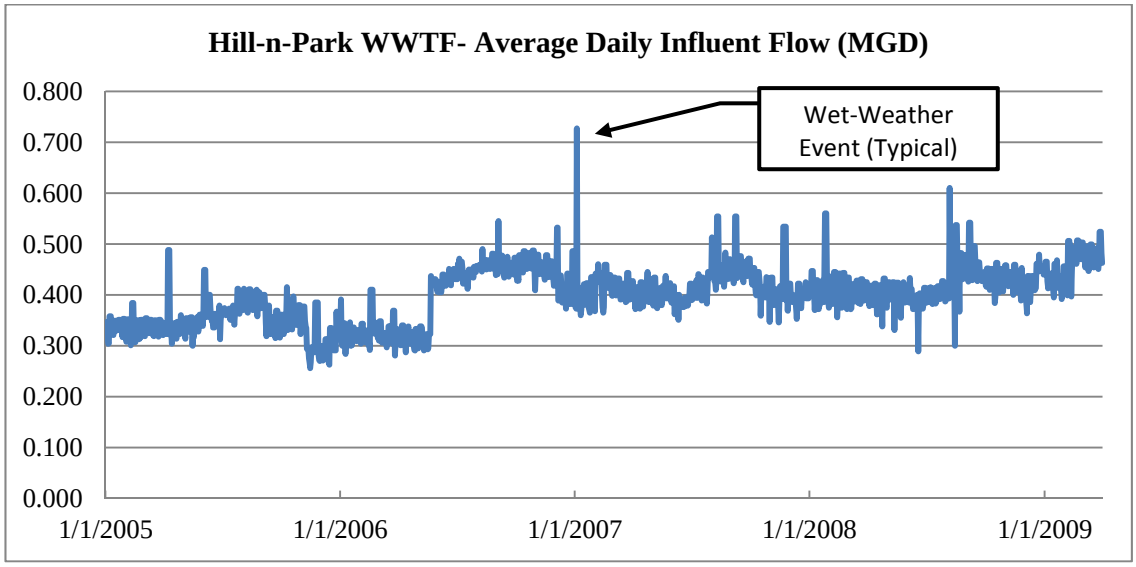
Average flow data from January 2005 through December 2009 for the Hill-n-Park WWTF can be found in Table 2-12. Chart 2-5 provides the influent flow reading for the following data set.

Table 2-12. Historical Recorded Flow for the Hill-n-Park WWTF

Month	2005 Flow, MGD	2006 Flow, MGD	2007 Flow, MGD	2008 Flow, MGD	2009 Flow, MGD
January	0.339	0.324	0.411	0.423	0.435
February	0.336	0.336	0.426	0.416	0.474
March	0.338	0.315	0.413	0.408	0.480
April	0.351	0.318	0.407	0.399	0.477
May	0.343	0.352	0.404	0.407	0.495
June	0.367	0.428	0.391	0.392	0.474
July	0.371	0.446	0.417	0.406	0.524
August	0.390	0.461	0.455	0.451	0.506
September	0.355	0.460	0.461	0.459	0.472
October	0.357	0.464	0.419	0.432	0.487
November	0.322	0.456	0.425	0.434	0.485
December	0.306	0.427	0.398	0.433	0.499
Yearly Average	0.384	0.399	0.419	0.422	0.484
Summer Average[1]	0.363	0.411	0.423	0.419	0.491
Winter Average[2]	0.333	0.387	0.419	0.424	0.477
Max Monthly Average	0.390	0.464	0.461	0.459	0.524

[1]Average of April through September
[2]Average of January, February, March, October, November, and December

Chart 2-5. Hill-n-Park WWTF – Influent Flow History



Data indicates that flows have been steadily increasing over the time period reported (January 2005 to March 2009). July 2009 provides the highest monthly average found in the reporting period (0.524 MGD) and the lowest monthly average flow (0.306 MGD) was reported in December 2005. The annual average influent flow for the last full year of data, 2009, was 0.484 MGD. The peak hour flow for the data was 0.9 MGD recorded on September 4, 2006. Similar

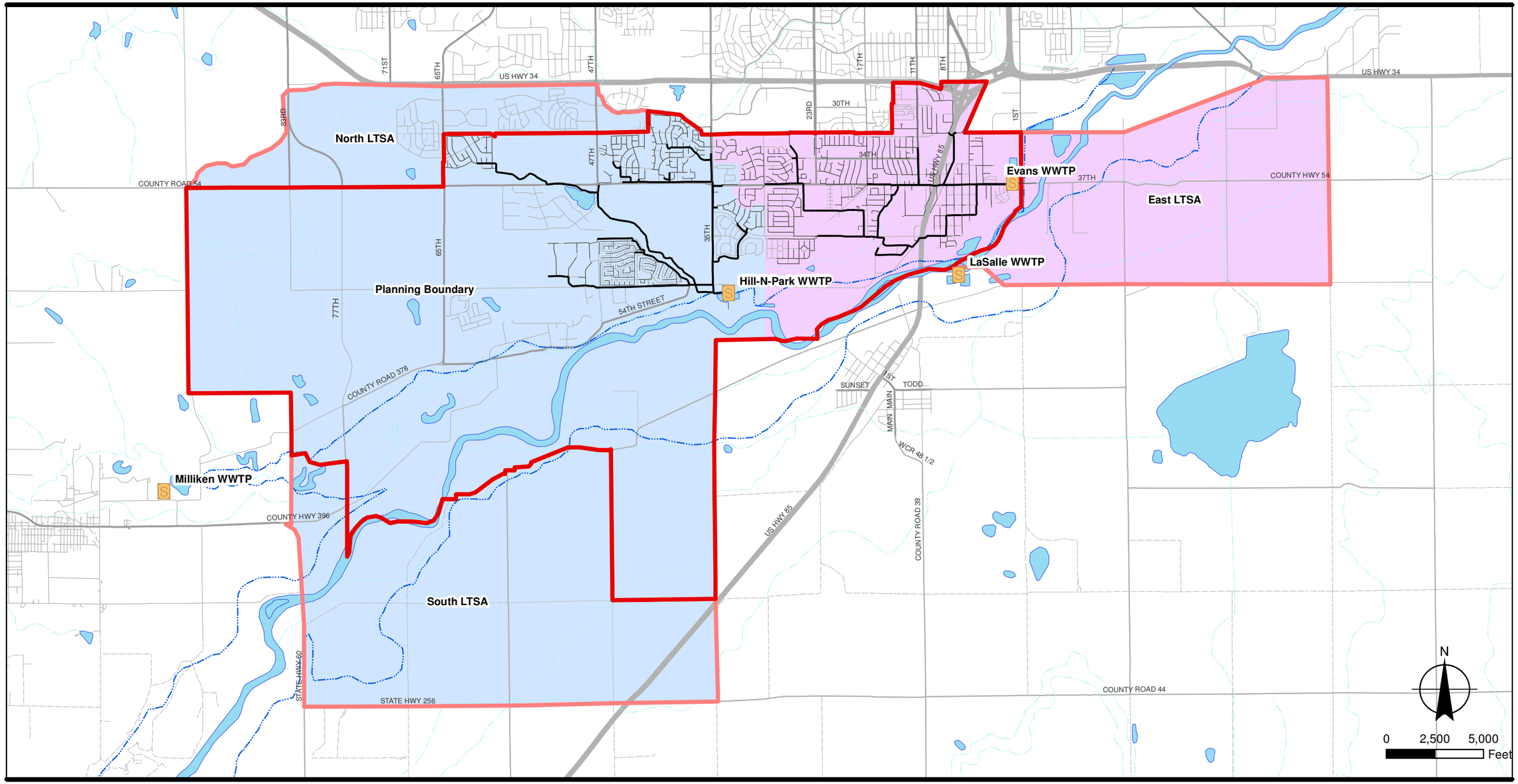
to the Evans WWTF system, the Hill-n-Park WWTF is influenced by wet weather rain events.

2.6.4. Wastewater Treatment Facility Service Basins

The City of Evans sewer collection system is comprised of two separate service basins, one for the Evans WWTF and one for the Hill-n-Park WWTF. Table 2-13 summarizes the service basins area, existing population and average daily flow. The existing area served by the Evans WWTF is approximately 1,132 acres and by the Hill-n-Park WWTF is approximately 488 acres. Based on a total existing population of approximately 21,746 inside the planning boundary, the estimated population being served by the Evans WWTF is approximately 12,380, while the estimated population served by the Hill-n-Park WWTF is approximately 9,366. The population split was established by delineating the service basins based on the service to parcels by each collection system and intersecting them with the landuse-based population projections. Figure 2-7 shows the collection system service basins served by two wastewater treatment facilities.

Table 2-13. Existing WWTF Service Basins

Service Basin	Total Served Area (Acres)	Existing Population	Average Daily Flow, gpd
Evans	1,132	12,380	1,187,000
Hill-n-Park	488	9,366	484,000
Total	**1,620**	**21,746**	**1,671,000**



Legend

- Treatment Plants
- Streams
- Floodplain (Approx.)
- Exist. Local Sewer
- Exist. Collector/Interceptor Sewer

Service Area

- Planning Boundary
- Long-term Growth/Service Area

WWTF Service Basin

- Evans
- Hill-n-Park

Evans Wastewater Utility Plan

Wastewater Treatment Facility Service Basins

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Figure 2.7
July 18th, 2011

2.6.5. Average Dry-Weather Unit Flow Factors

Average dry-weather flow (ADWF) projections of wastewater flow and loading are usually performed by first determining current unit flow factors from existing population, commercial, and industrial contributions. These unit flow factors, in conjunction with population and land use projections, can then be used to determine future treatment plant influent flows. Flow contribution from residential, commercial and industrial users is expected to remain constant over time. These unit flow factors are derived from flows within the planning boundary only; however they were applied as representative factors to future population and landuse inside the LTSA.

2.6.5.1. Unit Flow Factors

The current flow treated at both facilities consists of contributions from residential, commercial, and industrial users. Citywide there are currently 8,116 residential and 165 commercial/industrial wastewater service connections. A number of standard documents were reviewed for typical unit flow factors. Based on these typical unit factors and commercial/industrial flow data from Evans, system-specific unit flow factors were developed.

2.6.5.2. Commercial & Industrial

For commercial and industrial flows, typical unit factors are more difficult to establish since there can be a wide range of flows largely based on the type and size of the business. In standards including the NFRWQPA's Utility Plan Guidance, unit flow factors are typically based on gallons per employee or per customer. However, this level of detail is not available for Evans; therefore a gpd per acre unit factor were used for commercial and industrial flows.

A sampling of recent water demands for different commercial users in Evans was provided to HDR. For this particular analysis, it was assumed all of the water demand resulted in sanitary flow. Five representative commercial users were selected from the list excluding very high users, for example, a car wash. Table 2-14 presents the five representative commercial users with corresponding sanitary flows, acreage, and gpd/acre factors. The average sanitary flow was 919 gpd/acre over the five users. Therefore, a 1,000 gpd/acre unit flow factor for planning purposes has been utilized for commercial and industrial flows. This value is within the range of 80 to 1,500 gpd/acre documented by Metcalf & Eddy, Inc., Wastewater Engineering Treatment/Disposal/Reuse, for commercial land use.

Table 2-14. Evans Representative Commercial Users

Representative User	User Type	Average Daily Flow, gpd	Sewered Area, acre	Unit Flow, gpd/acre
1	Commercial	5,992	4.5	1,332
2	Restaurant	1,572	2.0	786
3	Restaurant	1,532	2.2	696
4	Commercial	1,071	2.6	412
5	Restaurant	1,370	1	1,370
Average	**-**	**2,307**	**2.5**	**919**

The sewered area of existing commercial and industrial areas in the planning boundary was established by taking the existing land use coverage and using recent aerial photos subtracting undeveloped and non-sewered parcels. The resultant commercial and industrial areas more accurately reflect sewered parcels that generate flows to the treatment facilities. It is assumed

that the average effective flow-generating area within the sewered parcels is half of the total parcel area.

2.6.5.3. Residential

For residential flows, unit factors are typically based on gallons per capita or household. Due to the population information gathered in the previous section, a gallons per capita per day (gpcd) unit flow factor were used for residential flows. EPA guidance documents recommend an average per capita flow of 120 gpcd or less. Recommended Standards for Wastewater Facilities (10 State Standards) use a contribution of 100 gpcd unit factor for residential flows. CDPHE guidelines advise using 80 gpcd for residential contributions only. NFRWQPA's Utility Plan Guidance document recommends using a residential wastewater flow factor from the 208 Plan of 85 gpcd, which includes a 10 gpcd base infiltration and inflow component.

The existing landuse data for Evans identifies residential and mobile home areas. A separate unit flow factor were assigned to population in mobile home areas due to the smaller sanitary flow contribution.

Evans WWTF

For the purpose of calculating unit flow factors for the Evans WWTF service basin, the average monthly flow of 1.101 MGD from January 2009 was used along with the estimated existing population served in the service basin. The purpose of using the average flow conditions for January 2009 is to determine loadings without significant influence from infiltration or inflow as there was little to no precipitation recorded during this month. Table 2-15 summarizes the unit flow factor determination for the Evans WWTF service basin. The estimated Evans WWTF per capita flow contribution of 79 gpcd is in line with both NFRWQPA and CDPHE recommended residential flow rates.

Table 2-15. Evans WWTF Unit Flow Factors

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	12,144	773	959,376	*79*	1,241
Mobile Home	236	17	15,340	*65*	902
Commercial / Industrial	0	124	124,100	-	*1,000*
Total	**12,380**	**914**	**1,098,816**	-	-

[1] Unit flow in gallons per day per capita (gpcd) was calculated by dividing flow by the residential population.
[2] Unit flow was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2.

Hill-n-Park WWTF

For the Hill-n-Park WWTF, the sewer collection system appears to experience minimal infiltration and inflow. Examination of flow data shows that August and September are typically the months with the highest flows while April through June see the lowest flows.

For the purpose of calculating per capita flow and loading, an average monthly flow of 0.435 MGD from January 2009 was used along with the estimated existing population served in the service basin. The purpose of using the January 2009 loading conditions is to determine loadings without significant influence from infiltration or inflow as there was little to no precipitation recorded during this month. Table 2-16 summarizes per capita wastewater loadings for the Hill-n-Park WWTF. The estimated Hill-n-Park WWTF per capita flow contribution of 46 gpcd is quite low compared to standard residential flow rates. The collection

system in the Hill-n-Park service area is relatively new and the pipe material is predominately PVC, so the base infiltration factor is much lower than what is found in other similar systems which accounts for a portion of the discrepancy. Other possible causes for the lower unit per capita include inaccurate readings from flow meters, and the inclusion of unoccupied platted areas in the analysis.

Table 2-16. Hill-n-Park WWTF Unit Flow Factors

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	8,625	449	396,750	*46*	884
Mobile Home	741	61	33,345	*45*	547
Commercial / Industrial	0	5	5,200	0	*1,000*
Total	**9,366**	**515**	**435,295**	-	-

[1] Unit flow in gallons per day per capita (gpcd) was calculated by dividing flow by the residential population.
[2] Unit flow was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2.

2.6.5.4. Recommended Unit Flow Factors

Due to the inconsistency of flow factors between the two service basins, recommended unit flow factors have been established based on the Evans WWTF service basin unit flow factors. The Evans WWTF service basin flow factors more closely resemble typical unit factors and will not cause low flow projection numbers for planning purposes. The higher flow factor also helps account for deteriorating pipe conditions which allows for increasing base infiltration over time. Table 2-17 presents recommended unit flow factors for the basis of planning and flow projection for all future flow contributions.

Table 2-17. Recommended Unit Flow Factors

Classification	Unit Flow, gpcd	Unit Flow, gpd/acre
Residential	80	-
Mobile Home	65	-
Commercial / Industrial	-	1,000[1]
Mixed Use	80	500[2]

[1] The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2.
[2] Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density of population and half of the Commercial unit flow.

2.6.6. Peaking Factors

It is necessary to estimate peaking factors during flow conditions for analysis and design purposes. Flow data collected over the past four years at each of the two treatment facilities was analyzed to establish the peaking factors. Peak hour dry-weather factors have been developed for use in this study. Peaking factors use daily flow from an average month as the basis of comparison. The maximum day peaking factor allows for analysis of flow conditions during a typical day of maximum ADWF. The peak hour peaking factor allows for analysis of flow conditions during a typical peak hour of ADWF. These two flow conditions aid in determining the upgrades that will need to be made to the treatment facilities and the conveyance system in order to meet the future projected demand.

Peaking factors are dependent on the diurnal flow pattern of residential, commercial, and industrial activities. Examination of flow data from each of the treatment facilities shows that wastewater generation reaches a daily maximum between the periods of 7:00 a.m. to 9:00 a.m. and 6:00 p.m. to 9:00 p.m. The daily minimum occurs between the hours of 11:00 p.m. to 6:00 a.m. Wastewater maximum and minimum flows are also dependent on the size and configuration of wastewater collection facilities. Typically the larger the system is, geographically and the amount of flow, the smaller the peaking factor. Systems with steep sewers result in rapid conveyance of flow and high peaks. Sewer collections systems with mild slopes tend to have moderate peaks.

The ratio of recorded peak hour flow to recorded average daily dry-weather flow for the Evans WWTF was approximately 1.6 to 1. For the Hill-n-Park WWTF, the ratio was approximately 1.4 to 1. Outside the planning boundary, a peaking factor of 1.5 (an average of the two peaking factors inside the planning boundary) is utilized to project flows in the LTSA. Table 2-18 presents the peaking factors for each WWTF and outside the planning boundary. Considering these peaking factors, both systems are only moderately affected by peak flows compared to other wastewater collection systems.

Table 2-18. WWTF ADWF Peaking Factors

	Evans WWTF	Hill-n-Park WWTF	Outside Planning Boundary
Average Month	1.0	1.0	1.0
Peak Hour	1.6	1.4	1.5

In addition to peaking factors minimum flow projections are important units to consider for planning purposes to size future collection systems and treatment processes. With the potential for growth in and around the Evans area the minimal flow conditions that have occurred in the last 5 years are taken into consideration in the recommendations for future improvements to the treatment facilities presented in Chapter 5.

2.6.7. Infiltration and Inflow (I/I)

Infiltration and Inflow (I/I) is an important aspect to consider when analyzing a system. With the presence of groundwater, infiltration can influence the system during dry weather. This groundwater induced infiltration is often referred to as base infiltration. More influential through a rainfall event and for a period afterwards, flows within the collection system rise in response to the storm. This increase in flow is known as the rainfall-induced I/I (RDII). These I/I conditions can increase flow in the system by more than 100 percent and stress the capacity of the sewer system. Therefore, the peak I/I flows for existing and buildout conditions should be developed and included in the analysis of the collection and treatment systems.

Infiltration occurs where groundwater levels above the sewer leak into the pipeline. Inflow occurs where surface water from snow melt or rain leaks into the sewer through manhole lids, cleanouts, or direct connections of storm drainage or roof drains to the sewer system. Inflow typically shows up as extraordinary peak flows during or immediately following storm events while infiltration typically results in additional base flow entering the system on a more consistent basis.

2.6.7.1. Base Infiltration

Base infiltration occurs even during dry weather from groundwater as influenced by Evans area creeks and irrigation ditches. Since a comprehensive I/I study has not been conducted, the base infiltration is captured in the unit flow factors used to establish ADWF for population and land use categories as discussed in previous sections. The base infiltration may be heavily influenced by the irrigation ditches when flowing and should be considered as a significant variable in future studies.

2.6.7.2. Wet-Weather Flows

Wastewater flows during wet weather or periods of high groundwater provide insight into the volume of I/I entering a system. Examination of flow data in Tables 2-11 and 2-12 show that August and September are typically the months with the highest flows while January through April see the lowest flows. In early August of 2008, the Evans service area received approximately 3 IN of precipitation over a 24 hour period that generated higher than average flows at each wastewater treatment facility. A peak flow of 3.02 MGD and 1.5 MGD was observed at the Evans WWTF and Hill-n-Park WWTF, respectively on the morning of August 7th, 2008. The difference between the normal ADWF and the peak I/I flow is approximately 2 MGD for the Evans WWTF and 1.1 MGD for the Hill-n-Park WWTF.

2.6.7.3. Peak Hour I/I Flow Factors

For the Evans wastewater systems, estimates for total I/I flow response throughout the system during wet-weather have been made based on a gallon per day per acre (gpd/ac) basis. Since detailed I/I analysis has not been conducted across the planning boundary, a system-wide I/I factor was be used. An estimate of a maximum I/I factor for wet-wet weather flows is arrived at by taking the total flow and dividing by the total service area. In the future, temporary flow meter data should be obtained so a better understanding of the system response to rainfall can be determined. As a result, the I/I determination method discussed above should be modified to reflect the system's time and location varying, dynamic response to rainfall.

The total existing sewer service area is approximately 1,620 acres with 1,132 and 488 acres in the Evans WWTF and Hill-n-Park WWTF service basins, respectively. This equates to peak hour I/I factors of approximately 1,767 gpd/ac and 2,254 gpd/acre in the Evans WWTF and Hill-n-Park WWTF service basins, respectively. The I/I factors differ between the service basins depending on the ratio of increased peak I/I flow to the total sewered area for each service basin and their individual response to rainfall induced I/I (RDII). From an average of the I/I factors from each service basin, a peak hour I/I factor of 2,011 gpd/acre was used system-wide as the basis of wet-weather flow projection. Metcalf & Eddy, Inc., Wastewater Engineering Treatment/Disposal/Reuse, reports a range of expected infiltration of 20 to 3,000 gallons per acre per day so the Evans system is in the middle of the range. Table 2-19 summarizes the approximate wet-weather flow increases (system response) at the WWTFs influent with the corresponding wet-weather I/I flow factors.

Table 2-19. Peak Hour I/I Flow Factors

Service Basin	Total Area (acres)	Increased Influent Flow(mgd)	I/I Factor (gpd/acre)
Evans WWTF	1,132	2.0	1,767
Hill-n-Park WWTF	488	1.1	2,254
Average	-	-	2,011

2.6.7.4. Wet-Weather Peaking Factor Comparison

Many times, peak hour wet-weather flows are established by peaking factor formulas from standard planning documents. A few standards have been referenced to compare these peaking factors to Evans wet-weather flow factors based on a gpd/acre I/I factor. Verification using this comparison can provide a level of consensus between wet-weather flow projection approaches.

CDPHE Policy 96-1, Design Criteria Considered in the Review of Wastewater Treatment Facilities recommends a peaking factor (ratio of peak hour wet-weather flow to average daily flow) not be less than 4 for laterals and sub-main sewers and not less than 2.5 for main, trunk, and outfall sewers.

NFRWQPA's Utility Plan Guidance document determines the hourly wet-weather peaking factor using the following formula based on average total flow in mgd.

$$Wet - Weather\ Peaking\ Factor = \frac{3.65}{(Average\ Total\ Flow\ MGD)^{0.167}}$$

$$(The\ maximum\ peaking\ factor\ value\ used\ in\ any\ assessment\ is\ 5.0)$$

Using this formula for 2008 yearly average daily flows, the peaking factors are 3.6 and 4.2 for the Evans and Hill-n-Park treatment facilities, respectively.

Recommended Standards for Wastewater Facilities (10 State Standards) determines the hourly wet-weather peaking factor using the following formula based on populations in thousands.

$$Wet - Weather\ Peaking\ Factor = \frac{18 + \sqrt{Population\ in\ 1000s}}{4 + \sqrt{Population\ in\ 1000s}}$$

Using this formula with the 2008 population numbers, the peaking factors are 2.9 and 2.1 for the Evans and Hill-n-Park treatment facilities, respectively.

In Section 2.7.3., these three standard wet-weather peaking factors values were compared to the wet-weather flow projection peaking factors based on the gpd/acre I/I factor approach.

2.7. Wastewater Utility Service Area Flow Projections

Total annual average dry weather flow (ADWF) or base flow projections in the collection system for future land use conditions are comprised of residential, commercial, industrial, and mixed used contributions. In addition to existing (2010) and future (2030) conditions, flow projections were developed for the intermediate year of 2020 for planning purposes. The existing and future flow projections are used as the basis of analysis for the collection system and treatment processes. The flow projections utilize the unit flow and I/I factors established previously. Mixed use projections are based on half residential and half commercial area.

2.7.1. Evans WWTF

Based on unit flow factor recommendations presented above, flow projections for the Evans WWTF have been developed. The unit flow and I/I factors were combined with existing and future population and landuse coverage and peaking factors to generate projections of future flow. Tables 2-20 and 2-21 show the Evans WWTF ADWF projections for the intermediate (2020) and future (2030) conditions. Table 2-22 summarizes flow projections for existing (2010), intermediate (2020), and future (2030) conditions.

Table 2-20. Evans WWTF Intermediate ADWF Projection (Year 2020)

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	12,863	863	1,029,040	*80*	1,192
Commercial / Industrial	0	234	234,000	-	*1,000*
Mixed Use	78	16	14,240	*80*	*500*
Total	**12,941**	**1,113**	**1,277,280**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density of population and half of the Commercial unit flow.

Table 2-21. Evans WWTF Future ADWF Projection (Year 2030)

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	13,352	936	1,068,160	*80*	1,141
Commercial / Industrial	0	320	320,000	-	*1,000*
Mixed Use	150	64	44,000	*80*	*500*
Total	**13,502**	**1,320**	**1,432,160**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density of population and half of the Commercial unit flow.

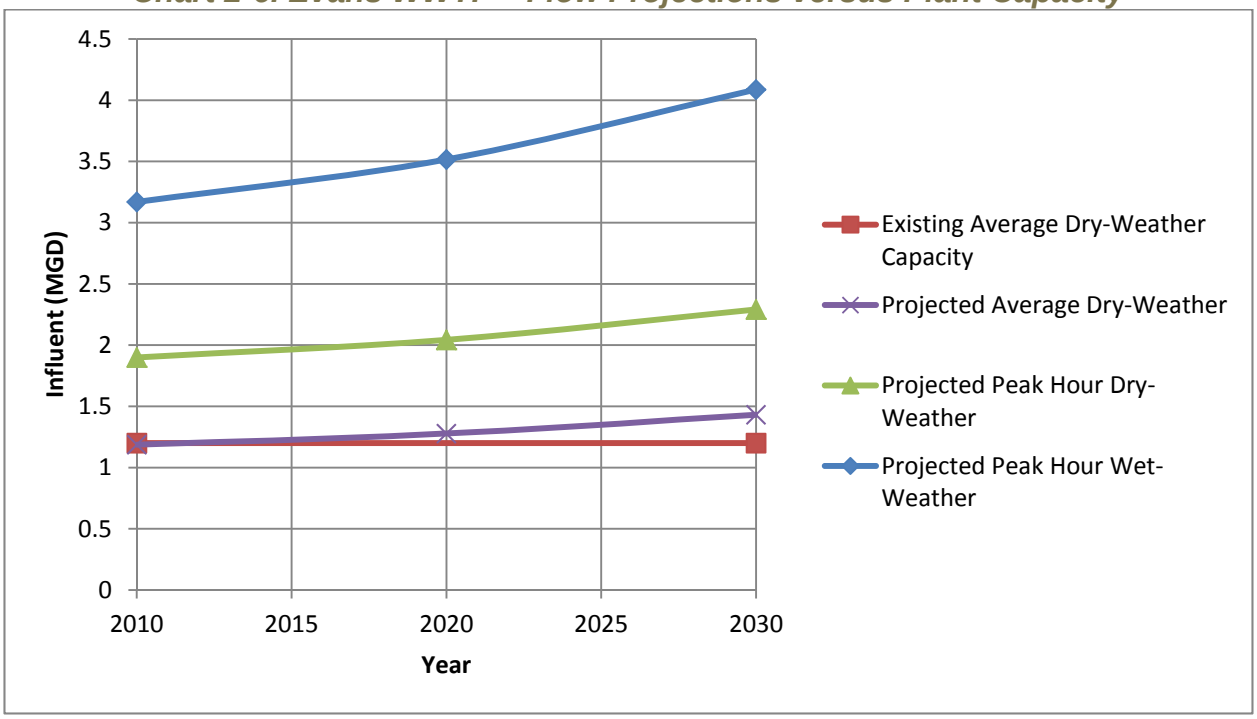
Table 2-22. Evans WWTF Projected Wastewater Flows

Criteria	Existing (2010)	Intermediate (2020)[1]	Future (2030)[1]
Sewered Population	12,380	12,941	13,502
Average Dry-Weather Flow, MGD[2]	1.230	1.277	1.432
Peak Hour Dry-Weather Flow, MGD[3]	1.968	2.043	2.291
Peak Hour Wet-Weather Flow, MGD[4]	3.284	3.516	4.087

[1]Assume half of the un-sewered parcels connect to the collection system by 2020 and the remaining by 2030.
[2]Yearly average from 2009 flow data.
[3]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor of 1.6.
[4]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).

Chart 2-6 depicts the information provided in Table 2-22 and plotting the increase in influent over time provides a visual illustration of the plant capacities required over time to buildout.

Chart 2-6. Evans WWTF – Flow Projections versus Plant Capacity



2.7.2. Hill-n-Park WWTF

Based on unit flow factor recommendations presented above, flow projections for the Hill-n-Park WWTF have been developed. At buildout, the existing developments with current septic system service are assumed to be served by a collection system and influent would be conveyed to the Hill-n-Park WWTF. The unit flow factors were combined with existing and future population and landuse coverage within the planning boundary and Saint Michaels Subdivision of Greeley, in conjunction with peaking factors to generate projections of future flow. Tables 2-23 and 2-24 provide the Hill-n-Park WWTF ADWF projections for the intermediate (2020) and future (2030) conditions. Table 2-25 summarizes flow projections for existing (2010), intermediate (2020), and future (2030) conditions.

Table 2-23. Hill-n-Park WWTF Intermediate ADWF Projection (Year 2020)

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	19,998	2,615	1,599,840	*80*	612
Commercial / Industrial	0	272	272,000	-	*1,000*
Mixed Use	307	106	77,560	*80*	*500*
Total	**20,305**	**2,993**	**1,949,400**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density of population and half of the Commercial unit flow.

Table 2-24. Hill-n-Park WWTF Future ADWF Projection (Year 2030)

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	28,038	4,781	2,243,040	*80*	469
Commercial / Industrial	0	517	517,000	-	*1,000*
Mixed Use	445	212	141,600	*80*	*500*
Total	**28,483**	**5,510**	**2,901,640**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow for Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density of population and half of the Commercial unit flow.

Table 2-25. Hill-n-Park WWTF Projected Wastewater Flows

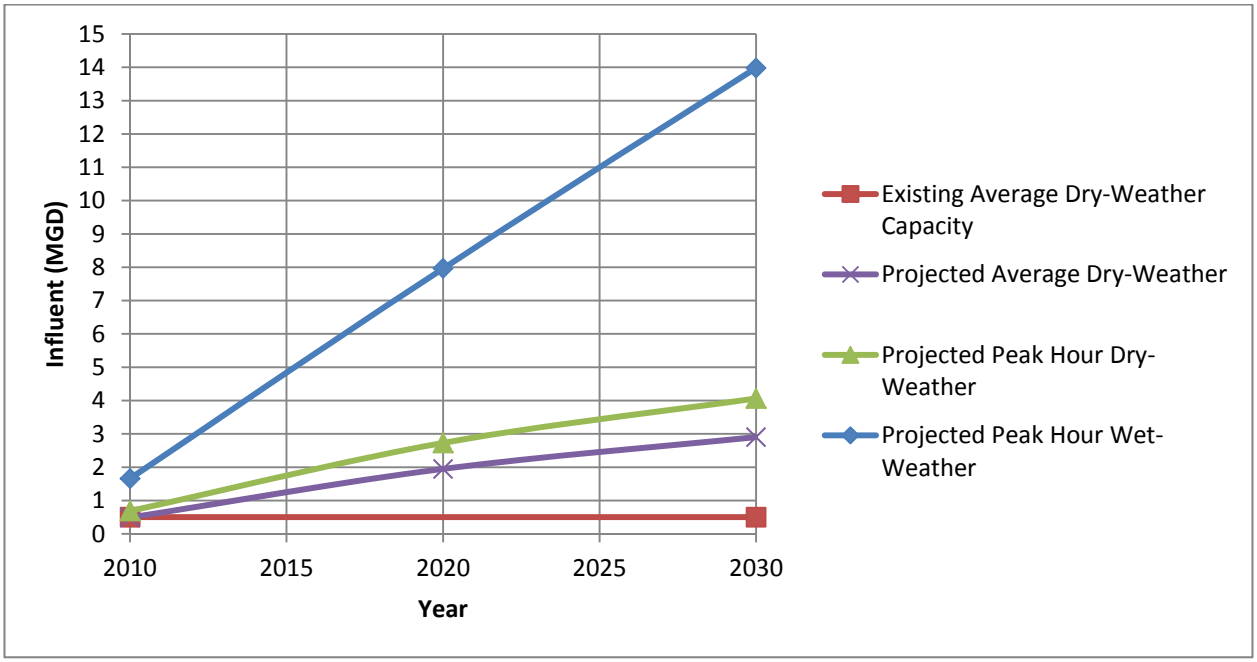
Criteria	Existing (2010)	Intermediate (2020)[1]	Future (2030)[1]
Sewered Population	9,366	20,305	28,483
Average Dry-Weather Flow, MGD[2]	0.491	1.949	2.902
Peak Hour Dry-Weather Flow, MGD[3]	0.687	2.729	4.063
Peak Hour Wet-Weather Flow, MGD[4]	1.660	7.968	13.982

[1]Assume half of the un-sewered parcels connect to the collection system by 2020 and the remaining by 2030.
[2]Yearly average from 2009 flow data.
[3]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor of 1.4.
[4]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).

With available land to expand, the future planning boundary could potentially include twice the amount of serviced acres than they have today, drastically increasing the infiltration potential which is reflected in the peak hour wet weather flow projection in table 2-25. This large peak hour wet-weather flow is fairly conservative.

Chart 2-7 depicts the information provided in Table 2-25 and plotting the growth over time provides a visual illustration of the plant capacities required over time to buildout.

Chart 2-7. Hill-n-Park WWTF – Flow Projections versus Existing Plant Capacity



2.7.3. Wet-Weather Peaking Factor Comparison

A comparison between approaches of developing peak hour wet-weather factors has been done to verify the gpd/acre-based I/I flow projections approach used. The CDPHE, NFRWQPA and 10 State Standard peaking factors are compared to the calculated peaking factor based on the wet-weather flow projections presented above. Table 2-26 shows the comparison between peaking factor approaches. The gpd/acre peaking factors are in the same range as the values given by the standard peaking formulas.

Table 2-26. Comparisons of Peaking Factors between Approaches

	Peaking Factors from Different Approaches			
Service Basin	**gpd/acre**	**CDPHE**	**NFRWQPA**	**10 State Standards**
Evans WWTF	2.67	> 2.5	3.6	2.9
Hill-n-Park WWTF	3.38	> 2.5	4.2	2.1

2.7.4. LTSA Flow Projections

Based on unit flow factor recommendations presented above, flow projections for the three LTSA have been developed. The unit flow and I/I factors were combined with existing and future population and landuse coverage and peaking factors to generate projections of future flow. Tables 2-27, 2-28, and 2-29 show the LTSA ADWF projections for the East, South, and North areas, respectively, for the buildout condition. Table 2-30 summarizes flow projections for theoretical build-out conditions.

Table 2-27. East LTSA Buildout ADWF Projection

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	8,321	1,846	665,680	*80*	361
Commercial / Industrial	0	87	87,000	-	*1,000*
Mixed Use	71	33	22,180	*80*	*500*
Total	**8,392**	**1,966**	**774,860**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow or Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density population and half of the Commercial unit flow.

Table 2-28. South LTSA Buildout ADWF Projection

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	5,417	1,994	433,360	*80*	217
Commercial / Industrial	0	593	593,000	-	*1,000*
Mixed Use	0	0	0	*80*	*500*
Total	**5,417**	**2,587**	**1,026,360**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow or Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density population and half of the Commercial unit flow.

Table 2-29. North LTSA Buildout ADWF Projection

Land Use Classification	Residential Population	Sewered Area, acres	Flow, gpd	Unit Flow[1], gpcd	Unit Flow[2], gpd/acre
Residential	4,535	1,278	362,800	*80*	284
Commercial / Industrial	0	59	59,000	-	*1,000*
Mixed Use	0	0	0	*80*	*500*
Total	**4,535**	**1,337**	**421,800**	**-**	**-**

[1] Recommended unit flow factor.
[2] Unit flow in gpd/acre was calculated by dividing flow by the sewered acres for Residential and Mobile Home. The unit flow or Commercial and Industrial is an established unit flow developed in Section 2.6.5.2. Mixed Use is assumed as half Residential and half Commercial and is allocated a lower density population and half of the Commercial unit flow.

Table 2-30. LTSA Projected Buildout Wastewater Flows

Criteria	East LTSA	South LTSA	North LTSA
Sewered Population	8,321	5,417	4,435
Average Dry-Weather Flow, MGD	0.775	1.026	0.422
Peak Hour Dry-Weather Flow, MGD[1]	1.163	1.539	0.633
Peak Hour Wet-Weather Flow, MGD[2]	4.728	6.229	3.111

[1]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[2]Calculated from the maximum day dry-weather flow with increased flow using the I/I factor multiplied by the sewered area.

2.8. Analysis of Wastewater Characteristics

In order to appropriately size and select process equipment for wastewater treatment facilities, characteristics of the wastewater, such as volumetric flow, organic strength, suspended solids content, and nutrient loading must be evaluated.

Important characteristics include biochemical oxygen demand, the total suspended solids, phosphorus and nitrogen. The five-day biochemical oxygen demand (BOD_5) measures the dissolved oxygen consumed in degrading organic matter, and is an indirect measure of the organic strength of the wastewater. Total suspended solids (TSS) is a measure of particulate matter in the wastewater. Phosphorus and nitrogen are plant nutrients that can cause unwanted algal growth. Phosphorous is found in wastewater both as organic and inorganic phosphate. Nitrogen is present in both un-oxidized (total Kjeldahl nitrogen (TKN), including ammonia and organic nitrogen), and oxidized forms (nitrates and nitrites).

2.8.1. Historical Loading Data

Historical loading data is required to accurately size treatment facilities and determine whether existing facilities are at their design capacity.

2.8.1.1. Evans WWTF

Average data from January 2005 through December 2009 for flow, BOD, TSS, and ammonia for the Evans WWTF can be found in Table 2-31.

Table 2-31. Historical Recorded Flow and Loading Data for the Evans WWTF

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Influent NH_3-N, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
January 05	0.922	260	271	-	27	51	29.8
February 05	0.933	266	271	-	22	58	29
March 05	0.938	256	302	-	25	65	26.8
April 05	0.927	261	335	-	28	52	25.8
May 05	0.989	268	306	-	36	64	21.7
June 05	1.129	257	464	-	26	55	3.1
July 05	1.051	266	523	-	26	44	0.8
August 05	1.059	336	654	-	24	44	0.2
September 05	1.039	305	826	30	18	36	0.5
October 05	1.046	301	750	30	14	23	11.4
November 05	0.978	312	629	30	18	34	23
December 05	0.979	258	298	37	27	38	29
January 06	0.957	274	322	35	29	46	29.6
February 06	0.942	271	318	35	24	48	32.6
March 06	0.932	273	328	35	22	53	30.9
April 06	0.936	322	366	33	28	75	27.5
May 06	0.992	309	367	33	27	55	23.9

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Influent NH_3-N, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
June 06	0.998	268	348	32	20	58	2.3
July 06	1.063	257	321	30	20	62	0.7
August 06	1.084	262	291	31	26	70	0.5
September 06	1.067	278	298	29	27	58	3.8
October 06	1.043	291	271	32	22	43	18.3
November 06	0.983	329	294	31	23	48	28.4
December 06	0.951	288	326	34	32	48	30
January 07	0.905	317	324	37	30	41	31.8
February 07	0.946	304	310	37	35	45	32
March 07	0.956	300	319	31	30	56	30.8
April 07	0.979	309	294	31	29	62	28.1
May 07	1.048	269	243	25	28	50	13.9
June 07	1.081	246	211	21	18	48	6.7
July 07	1.090	241	183	24	19	51	3.1
August 07	1.126	228	188	22	22	46	0.8
September 07	1.153	250	203	24	17	39	2.7
October 07	1.118	282	254	26	20	27	17.3
November 07	1.083	288	278	29	19	34	26.3
December 07	1.079	291	255	27	25	42	30.5
January 08	1.080	287	314	27	33	61	29.9
February 08	1.073	295	299	29	30	49	31.8
March 08	1.005	293	308	29	30	62	32.7
April 08	1.022	297	271	32	26	75	27.1
May 08	1.102	278	261	28	30	104	17.9
June 08	1.109	252	240	24	19	51	9.9
July 08	1.149	238	225	30	20	45	21.2
August 08	1.264	220	228	21	29	66	10.4
September 08	1.267	233	212	27	29	65	12.5
October 08	1.215	239	221	33	26	75	18
November 08	1.173	271	240	31	30	75	23.4
December 08	1.120	270	206	28	33	82	29.7
January 09	1.101	289	257	32	35	52	32
February 09	1.084	297	267	30	27	57	33
March 09	1.072	309	272	33	29	63	35.5

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Influent NH_3-N, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
April 09	1.10	307	269	-	20	53	34.6
May 09	1.14	304	290	-	25	52	30
June 09	1.31	244	229	-	18	43	15
July 09	1.41	221	206	-	22	38	3.7
August 09	1.24	249	238	-	20	36	0.3
September 09	1.23	255	257	-	19	25	2.6
October 09	1.21	256	219	-	21	24	21.2
November 09	1.17	294	261	-	21	38	29.9
December 09	1.18	329	318	-	29	41	28
2005 Average	0.999	279	467	30	24	47	16
2006 Average	0.968	285	320	32	25	55	19
2007 Average	1.048	277	255	27	26	45	19
2008 Average	1.132	265	252	28	28	68	22.5
2009 Average	1.187	280	257	32	24	44	22
Sum 2005 Ave[1]	1.032	283	521	30	26	49	7.9
Sum 2006 Ave[1]	1.022	282	331	31	25	63	9.2
Sum 2007 Ave[1]	1.080	257	220	24	26	49	9.1
Sum 2008 Ave[1]	1.153	254	240	27	25	68	16.7
Sum 2009 Ave[1]	1.238	263	248	-	21	41	14
Wint 2005 Ave[2]	0.966	275	417	33	22	45	24
Wint 2006 Ave[2]	0.968	288	309	34	25	48	28.4
Wint 2007 Ave[2]	1.015	297	290	31	26	41	29.3
Wint 2008 Ave[2]	1.111	275	262	29	30	68	26.4
Wint 2009 Ave[2]	1.136	296	266	32	27	46	30

[1] Average of April through September
[2] Average of January, February, March, October, November, and December

Influent BOD, TSS, and ammonia influent loading appears to be remaining relatively consistent on a concentration basis with the four year average being 281 mg/L, 273 mg/L, and 30 mg/L respectively. In 2005, the influent TSS was measurably higher than in the following years with an average concentration of 467 mg/L. This was due to systematic cleaning of the sewer system during this calendar year.

Effluent BOD and TSS are normally below the permitted limits of 30 mg/L and 75 mg/L, respectively; however effluent concentrations have been above the permitted limits on a number of occasions. Effluent ammonia is below the required permit limit of 44 mg/L and 43 mg/L for April and July, respectively.

2.8.1.2. Hill-n-Park WWTF

Average data from January 2005 through December 2009 for flow, BOD, TSS, and ammonia for the Hill-n-Park WWTF can be found in Table 2-32.

Table 2-32. Historical Recorded Loading Data for the Hill-n-Park WWTF

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
January 05	0.339	258	192	25	24	-
February 05	0.336	337	192	16	22	-
March 05	0.338	276	248	23	38	-
April 05	0.351	310	276	30	46	-
May 05	0.343	257	416	22	20	-
June 05	0.367	266	272	37	55	-
July 05	0.371	288	400	15	25	-
August 05	0.390	263	208	3	4	-
September 05	0.355	241	284	5	24	-
October 05	0.357	273	392	15	19	-
November 05	0.322	251	300	26	45	-
December 05	0.306	255	212	20	50	-
January 06	0.324	299	344	30	34	-
February 06	0.336	296	316	23	46	-
March 06	0.315	270	356	12	29	-
April 06	0.318	318	372	23	44	-
May 06	0.352	304	256	29	41	-
June 06	0.428	266	296	20	40	-
July 06	0.446	267	228	22	28	-
August 06	0.461	320	356	10	25	-
September 06	0.460	287	288	20	13	-
October 06	0.464	259	228	10	16	-
November 06	0.456	302	276	18	26	-
December 06	0.427	314	240	21	23	-
January 07	0.411	304	208	26	13	-
February 07	0.426	269	248	-	50	-
March 07	0.413	281	200	14	23	-
April 07	0.407	250	276	14	25	-
May 07	0.404	278	224	27	23	-
June 07	0.391	285	252	29	43	-

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
July 07	0.417	251	228	27	60	-
August 07	0.455	228	192	5	17	-
September 07	0.461	283	300	5	13	-
October 07	0.419	263	216	9	16	-
November 07	0.425	272	232	12	26	-
December 07	0.398	280	284	27	30	-
January 08	0.423	235	180	18	12	-
February 08	0.416	269	244	30	44	-
March 08	0.408	263	196	12	24	-
April 08	0.399	275	204	20	41	-
May 08	0.407	272	332	29	35	-
June 08	0.392	259	204	26	74	-
July 08	0.406	270	216	20	25	-
August 08	0.451	205	232	12	24	
September 08	0.459	236	260	7	15	12
October 08	0.432	237	280	9	13	23
November 08	0.434	235	246	15	16	30
December 08	0.433	255	346	18	14	32
January 09	0.435	220	152	27	13	31.9
February 09	0.474	219	194	17	35	34.6
March 09	0.480	250	272	27	61	31.5
April 09	0.477	270	272	29	37	26.6
May 09	0.495	319	226	29	32	32.2
June 09	0.474	357	300	20	38	30.5
July 09	0.524	233	114	29	51	0.3
August 09	0.506	333	272	12	18	1.7
September 09	0.472	248	192	8	22	0.4
October 09	0.487	261	196	9	15	19.0
November 09	0.485	282	248	13	20	25.7
December 09	0.499	233	244	24	44	32.5
2005 Average	0.346	273	283	20	31	-
2006 Average	0.399	292	296	20	31	-
2007 Average	0.419	270	238	18	28	-
2008 Average	0.422	251	245	18	28	24.25

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
2009 Average	0.484	269	224	24	44	32.5
Sum 2005 Ave[1]	0.363	271	309	19	29	-
Sum 2006 Ave[1]	0.411	294	299	21	34	-
Sum 2007 Ave[1]	0.422	263	238	18	30	-
Sum 2008 Ave[1]	0.419	253	241	19	36	-
Sum 2009 Ave[1]	0.491	293	229	21	33	15.2
Wint 2005 Ave[2]	0.333	275	256	21	33	-
Wint 2006 Ave[2]	0.387	290	293	19	29	-
Wint 2007 Ave[2]	0.415	278	231	18	26	-
Wint 2008 Ave[2]	0.424	247	239	17	19	-
Wint 2009 Ave[2]	0.477	244	218	19.5	31	29.2

[1] Average of April through September
[2] Average of January, February, March, October, November, and December

Influent BOD and TSS loading appears to be remaining relatively consistent on a concentration basis with the four year average being 263 mg/L, and 254 mg/L, respectively. Except for June 2005, effluent BOD and TSS are consistently well below the permitted limits of 30 mg/L and 75 mg/L, respectively. The City began sampling for effluent ammonia in the fall of 2008 and currently has not exceeded the April or July permit limit.

2.8.2. Unit Loading Factors

2.8.2.1. Evans WWTF

The following unit flow factors are based on a current sewered population of 12,380, an average day flow of 1.187 MGD at the Evans WWTF and the 2009 average influent loading. For the BOD contribution, an influent loading of 280 mg/L was used resulting in a residential contribution of 0.23 pounds per capita per day. This value is on the high end of the range found in reference literature of 0.17 pounds per capita per day (without garbage grinders) to 0.22 pounds per capita per day (with garbage grinders) recommended by Ten State Standards for design of new systems. Based on an influent TSS loading of 257 mg/L, the estimated per capita suspended solids contribution is 0.21 pounds per capita per day. This value is on the lower end of the range for the Ten States Standards of 0.20 pounds per capita per day (without garbage grinders) to 0.25 pounds per day (with garbage grinders) recommended for new systems. Finally, an influent ammonia loading of 32 mg/L results in an estimated residential per capita contribution of 0.0265 pounds per capita per day. Table 2-33 summarizes per capita wastewater loadings for the Evans WWTF.

Table 2-33. Evans WWTF Unit Loading Factors

Classification	Residential Population	Average Dry-Weather Flow, MGD	Unit Flow, gal/cap/day	BOD Concentration, mg/L	BOD, lb/day	Per Capita BOD, lb/cap/day	TSS Concentration, mg/L	TSS, lb/day	Per Capita TSS, lb/cap/day	Ammonia Concentration, mg/L	Total Ammonia, lb/day[1]	Per Capita Total Ammonia, lb/cap/day
Residential	12,380	1.230	99	280	2,874	0.23	257	2,638	0.21	32	328	0.026

2.8.2.2. Hill-n-Park WWTF

Table 2-34 summarizes per capita wastewater loadings for the Hill-n-Park WWTF. However, as discussed previously, due to the lower unit flow factor for the Hill-n-Park WWTF, the loading is also significantly lower than the Evans WWTF. Therefore, for future projections, the Evans WWTF loading factors was utilized.

Table 2-34. Hill-n-Park WWTF Unit Loading Factors

Classification	Residential Population	Flow, MGD	Unit Flow, gal/cap/day	BOD Concentration, mg/L	BOD, lb/day	Per Capita BOD, lb/cap/day	TSS Concentration, mg/L	TSS, lb/day	Per Capita TSS, lb/cap/day	Ammonia Concentration, mg/L	Total Ammonia, lb/day[1]	Per Capita Total Ammonia, lb/cap/day
Residential	9,633	0.484	50	269	1,086	0.11	224	905	0.09	24	97	0.01

2.8.3. Wastewater Loading Projections

2.8.3.1. Evans WWTF

The per capita unit loadings are combined with population and peaking factors to generate projections of future loading. Table 2-35 summarizes loading projections for 2010, 2020, and 2030 conditions for the Evans WWTF.

Table 2-35. Evans WWTF Existing and Projected Wastewater Loading in Planning Boundary

Criteria	Year 2010	Year 2020	Year 2030
Sewered Population	12,380	12,941	13,502
Average Dry-Weather Flow, MGD	1.230	1.277	1.432
Peak Hour Dry-Weather Flow, MGD[1]	1.968	2.043	2.291
Peak Hour Wet-Weather Flow, MGD[2]	3.284	3.516	4.087
Influent BOD_5, lb/d[3]	2,874	2,984	3,346
Influent TSS, lb/d[3]	2,638	2,739	3,071
Influent Total Ammonia, lb/d[3]	328	341	382

[1]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[2]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).
[3]Calculated by average dry-weather flow multiplied by average influent concentration.

2.8.3.2. Hill-n-Park WWTF

The per capita unit loadings from the Evans WWTF were combined with population and peaking factors for the Hill-n-Park WWTF to generate projections of future flow and loading. Table 2-36 summarizes flow and loading projections for 2010, 2020, and 2030 conditions for the Hill-n-Park WWTF. Finally, an alternative for transferring all flow to the Hill-n-Park WWTF and decommissioning the Evans WWTF is summarized in Table 2-37. Chapters 4 and 5 present the treatment and conveyance facilities required for this alternative.

Table 2-36. Hill-n-Park WWTF Existing and Projected Wastewater Flow and Loading in Planning Boundary (Maintain Current Flow Split Between WWTFs)

Criteria	2010	Year 2020	Year 2030
Population	9,366	20,305	28,483
Average Dry-Weather Flow, MGD	0.491	1.949	2.902
Peak Hour Dry-Weather Flow, MGD[1]	0.687	2.729	4.063
Peak Hour Wet-Weather Flow, MGD[2]	1.660	7.968	13.982
Influent BOD_5, lb/d[3]	1,134	4,554	6,780
Influent TSS, lb/d[3]	1,012	4,180	6,223
Influent Total Ammonia, lb/d[3]	122	520	775

[1]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[2]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).
[3]Calculated by average dry-weather flow multiplied by average influent concentration.

Table 2-37. Hill-n-Park WWTF Existing and Projected Wastewater Flow and Loading in Planning Boundary (Evans WWTF Decommissioned)

Criteria	2010	Year 2020	Year 2030
Population	21,746	33,246	41,985
Average Dry-Weather Flow, MGD[1]	1.721	3.226	4.334
Peak Hour-Dry Weather Flow, MGD[2]	2.655	4.772	6.354
Peak Hour-Wet Weather Flow, MGD[3]	4.944	11.484	18.069
Influent BOD_5, lb/d[4]	4,020	7,537	10,126
Influent TSS, lb/d[4]	3,691	6,918	9,294
Influent Total Ammonia, lb/d[4]	460	861	1,157

[1]Yearly average from 2009 flow data.
[2]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[3]Calculated by adding the average dry-weather flow with peak hour storm I/I flow (I/I factor multiplied by the sewered area).
[4]Calculated by average dry-weather flow multiplied by average influent concentration.

2.8.4. Environmental Components

The existing Evans WWTF and Hill-n-Park WWTF sites are owned by the City and it is anticipated any new improvements to these two facilities will be within the existing property line. If the City pursues constructing a new WWTF on a different site or if State Revolving Fund (SRF) or other Federal funds are pursued, a NEPA review and approval will be required.

2.8.5. Industrial Pretreatment Program

Federal regulations (40 CFR part 403) and the CDPHE CDPS permit require Evans to administer an industrial pretreatment program. Under the pretreatment program, the City establishes allowable pollutant load limits, issues industrial discharge permits, conducts compliance monitoring of industrial dischargers, and enforces the industrial discharge permit conditions when necessary. The discharge permits provide several enforcement actions to ensure compliance with industrial discharge permits including a Notice of Violation (NOV), civil penalties, and other legal remedies. As part of the pretreatment program, the City is responsible for influent and effluent sampling of metals and cyanide from dischargers and the City must submit an annual report describing pretreatment activities over the previous calendar year.

Currently, the City does not have any industrial dischargers. The City will continue to monitor building applications and will implement a sampling program if necessary.

3.0 Regulatory Drivers

3.1. Introduction

The purpose of this chapter is to present a review of the regulatory and effluent management issues which may influence wastewater facilities planning for the City of Evans. Chief among the potential permitting requirements are the following:

- 2013 PEL #200089, which provides the updated effluent discharge standards for the Evans Wastewater Treatment Facility (WWTF) based on a rerating of 1.9 MGD and 4,500 lbs/day of BOD.
- 2009 draft PEL #200090, which set updated effluent discharge standards for the City of Evans Hill-n-Park WWTF.
- Reassessment of the South Platte River as a potential water supply.
- Colorado Department of Health and Environment (CDPHE) Regulations 85 and 31.
- EPA Revised Federal Ambient Water Quality Criteria for Ammonia.

These permit issues govern effluent quality and have implications for the wastewater management choices available to the City.

The City of Evans currently owns and operates two wastewater treatment facilities; the Evans WWTF and the Hill-n-Park WWTF. Both facilities discharge to the main stem of the South Platte River, Middle South Platte River Sub-Basin, with the Hill-n-Park WWTF discharging just downstream of the confluence with the Big Thompson River and the Evans WWTF approximately 4 miles further downstream. Additionally, the La Salle WWTF discharges approximately 2.5 miles downstream of the Hill-n-Park WWTF.

The South Platte River along this stretch is designated for Existing Primary Contact Recreation, Class 2 Warm Water Aquatic Life, agriculture, and water supply. At this time, no water is drawn off for use as potable water downstream of the discharges. However, a number of cities and towns between the City of Evans and the border with Nebraska pull drinking water from wells located adjacent to the river. Conversations with CDPHE indicate they expect a direct potable water draw from the South Platte River within the next 10 years. This would result in significant changes to all discharge permits upstream and would include at a minimum, a requirement for nitrate removal.

3.2. Water Quality Agencies

Surface water quality is monitored, permitted and controlled by the CDPHE Water Quality Control Division (WQCD). 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. The WQCC has delegated responsibility for implementing the water quality regulations to the WQCD of CDPHE. 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 and future uses and maintain the highest water quality possible. The WQCC has established water quality standards to protect and maintain designated uses corresponding to each use classification.

Additionally, the North Front Range Water Quality Planning Association (NFRWQPA) develops and maintains the Clean Water plan to preserve and enhance water quality for both surface and groundwaters in Weld County.

3.3. Receiving Water

The South Platte River originates in the central mountains of Colorado and is fed initially by snowmelt (a map of the watershed is shown on Figure 3-1). As the river winds from the southwest to the northeast through the Metro Front Range, it is dominated by diversions and return flows. Additionally, more than 400,000 acre-feet of water is imported into the drainage through inter-basin transfers. During a portion of irrigation season, the Burlington Ditch diverts flow from the South Platte River resulting in the river section upstream of the Metro Wastewater Reclamation District Robert W. Hite Treatment Facility to run dry. Flow is returned to the river downstream of the treatment facility.

Historically, the South Platte River was a dry stream during certain times of the year. Increased irrigation along the eastern section of the river has caused higher groundwater levels resulting in inflow which has kept the river flowing year round. It is estimated along the length of river which runs through the City of Evans, groundwater seepage accounts for approximately 5 cubic feet per second per mile (cfs/mile).

The closest flow gauging station to the Evans and Hill-n-Park WWTF outfalls is USGS Gage Station 06752500 which is located on the main stem of the South Platte River after the confluence of the Cache le Poudre River near Greeley. An additional USGS gauging station is located on the Cache le Poudre River near the confluence. To determine the flow in the river at the outfalls, the flow from the Cache le Poudre River gauging station is subtracted from the South Platte River gauging station and then the seepage and diversions along the stretch must be taken into account. Based on flow data from October 1996 through September 2007, the 30-day average low flow recurring in a three-year interval (30E3) for this stretch of river is approximately 52 cfs. This value is used in developing effluent limitations based on a chronic standard. For effluent limitations based on an acute standard, the 1E3 or one-day low flow recurring in a three-year interval is used. For this stretch of river, the 1E3 is approximately 15 cfs. CDPHE uses these flow values in determining the effluent limits for the two facilities. The ratio of the Evans WWTF and Hill-n-Park WWTF effluent flow to the 30E3 flow is approximately 35:1 and 42:1, respectively.

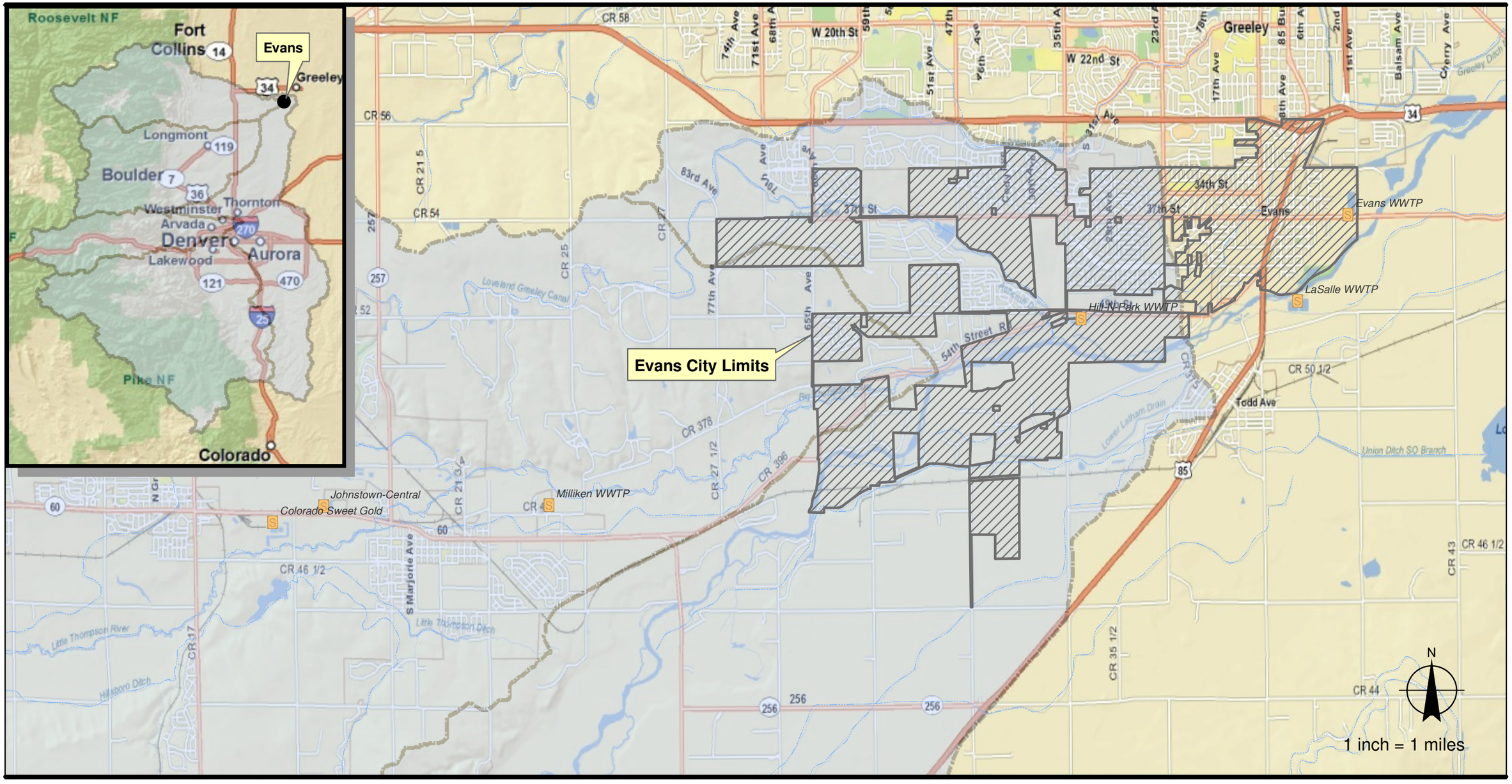
3.3.1. Water Quality

The middle section of the South Platte River is heavily influenced by point and non-point source dischargers. Additionally, during certain times of the year, sections of the river are

dominated by wastewater effluent. The result is an accumulation of nitrogen, phosphorus, and *E. Coli*. A number of water quality studies have been completed on the South Platte River including the Urban Corridor Study completed in 1996 and a U.S. Geological

Survey water quality assessment completed in 1995. The studies showed the middle section of the South Platte River consistently exceeds the *E. Coli* standard set for recreational uses. Groundwater samples taken along the river show high levels of nitrate, pesticides, and herbicides. Further downstream and closer to the Nebraska border, the river shows increasing levels of nitrogen, phosphorus, suspended solids, and salinity. This stretch of river was recently added to the 303(d) list for Selenium.

As mentioned previously, the middle segment of the South Platte River is classified Class 2 Warm Water Aquatic Life , Existing Primary Contact Recreation, Agriculture, and Water Supply. The site specific in-stream water quality standards for this stretch of river are provided in Table 3-1. Metals are generally provided as Table Value Standard (TVS) which are derived from equations that depend on the receiving water hardness or species of fish present. The mean hardness for this stretch of river was calculated through sampling by Water Quality Control Division (WQCD) to be approximately 411 mg/L as $CaCO_3$. However, hardness must be capped at 400 mg/L when determining in-stream metal water quality standards using equations in the TVS.



Legend

- Treatment Plants
- Evans City Limits
- Streams
- Watershed Subbasins (USDA datasource)

Evans Wastewater Utility Plan

Watershed Map

HDR
ONE COMPANY | *Many solutions*

Figure 3-1
May 18th, 2011

Table 3-1. Water Quality Standards for Stream Segment COSPMS01b

Parameter	Value	Units
Physical and Biological		
Dissolved Oxygen (DO)	7	mg/L
pH	6.5-9	s.u.
E. Coli	126	Colonies/100mL
Temperature	30	degC
Inorganic		
Ammonia	TVS	
Un-Ionized Ammonia Acute and Chronic	TVS	
Chlorine Acute	0.019	mg/L
Chlorine Chronic	0.011	mg/L
Free Cyanide Acute	0.005	mg/L
Sulfide Chronic	0.002	mg/L
Boron Chronic	0.75	mg/L
Nitrite	0.5	mg/L
Nitrate	10	mg/L
Chloride Chronic	250	mg/L
Sulfate Chronic	Water Standard	mg/L
Metals		
Total Recoverable Arsenic – acute	50	µg /L
Dissolved Cadmium – acute/chronic	9.2/1.2	µg /L
Total Recoverable Trivalent Chromium – acute	50	µg /L
Dissolved Hexavalent Chromium – acute/chronic	16/11	µg /L
Dissolved Copper – acute/chronic	49.6/29.3	µg /L
Dissolved Iron – chronic	Water Standard	µg /L
Total Recoverable Iron – chronic	1,000	µg /L
Dissolved Lead – acute/chronic	281/10.9	µg /L
Dissolved Manganese – acute/chronic	4,738/2,618	µg /L
Total Mercury – chronic	0.01	µg /L
Dissolved Nickel – acute/chronic	1,513/168	µg /L
Dissolved Selenium – acute/chronic	18.4/4.6	µg /L
Dissolved Silver – acute/chronic	22/3.5	µg /L
Dissolved Zinc – acute/chronic	467/405	µg /L

Water quality samples were collected by the WQCD at location 00022 from 1998 to 2003. This sampling location is approximately 8 miles downstream from the City of Evans. A summary of the WQCD data is provided in Table 3-2.

Table 3-2. Ambient Water Quality for the South Platte River Near the City of Evans (From Hill-n-Park WWTF PELs)

Parameter	No. of Samples	15th Percentile	50th Percentile	85th Percentile	Mean	Chronic Stream Standard	Notes
Temp (ºC)	100	NA	NA	NA	25	30	3
E. coli, (#/100 ml)	22	NA	NA	NA	81	126	1
Hardness (mg/l CaCO3)	53	NA	NA	NA	411	NA	
As, Dis (µg/l)	8	NA	NA	0	NA	NA	4
As, Trec (µg/l)	9	NA	0	NA	NA	50	4
Cd, Dis (µg/l)	55	NA	NA	0	NA	1.2	2
Cr^{+3}, Trec (µg/l)	8	NA	1	0	NA	NA	4
Cr^{+6}, Dis (µg/l)	8	NA	NA	1	NA	11	4
Cu, Dis (µg/l)	24	NA	NA	7	NA	29	2
CN, Free (µg/l)	14	NA	NA	0	NA	NA	4
Fe, Dis (µg/l)	24	NA	NA	170	NA	Water Standard	2
Fe, Trec (µg/l)	53	NA	410	NA	NA	1,000	2
Pb, Dis (µg/l)	55	NA	NA	0	NA	11	2
Mn, Dis (µg/l)	55	NA	NA	71	NA	Water Standard	2
Hg, Tot (µg/l)	12	NA	NA	0	NA	0.01	4
Ni, Dis (µg/l)	6	NA	NA	3.3	NA	168	4
Se, Dis (µg/l)	55	NA	NA	4	NA	4.6	2
Ag, Dis (µg/l)	12	NA	NA	0.11	NA	3.5	4
Zn , Dis (µg/l)	55	NA	NA	31	NA	405	2
NH_3, Tot (mg/l)	47	NA	NA	0.78	NA	Table Value Standard	2

Note 1: The calculated mean is the geometric mean. Note that for summarization purposes, the value of one was used where there was no detectable amount because the geometric mean of one is equal to zero.
Note 2: When sample results were below detection levels, the value of zero was used in accordance with the CO WQCD's standard approach for summarization purposes.
Note 3: Based on limited historic temperature data.
Note 4: Because of a lack of recent data, data from an older POR (1979-2000) is used for these parameters.

Additionally, CDPHE has identified the following key pollutants of concern for this stretch of river:

- Biochemical oxygen demand (BOD_5)

- Total suspended solids (TSS)
- pH
- total residual chlorine
- E. Coli
- Ammonia
- Temperature

Effluent limits for all pollutants except ammonia are calculated based on lowest of the monthly low flows. Ammonia limits are determined through the use of the Ammonia Toxicity Model (AMMTOX) and the 1E3 and 30E3 flows.

3.3.2. Additional Wastewater Dischargers

In addition to the Evans WWTF and Hill-n-Park WWTF, the Town of La Salle located approximately 2.5 miles downstream of the Hill-n-Park WWTF and the Milliken WWTF located approximately 4 miles upstream discharge to this segment of the South Platte River. There are other dischargers to the South Platte river in the general region, however, Milliken and La Salle are included in the CDPHE water quality model for this stretch of river.

3.4. Current Effluent Discharge Conditions

This section describes the current Evans WWTF and Hill-n-Park WWTF discharge permits.

3.4.1. Evans WWTF Colorado Pollutant Discharge Elimination System (CPDES) Permit

The Evans WWTF operates under Colorado Discharge Permit System Permit Number CO-20508 which was renewed in August 2008 and is not scheduled to expire until September 30, 2013. The current permit establishes maximum limits for BOD_5, TSS, fecal coliform bacteria, pH, oil and grease, ammonia and chlorine residual. Minimum removals of BOD and TSS (85 percent) are also identified. The City of Evans is also required to monitor effluent concentrations of total ammonia and cyanide on a monthly basis. Whole effluent toxicity (WET) tests are required for acute exposure. There are currently no limits placed on phosphorus, nitrate, or total nitrogen. The facility is permitted for a design hydraulic capacity of 1.2 million gallons per day (MGD) and a BOD_5 loading of 2,772 pounds per day. A copy of the permit is included in Appendix A of this report.

Table 3-3 below provides a summary of the current discharge requirements.

Table 3-3. Evans WWTF Current Effluent Discharge Limits

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
Flow	MGD	1.2	NA	Report
BOD_5	mg/L	30	45	NA
Total Suspended Solids	mg/L	75	110	NA
E. Coli	#/100mL	986	1972	NA
Total Residual Chlorine	mg/L	0.22	NA	0.17
pH	s.u.	NA	NA	6.5-9
Oil and grease	mg/L	NA	NA	10
Total Ammonia as N	mg/L			
January		Report	NA	Report
February		Report	NA	Report
March		Report	NA	Report
April		44	NA	44
May		Report	NA	Report
June		Report	NA	Report
July		43	NA	Report
August		Report	NA	Report
September		Report	NA	Report
October		Report	NA	Report
November		Report	NA	Report
December		Report	NA	Report
Cyanide, WAD- through Aug 31, 2012	µg /L	Report	NA	NA
Cyanide, WAD- beginning September 1, 2012	µg /L	45	NA	NA
Whole Effluent Toxicity, Acute		NA	NA	LC50>100%

Changing water quality standards will undoubtedly impact permit parameters in the future. Although not the City of Evans' stream section, CDPHE recently completed a total maximum daily load (TMDL) study for the stretch of the South Platte between Chatfield Reservoir and the Burlington Ditch. The TMDL study addresses nitrate concentrations above the drinking water standard of 10 mg/L and set total inorganic nitrogen limits for all point source dischargers.

3.4.1.1. Ammonia Limits

As noted in Table 3-3, the current permit requires the Evans WWTF to meet effluent ammonia limits of 44 mg/L and 43 mg/L for the months of April and July, respectively. The City typically measures the influent and effluent ammonia concentration

approximately ten times per month and averages the values. Data from 2005 through March 2009 show the Evans WWTF has never exceeded the ammonia limit.

Additionally, whole effluent toxicity (WET) testing is also required. WET testing measures the toxicity of the effluent with regard to the growth and reproduction of fathead minnows (Pimephales promelas) and Ceriodaphnia dubia. Toxicity is expressed as a percentage of the effluent that causes a statistical difference in the growth and reproduction end points. Toxicity is typically attributed to effluent ammonia concentrations and may cause a failure of a WET test even though specific ammonia discharge limits are met. An acute WET test is failed whenever an effluent concentration of 100 percent or less is lethal to 50 percent or more of the organisms.

Even though the Evans WWTF has consistently met the discharge ammonia limits, on occasion effluent from the facility has failed the WET test. Based on further research from the testing laboratory ENSR, toxicity was mainly due to high ammonia concentrations.

3.4.2. Hill-n-Park WWTF Effluent Discharge Permit

The Hill-n-Park WWTF operates under Colorado Discharge Permit System Permit Number CO-47287 which was renewed in June 2008 and is not scheduled to expire until July 31, 2013. The current permit establishes maximum limits for BOD_5, TSS, fecal coliform bacteria, pH, ammonia and chlorine residual. Minimum removals of BOD_5 and TSS (85 percent) are also identified. There are currently no limits placed on phosphorus, nitrate, or total nitrogen. The facility is permitted for a design hydraulic capacity of 0.5 million gallons per day (MGD) and a BOD_5 loading of 1,000 pounds per day. A copy of the permit is included in Appendix A of this report.

Table 3-4 below provides a summary of the current discharge requirements.

Table 3-4. Hill-n-Park WWTF Current Effluent Discharge Limits

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
Flow	MGD	0.5	NA	Report
BOD_5	mg/L	30	45	NA
Total Suspended Solids	mg/L	75	110	NA
E. Coli	#/100mL	986	1972	NA
Total Residual Chlorine-Through June 30, 2011	mg/L	NA	NA	0.5
Total Residual Chlorine-Beginning July 1, 2011	mg/L	NA	NA	0.17
pH	s.u.	NA	NA	6.5-9
Total Ammonia as N	mg/L			
January		Report	NA	Report
February		Report	NA	Report
March		Report	NA	Report

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
April		44	NA	44
May		Report	NA	Report
June		Report	NA	Report
July		43	NA	Report
August		Report	NA	Report
September		Report	NA	Report
October		Report	NA	Report
November		Report	NA	Report
December		Report	NA	Report

3.4.2.1. Ammonia Limits

As noted in Table 3-4, the current permit requires the Hill-n-Park WWTF to meet effluent ammonia limits of 44 mg/L and 43 mg/L for the months of April and July, respectively. The City typically measures the effluent ammonia concentration approximately two times per month and averages the values. Data from August 2008 through March 2009 show the Hill-n-Park WWTF has never exceeded the ammonia limit.

Because the facility is permitted for a flow rate below 1 MGD, whole effluent toxicity (WET) testing is not required.

3.5. Future Effluent Discharge Permit Issues

3.5.1. Evans WWTF 2013 PEL #200089

The purpose for updating this wastewater utility plan is to rerate the Evans WWTF from the current maximum month flow of 1.2 MGD to 1.9 MGD and from a BOD load of 2,772 lbs/d to 4,500 lbs/d. As part of the rerating, the Water Quality Control Division (Division) of CDPHE prepared Preliminary Effluent Limits (PELs) for the rehabilitation and expansion of the Evans WWTF. For the purposes of this evaluation, the Evans WWTF will need to meet the requirements of the PELs in the near term. However, it is expected the Revised Federal Ammonia Criteria and the requirements of Regulation 31 will need to be met in the upcoming permit cycles. Therefore, the Evans WWTF effluent design criteria are as described in Section 3.6.2.3. Table 3-5 provides a summary of the PEL discharge requirements.

Table 3-5. Evans WWTF Effluent Discharge Limits Based on PEL #200089

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
Flow	MGD	1.9	NA	Report
BOD_5	mg/L	30	45	NA
Total Suspended Solids	mg/L	75	110	NA
E. Coli	#/100mL	126	252	NA
Total Residual Chlorine	mg/L	0.18	NA	0.27
pH	s.u.	NA	NA	6.5-9
Oil and grease	mg/L	NA	NA	10
Total Ammonia as N	mg/L			
January	mg/L	180	NA	360
February	mg/L	160	NA	440
March	mg/L	44	NA	130
April	mg/L	15	NA	60
May	mg/L	17	NA	65
June	mg/L	14	NA	105
July	mg/L	13	NA	95
August	mg/L	13	NA	100
September	mg/L	12	NA	80
October	mg/L	34	NA	110
November	mg/L	100	NA	320
December	mg/L	170	NA	370
As, TR (µg/l)	ug/L	NA (daily maximum), 0.45 (30-day average)		NA
As, Dis (µg/l)	ug/L	4,912 (daily maximum)		56 (30-day average)
Cd, Dis (µg/l)	ug/L	109 (daily maximum), 12 (30-day average)		NA
Cr+3, TR (µg/l)	ug/L	722 (daily maximum)		NA
Cr+3, Dis (µg/l)	ug/L	NA (daily maximum)		3,869 (30-day average)
Cr+6, Dis (µg/l)	ug/L	231 (daily maximum)		185 (30-day average)
Cu, Dis (µg/l)	ug/L	562 (daily maximum)		377 (30-day average)
CN, Free (µg/l)	ug/L	72 (daily maximum)		15 (30-day average)
Fe, TR (µg/l)	ug/L	NA (daily maximum), 10,043 (30-day average)		NA
Pb, Dis (µg/l)	ug/L	3,466 (daily maximum),		NA

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
		119 (30-day average)		
Mn, Dis (µg/l)	ug/L	63,336 (daily maximum)		42765 (30-day average)
Hg, Tot (µg/l)	ug/L	NA (daily maximum)		0.17 (30-day average)
Ni, Dis (µg/l)	ug/L	19,435 (daily maximum)		2,764 (30-day average)
Se, Dis (µg/l)	ug/L	18.4 (daily maximum), 4.6 (30-day average)		NA
Ag, Dis (µg/l)	ug/L	246 (daily maximum)		18 (30-day average)
Zn, Dis (µg/l)	ug/L	5,325 (daily maximum)		6,292 (30-day average)
Nonylphenol (µg/l)	ug/L	Report (daily maximum), Report (30-day average)		NA

3.5.2. Numeric Nutrient Criteria

The Environmental Protection Agency (EPA) has been working with States to reduce nutrient levels. The emphasis being placed on developing numeric nutrient criteria is specifically tied to the control of "nitrogen and phosphorus pollution". The intent of numeric nutrient criteria is to ensure a level of water quality that will protect the beneficial uses of these water bodies. The presence of nitrogen and phosphorus in surface waters leads to a phenomenon referred to as eutrophication. Eutrophication is characterized by an abundant accumulation of nutrients that support a dense growth of algae and other organisms, the decay of which depletes the shallow waters of oxygen. Nitrogen and phosphorus criteria are set so that they protect streams from the impacts of eutrophication, which include both nuisance algae growth and reduced dissolved oxygen levels which impact fish and aquatic life.

In 2001, EPA published eco-regional criteria which provide recommendations to States for use in establishing their water quality standards consistent with section 303(c) of the Clean Water Act (CWA). The eastern plains of Colorado fall under Ecoregion IV (http://www.epa.gov/waterscience/criteria/nutrient/ecoregions/rivers/rivers_4.pdf). Table 3-6 provides a summary of the limits for total nitrogen and phosphorus developed by EPA for the Eco-regions.

Table 3-6. EPA Eco-region Criteria for Rivers and Streams

Eco-region	Total Nitrogen (mg/L)	Total Phosphorus (mg/L)
I: Willamette and Central Valley	0.66	0.055
II: Western Forested Mountains	0.12	0.010
III: Xeric West	0.38	0.022
IV: Great Plains Grass and Shrublands	*0.56*	*0.023*
V: South Central Cultivated Great Plains	0.88	0.067
VI: Corn Belt and Northern Great Plains	2.18	0.076
VII: Mostly Glaciated Dairy Region	0.54	0.033
VIII: Nutrient Poor, Largely Glaciated Upper Midwest and Northeast	0.38	0.010
IX: Southeastern Temperature Forested Plains and Hills	0.69	0.037
X: Texas-Louisiana Coastal and Mississippi Alluvial Plains	0.57	0.060
XI: The Central and Eastern Forested Uplands	0.31	0.010
XII: Southeastern Coastal Plain	0.90	0.040
XIII: Southern Florida Coastal Plain	1.14	0.015
XIV: Eastern Coastal Plain	0.71	0.031

EPA further breaks down Eco-region IV in aggregate levels. Northeastern Colorado is included in Aggregate Level III-South Central Cultivated Great Plains V. Table 3-7 provides the 25th percentile aggregate nutrient reference condition for rivers and streams located near the Evans and Hill-n-Park WWTFs.

Table 3-7. Rivers and Streams in Nutrient Eco-region V (25th Percentile)

Nutrient Parameter	Aggregate Nutrient Eco-region V Reference Conditions
Total Phosphorus (mg/L)	0.067
Total Nitrogen (mg/L)	0.88
Chlorophyll a (ug/L)	3
Turbidity (FTU)	7.83

CDPHE chose to develop its own nutrient quality rules, which were adopted by the Water Quality Control Commission (WQCC) in May 2012. The State adopted a phased approach to establishing numeric nutrient standards throughout Colorado. These regulations set total phosphorus (TP) and total inorganic nitrogen (TIN) permit limits for the largest wastewater dischargers and set phosphorus and nitrogen interim values for both lakes and reservoirs and rivers and streams.

The first phase is implementation of CDPHE Regulation 85 (Regulation 85), which set interim effluent standards for TP and total inorganic nitrogen (TIN) of 1.0 mg-P/L of TP and 15 mg-N/L, respectively. Regulation 85 sets permit limits for new dischargers and existing dischargers (excluding existing dischargers ≤2 MGD). The permit limits will be incorporated into permits at the next renewal and compliance schedules will be used to allow the permittee time to come into compliance with these limits.

The second phase of the State's roll-out of nutrient quality criteria is implementation of Regulation 31. This regulation sets interim annual median in-stream nutrient quality values, and the rule was approved with the presumption that these values would not be established as definitive water quality criteria until 2022 except in very limited cases. The in-stream TP and TN values for warm water streams are 0.17 mg-P/L and 2.01 mg-N/L, respectively.

Table 3-8 summarizes the regulatory requirements of Regulations 85 and 31. One advantage to Colorado's phased approach to implementing nutrient rules is that it provides time for both water quality assessment and treatment technology initiatives to be developed, proven, and rolled out into the marketplace.

Table 3-8. Nutrient-Related Effluent Standards (Regulation 85) and In-Stream Nutrient Values (Regulation 31)

Parameter	Regulation 85 (Effluent Standards)	Regulation 31 (Warm Water In-Stream Values)	Regulation 31 (Cold Water In-Stream Values)
TP (mg-P/L)	1 (1)	0.17 (2)	0.11 (2)
TIN (mg-N/L)	15 (1)	N/A	N/A
TN (mg-N/L)	N/A	2.01 (3)	1.25 (3)
Attached Algae Chlorophyll *a*, milligrams per square meter (mg/m^2)	N/A	150	150

(1) Running Annual Median: The median of all samples taken in the most recent 12 calendar months.
(2) To be determined.
(3) Determined as the sum of nitrate as N, nitrite as N, and ammonia as N.

The total nitrogen concentration presented in Regulation 31 is lower than treatment technologies are capable of achieving if applied to "end of pipe". Attainment of effluent limits based on predicted in-stream numeric criteria at the point of discharge may be possible for phosphorus, but technology does not exist today that can reduce nitrogen to the required level. Table 3-10 provides a summary of the effluent limits that can be met for nitrogen and phosphorus for different available technologies. To meet the lower end of the nitrogen limits consistently, advanced technologies such as reverse osmosis (RO) must be implemented.

CDPHE has acknowledged the fact that the nitrogen limit may not be attainable and has discussed the inclusion of variances based on "limits of technology". Based on the current discussions, limit of technology for nitrogen removal is considered enhanced nutrient removal (see Table 3-10). For phosphorus removal, an additional filtration step

will be required. However, facilities which do not currently meet the limit of technology standard are expected to be required to implement these improvements.

Table 3-10. Numeric Nutrient Criteria and Limits of Wastewater Technology

Parameter	Typical In Stream Nutrient Criteria (mg/L)	Typical Municipal Raw Wastewater (mg/L)	Standard Secondary Treatment (mg/L)	Typical Advanced Treatment Nutrient Removal (mg/L)	Enhanced Nutrient Removal (mg/L)	Limits of Treatment Technology (mg/L)
Total Phosphorus	0.020 to 0.050	4 to 8	4 to 6	1	0.25 to 0.50	0.05 to 0.07
Total Nitrogen	0.3 to 0.6	25 to 35	20 to 30	10	4 to 6	3 to 4

3.5.3. Revised Federal Ammonia Criteria

The US EPA recently released a revised freshwater ammonia criteria first issued for public comment in December 2009. The revised criteria update the current 1999 ammonia criteria included in most state water quality standards and lead to more stringent effluent ammonia limits in NPDES permits for many wastewater treatment facilities.

The 1999 criteria are based on ammonia toxicity to fish and whether or not sensitive fish species are present in the water body. Revised ammonia criteria are being proposed for the protection of certain species of freshwater unionid mussels and snails, which recent studies have shown to be more sensitive to ammonia toxicity than fish. The revised criteria include a bifurcated criteria approach, with different sets of acute and chronic values depending on mussels being present or absent in the water body.

Table 3-11 provides a comparison of the 1999 criteria with what EPA originally published as a draft revision in 2009 and with the final 2013 criteria.

Table 3-11. Summary Comparison of Ammonia Criteria at pH 7 and Temperature 20°C, and pH 8 and Temperature 25°C

Criterion Duration	1999 Criteria Based on Juvenile Salmonids		2009 Draft Revised Criteria Mussels Present		Final 2013 Criteria Single Criteria Mussels Present	
	pH 8, Temp=25°C	pH 7, Temp=20°C	pH 8, Temp=25°C	pH 7, Temp=20°C	pH 8, Temp=25°C	pH 7, Temp=20°C
Acute (mg/L)	5.6	24	2.9	19	2.6	17
Chronic (mg/L)	1.2	4.5	0.26	0.91	0.56	1.9

States are now in the process of adopting and incorporating the revised criteria into their state water quality standards. The state adoption process typically is a two to three year process initiated during the states' triennial review of their water quality standards. It is expected revised criteria will start to appear in state water quality standards in mid- 2015 through 2016, with potential revisions wastewater facilities' NPDES discharge permits to follow.

3.5.4. Temperature Numeric Standards

In 2007 the WQCD implemented basin-wide interim numeric temperature standards for each drainage within Colorado. Segment specific temperature standards were implemented in 2008 for the Upper and Lower Colorado River Basins and in 2009 for the South Platte River Basin. In May 2009, the WQCD developed a Draft Final Action 2010 303(d) Listing Methodology which describes how WQCD will regulate temperature based on chronic standards, acute temperature standards, and excursions from temperature standards. Regulation 38 – Classifications and Numeric Standards for South Platte River Basin, Laramie River Basin, Republican River Basin, Smokey Hill River Basin establishes classification and numeric standards for the previously mentioned river basins and tributaries. The regulation describes the temperature numeric standard as:

"Temperature shall maintain a normal pattern of diurnal and seasonal fluctuations with no abrupt changes and shall have not increased in temperature of a magnitude, rate, and duration deemed deleterious to the resident aquatic life."

Based on Regulation 38, the South Platte River is classified as a Warm Water Aquatic Life 2 which results in a maximum weekly average temperature of 27.5 degC for the months of March through November and 13.7 degC for the months of December through February. Additionally, the regulation provides a daily maximum temperature of 28.6 degC and 14.3 degC, respectively for the same months listed previously. Based on effluent data, neither the Evans WWTF or Hill-n-Park WWTF has violated the temperature standard.

3.5.5. Economic Hardship

The EPA realizes that technology does not exist today that can remove nitrogen to the above-stated concentrations, and that attempting to achieve numeric standards such as those listed above at the point of discharge will be very costly. EPA is considering allowing variances in treatment standards based on a community's financial ability to make improvements to their facilities to perform nutrient removal. It is proposed that, if a community's average monthly residential user fee for sewer service is at or above 1 or 2 percent of the median household income (MHI), the community will not be expected to make improvements to their WWTFs to perform nutrient removal. If a community's average monthly residential user fee for sewer service is below the goal percentage of the MHI, the community will have to make improvements to perform nutrient removal until the average residential user fee reaches the 1 to 2 percent goal, or they are performing nutrient removal to the limits of technology. Table 3-12 summarizes MHI and sewer use fee data for Evans.

Table 3-12. Evans MHI and Rate Summary

MHI (2005-2007 US Census, American Community Survey Estimate)	Target Monthly Sewer User Fee at 1 Percent of MHI	Target Monthly Sewer User Fee at 2 Percent of MHI	Current Average Evans Sewer User Fee (2010)
$49,225	$41.02	$82.04	$9.89

If a variance is allowed, it would result in a temporary water quality standard for the discharger, for which a permit would be formulated. Variances would be considered temporary, even though they have a 20-year life, and they would be re-visited every five years in conjunction with permit renewals.

3.5.6. Effluent Discharge Mixing Zone Rules

A mixing zone is defined as a limited area of a surface water body, or a portion of an aquifer, where initial dilution of a discharge occurs, where water quality changes may occur, and where certain water quality standards may be exceeded. A mixing zone exists for both the Evans WWTF and Hill-n-Park WWTF discharges. Narrative water quality standards, standards for harmful substances, numeric acute and chronic standards for aquatic life, and standards for human health must not be exceeded beyond the boundaries of the mixing zone. Within the mixing zone, acute standards for aquatic life may not be violated, unless it will not threaten or impair existing beneficial uses. Potential contaminants of concern that may be present at acute concentrations (toxic to aquatic organisms in a short time) in mixing zones include residual chlorine and un-ionized ammonia.

If the mixing ratio is less than 20:1 dilution or the facility is categorized as "major" with a flow rate higher than 1 MGD, mixing zone considerations pursuant to the *Colorado Mixing Zone Implementation Guidance* are applied. The Evans WWTF has a mixing ratio greater than 20:1 but is categorized as "major" and therefore mixing zone considerations were applied to the discharge permit. While a mixing zone study has not been completed for the Evans WWTF, CDPHE has discussed including the requirement as part of the next permit cycle.

The Hill-n-Park WWTF has a mixing ratio greater than 20:1 and is not currently classified as a "major" facility; therefore the facility is exempt from further mixing zone considerations.

3.5.7. Expansion of Hill-n-Park WWTF

As discussed in Chapter 2, the majority of growth in the Evans area is located near the Hill-n-Park WWTF. The existing Hill-n-Park WWTF is currently at greater than 90 percent of the rated capacity. To accommodate future growth, capacity of the treatment facility must be increased. In 2007, the City of Evans began discussions with CDPHE concerning the expansion of the Hill-n-Park WWTF capacity from 0.5 MGD to 0.99 MGD. The Water Quality Control Division of CDPHE prepared Preliminary Effluent Limits (PELs) for the proposed expansion in January 2009. The PELs developed were based on updated flow data for the South Platte River, effluent limits for pollutants of concern as established in the *Regulations for Effluent Limitations* (Regulation No. 62), and water quality-based effluent limits necessary for the protection of the water quality in the receiving water. Table 3-13 below provides a summary of the updated effluent discharge limits.

Table 3-13. Preliminary Effluent Limits for Hill-n-Park WWTF at 0.99 MGD Flow

Parameter	Units	30-Day Average	7-Day Average	Maximum or Range
Flow	MGD	0.99	NA	Report
BOD_5	mg/L	30	45	NA
Total Suspended Solids	mg/L	75	110	NA
E. Coli	#/100mL	827	1,654	NA
Total Residual Chlorine	mg/L	NA	NA	0.144
pH	s.u.	NA	NA	6.5-9
Total Ammonia as N	mg/L			
January		Report	NA	Report
February		Report	NA	Report
March		Report	NA	Report
April		Report	20	20
May		Report	26	26
June		41	NA	Report
July		24	NA	Report
August		29	NA	Report
September		40	NA	Report
October		Report	NA	Report
November		Report	NA	Report
December		Report	NA	Report

The PELs require significantly lower effluent ammonia concentrations for the months of April and May compared to the current discharge permit and new requirements for June through September. As a result, operational control of the Hill-n-Park WWTF during these months must be improved to bring effluent ammonia concentrations down from the monthly average of 12 - 32 mg/L that were reported between September 2008 and March 2009. The following sections discuss nutrient control approaches to achieve the proposed lower limits.

3.6. Nutrient Control Approaches

As discussed in the previous sections, the City of Evans will likely be required to reduce the discharge of nutrients to the South Platte River. The two general approaches to accomplishing this goal are to divert the treated effluent to land application, or to implement additional advanced methods of wastewater treatment. Both WWTFs currently employ aerated lagoons for treatment of BOD, TSS, and ammonia.

More advanced wastewater treatment, particularly biological nutrient removal, has been widely used in the United States, Canada, Europe, and South Africa for more than three decades. Advanced wastewater treatment is a reliable and effective means for complying with water quality based discharge standards. Natural treatment techniques, such as land application and constructed wetlands, have also been successfully implemented in other locations. This approach is most often used in small communities where adequate land is available. The following sections describe alternatives for nutrient control.

3.6.1. Wastewater Reuse

Wastewater reuse may be considered as a means for avoiding the environmental consequences of additional water supply development or to divert the discharge wastewater effluent from surface water to land. The concept is to substitute reclaimed effluent for potable water in cases where the quality of the supply source is not a critical concern. The most common reuse approach is to use wastewater effluent for irrigation purposes.

3.6.1.1. Colorado State Regulations

Colorado State regulations for water reuse are presented in Colorado Department of Public Health and Environment Water Quality Control Commission, Regulation No. 84, Reclaimed Water Control Regulation. Wastewater used for irrigation on restricted access sites is required to receive secondary treatment and disinfection. For sites with unrestricted access, the wastewater must receive secondary treatment, filtration, and disinfection.

3.6.1.2. City of Evans Reuse Opportunities

The City of Evans receives potable water from transbasin and ditch shares including the following:

- Colorado Big Thompson Water Project
- Wind Gap Project
- Agriculture water from the GL Irrigation Company (Greeley-Loveland system, Loveland-Greeley Reservoir, Seven Lakes System)

Water from the Colorado Big Thompson Project (C-BT Project) is transbasin and does not have a return flow obligation. However, the water can not be reused as it is required for additional downstream uses. The water available from the C-BT Project varies from year to year and is dependent upon the available water supplies, the needs of the shareholders, and the annual quota. Typically, the quota has averaged 0.7 acre-ft/unit. The City of Evans transfers the water from the C-BT Project to the City of Greeley for treatment. Greeley treats the water at the Bellevue Water Treatment Plant and then transmits it back to Evans for distribution.

The City of Evans has a second transbasin water source in the Windy Gap Project. Evans currently owns 5 units at 76 acre-ft/unit. The yield is expected to increase to 100 acre-ft/unit once the firming project is completed. This water does not have a return flow obligation and can be used for water reuse.

Evans owns approximately 3,926 acre-ft of ditch water which has been converted over to potable water rights. The Greeley-Loveland Irrigation Company delivers water to the City of Evans through three sources: 1) Greeley-Loveland System, 2) Seven Lakes System, and 3) Loveland-Greeley Reservoir.

A summary of the Evans potable water sources and available water for reuse is provided in Table 3-14.

Table 3-14. City of Evans Potable Water Rights

Water Right Name or Source	Number of Shares or Units Owned	Avg Annual Yield (ac-ft/unit)	Avg Annual Yield (ac-ft)	Water Reuse Available?	Return Flow Obligation % of Annual Yield	Available Volume for Water Reuse (ac-ft)
Ditch Shares						
Greeley-Loveland System (GL Irrigation Company)	147.9	10.5	1,553	Yes	61%	605
Seven Lakes (GL Irrigation Company)	44.8	15	672	Yes	55%	302
Loveland-Greeley Res (GL Irrigation Company)	19.8	33.1	654	Yes	43%	372
Total Native Water			*2,879*			*1,279*
Transbasin Shares						
Colorado Big Thompson	3,460	0.7[1]	2,422	No	0%	0
Windy Gap Project	5	76	380	Yes	0%	380
Total Transbasin			*2,802*			*380*
Total Potable Water			*5,681*			*1,659*

[1] Colorado Big Thompson yield is variable from year to year. Assumed to be 0.7 acre-ft/unit for this analysis.
Return flow obligation for GL Irrigation Company water is based on Evans Change Case 96CW958.
Return flow obligation for Godfrey Ditch is based upon engineering analysis and 2008-2009 SWSP.
Wind Gap yield is expected to increase to 100 acre-ft/unit after firming project completed.
Greeley-Loveland return flow obligation based upon storage component.

Based on the water which is not obligated for return flow or is designated for use downstream, approximately 1,659 acre-ft of treated effluent from the Evans WWTF and Hill-n-Park WWTF could be reused or approximately 1.7 MG per day. Based on the current effluent flow rate for each treatment facility, this results in the entire effluent available for reuse.

3.6.2. Advanced Wastewater Treatment

Advanced treatment techniques using biological, chemical, and physical processes are available for removing nutrients from wastewater. These processes have become much more prevalent in recent years; in some areas of the country nutrient removal is relatively

common. The main benefits of advanced treatment are that performance is controllable and predictable, operational strategies can be employed to modify performance, weather impacts on operation are minimized, and compact physical sites can be utilized. Substantial investments in land purchases are reduced when compared to land treatment. The primary disadvantages are that a potentially greater investment in capital facilities is required, along with increased operational costs. This impact applies to not only liquid stream processes, but also to solids stream facilities, which will receive greater quantities of solids. Advanced treatment is the most adaptable alternative for winter nutrient removal, should it become necessary in order to satisfy surface water discharge requirements.

3.6.2.1. Nitrogen Removal

Nitrogen in untreated domestic wastewater is normally in the form of ammonia (NH_3) with typical concentrations ranging from 30 to 45 mg/L. Nitrogen can be removed through biological means. Typically, an aeration basin for this process includes zones with no oxygen (anaerobic) and chemically combined oxygen (anoxic), as well as conventional aerated zones. Ammonia is removed from wastewater by a microbial process which sequentially oxidizes ammonia to nitrite (NO_2-) and then to nitrate (NO_3-). The process requires excess amounts of dissolved oxygen and alkalinity and is termed nitrification. For a majority of wastewater treatment systems, the conversion of ammonia to nitrate is all that is required. However, if a permit requires the removal of nitrate from the effluent, nitrate must be denitrified and reduced to nitrogen gas which is then stripped from the water and discharged to atmosphere. Denitrification can only be accomplished in anoxic conditions. During these conditions, bacteria metabolize biodegradable substrate using nitrate as the electron acceptor. This process also returns alkalinity to the system.

To meet an effluent nitrate concentration of less than 10 mg/L consistently, the treatment system must provide for greater process control than a typical aerated lagoon treatment system. While lagoon systems normally provide for a large treatment volume resulting in low organic and nutrient concentrations, these systems do not allow for adequate process control to meet low effluent nutrient limits.

The anaerobic zones in the oxidation system encourage the growth of specific bacteria that consume high quantities of phosphorus. With this approach, phosphates can be removed from the liquid stream.

3.6.2.2. Phosphorus Removal

Phosphorus removal can be accomplished through chemical addition and precipitation or biological uptake. Either method removes phosphorus from the liquid stream and transfers to the biosolids. Because biosolids from both facilities are beneficially used through land application, there is a potential for the phosphorus to leach and harmfully impact the watershed. Researchers have found that conventional biosolids release approximately half as much phosphorus to plants as chemical fertilizers. Regulatory agencies are now revisiting how biosolids are governed which may result in a limit on the total amount of phosphorus applied to land via biosolids.

3.6.2.3. Treatment Process Planning

Treatment facility planning involves consideration of a number of factors including receiving water requirements, selection of the treatment process configuration, utilization of existing facilities, correction of existing system deficiencies, biosolids management, plant site layout, operation and maintenance, budget management, implementation schedule, permitting, and user charges. This multi-dimensional planning process often results in competing priorities. For example, all facility needs may not be able to be satisfied with the resources currently available. Future performance requirements may not be clear, resulting in the need for flexibility to adapt to alternative scenarios. Addressing these issues and choosing a course of action is a large part of the facility planning process.

The fact that future discharge permit levels are not yet included in the current permit should not impede planning for possible future conditions. In fact, it may be detrimental to utilities to avoid planning for future discharge standards until permit levels are defined. This approach limits the vision of utility planners and managers, and may prevent them from developing effective strategies for process modifications, arranging for adequate plant site space, developing compatible biosolids management plans, preparing for financial impacts, and understanding the full implications of offers of sewer service to new developments. This is a situation faced by many Front Range communities.

Base Treatment Scenario. Current discharge permit requirements for the Evans WWTF and Hill-n-Park WWTF require oxidized and disinfected effluent. Limits on ammonia nitrogen are limited to April and July and have been consistently met.

Biological Nutrient Removal Scenario. Regulation 31 receiving water discharge permit limitations will be key to governing the sizing of biological nutrient removal processes and their performance requirements. Total nitrogen and phosphorus limits for discharge to the South Platte River will be included in permits within the next 10 years. Additionally, the Revised Federal Ammonia Criteria is expected to be included in permits within the next 5 years. Planning for nitrogen and phosphorus removal and more restrictive discharge permit conditions to accommodate increased loads will be prudent. In fact, planning for nutrient removal requirements is well advised since regulatory agencies are continually investigating receiving water conditions and a single potable water draw downstream of the City of Evans will result in a nitrate limit.

For the purposes of this Wastewater Utility Plan Update, the following effluent nutrient limits will be assumed to be in the Evans WWTF and Hill-n-Park WWTF discharge permits.

- Near term – next 5 years
  - Ammonia < 1 mg/L
  - BOD < 30 mg/L
  - TSS < 30 mg/L
  - Regulation 85 nutrient monitoring
- Long term – outside 10 years
  - Ammonia <1 mg/L
  - BOD < 25 mg/L
  - TSS < 25 mg/L

- TN < 5 mg/L
- TP < 0.5 mg/L

The goals of the phase 1 improvements are as follows:

- Increase flow and load capacity
- Remove ammonia below 1 mg/L
- Plan for expansion to meet future total nitrogen and phosphorus effluent limits

3.7. Biosolids Regulations

The Clean Water Act Amendments of 1987 required the US Environmental Protection Agency (EPA) to develop new regulations pertaining to biosolids. In February, 1993, EPA published 40 CFR Part 503 (e.g. Part 503). The Part 503 Rule is a complex, risk-based assessment of potential environmental effects of pollutants that may be present in biosolids (EPA, 1995). These guidelines regulate pollutant and pathogen concentrations as well as vector attraction reduction (VAR). The guideline defines biosolids as Class A or Class B, depending on the potential level of pathogens. Class A biosolids must meet strict pathogen standards and can be used with no restrictions, while Class B biosolids must meet less stringent pathogen requirements, with application restricted to crops with limited human and animal exposure. Biosolids in both classes must meet VAR requirements.

Additionally, CDPHE regulates biosolids through the Water Quality Control Commission Biosolids Regulation No. 64 the purpose of which is to establish requirements, prohibitions, standards and concentration limits on the use of biosolids as a fertilizer and/or organic soil.

Two approaches to meeting the metals limits are allowed: 1) a maximum concentration must be met, or 2) a maximum cumulative amount of metals added to the soil via biosolids must be met. Biosolids meeting the requirements by method one are called pollutant concentration (PC) biosolids and limits are shown in Table 3-15. If biosolids metals meet these concentrations, no record keeping of cumulative loading to solids is required. If PC biosolids also meet Class A pathogen reduction standards, they are considered exceptional quality (EQ), and may be distributed to the public.

Table 3-15. Pollutant Concentration (PC) Biosolids (Table 3 of 40 CFR 503.13)

Pollutant	Cumulative Pollutant Loading Rates (mg/kg dry weight basis)	Ceiling Concentration Limits (mg/kg dry weight basis)
Arsenic (As)	41	75
Cadmium (Cd)	39	85
Copper (Cu)	1,500	4,300
Lead (Pb)	300	840
Mercury (Hg)	17	57
Molybdenum (Mo)	-	75
Nickel (Ni)	420	420
Selenium (Se)	100	100
Zinc (Zn)	2,800	7,500

ND=Non-detect

An effective industrial pretreatment program is the key to complying with the metal limits, as industrial inputs into the collection system are the primary source of metals. The EPA is currently considering adding 15 additional chemicals to be regulated. Those include acetone, anthracene, barium, beryllium, carbon disulfide, 4-chloroaniline, diazinon, fluoranthene, manganese, methyl ethyl ketone, nitrate, nitrite, phenol, pyrene, and silver.

Management practices required by the Part 503 regulations include providing buffer zones around wells, surface water, and property boundaries; nutrient management including only applying biosolids at or below agronomic rates; not causing any adverse impact to threatened or endangered species; and not applying biosolids to flooded, frozen or snow-covered land where runoff is possible. This section also includes requirements on monitoring and reporting.

Pathogen and Vector Attraction Criteria – The Part 503 regulations require biosolids that are land applied to meet two distinct criteria in addition to metal limits: pathogen destruction and VAR. As mentioned previously, EPA divides biosolids into two categories in terms of the potential level of pathogens, Class A and Class B. Part 503 describes several methods for meeting both Class A and Class B criteria. Evans currently meets Class B criteria through pathogen certification, vector/attraction certification, and management practice certification.

Future Regulations – The National Academy of Sciences (NAS) recently completed an assessment of the science that supports the Part 503 Rule, and concluded that more research is required to update that science (NRC,2002). NAS concerns included chemical pollutants and pathogens not considered in the risk assessment of the Part 503 Rule, as well as their synergistic effects. As a result of NAS recommendations, EPA may begin a review of the Part 503 Rule every five years. It is possible that Class B biosolids may not be acceptable for land application or other beneficial reuse in the future due to regulatory changes and concerns with the phosphorus uptake, or that the pathogen levels for Class B biosolids may be lowered. EPA has reaffirmed its endorsement of biosolids land application in a letter to state biosolids coordinators and will ultimately decide whether Class B biosolids will be viable in the future.

Table 3-16: Wastewater Biosolids Quality

Constituent	EPA Limit for Class B Biosolids mg/kg dry weight [1]	EPA Limit for Giveaway Use On Lawn or Garden, mg/kg dry weight [2]
Arsenic	75	41
Cadmium	85	39
Chromium	3,000	1,200
Copper	4,300	1,500
Lead	840	300
Mercury	57	17
Molybdenum	75	18
Nickel	420	420
Selenium	100	36
Zinc	7,500	2,800

[1] Ceiling concentration from EPA Sludge Regulations (Table 1, Section 503.13)
[2] Pollutant concentration from EPA Sludge Regulations (Table 3, Section 503.13)
ND= Non-detect

4.0 Wastewater Treatment Facilities Evaluation & Alternatives Development

4.1. Introduction

The City of Evans is served by a gravity sanitary sewer system and two wastewater treatment facilities- the Evans Wastewater Treatment Facility (WWTF) and the Hill-n-Park WWTF. Demands on these wastewater facilities have expanded especially at the Hill-n-Park WWTF due to development. Additionally, higher levels of wastewater treatment will be required to protect receiving water quality in the future. Finally, flow and loading to the Evans WWTF is greater than 80 percent of permitted capacity which triggers a requirement by CDPHE to develop a schedule for facility expansion. This wastewater utility plan update identifies solutions for expanding the facility and provides a capital improvements plan for achieving.

To assess the magnitude of these impacts and identify future needs, the capacity of the existing wastewater treatment facilities must first be confirmed. The first objective of this chapter is therefore, to evaluate the performance and estimate the capacity of the wastewater treatment facilities.

The second objective of this chapter is the development and evaluation of alternatives for improvement and expansion of both the Evans WWTF and the Hill-n-Park WWTF. This section considers plans to address existing system deficiencies, as well as future needs. This is done through the development and evaluation of alternatives. As described in Chapter 2, baseline wastewater conditions and established projections of future wastewater flows and loadings were documented. Chapter 3 developed the level of performance required of treatment facilities. Effluent discharge standards will become more stringent in the future and alternatives for improvement and expansion should be considered in that light.

4.1.1. Evaluation Goals

An engineering evaluation of the wastewater collection system and treatment facilities is generally recommended to identify the limitations of the existing facilities and identify approaches to correction, as well as define the capacity of the wastewater collection and treatment system. Current development pressures in the Evans area may exceed the available capacity of the existing collection system and treatment facilities. Limited funds are available for construction of additional facilities to accommodate new growth and development in the Evans area, thus before making capital investments, it is worthwhile to fully define the capacity available in the existing system and develop a plan to maximize its use. The first priority for expanding plant capacity is alleviation of capacity restrictions. This process of identifying available capacity is often referred to as a rating study. In Chapter 5, the collection system is evaluated.

4.2. Existing Facilities Evaluation – Evans WWTF

The Evans WWTF accomplishes secondary treatment of domestic and commercial wastewater collected within the sewer service area. Secondary treatment consists of the removal of 85 percent or more of the incoming organic and solids materials, in contrast with primary treatment, which generally removes only about 60 percent of the solids and 30 percent of the organic matter. Additionally, the facility is currently required to remove ammonia during the months of April and July.

The WWTF consists of a three lagoon system rated for a 30-day average daily flow of 1.2 million gallons per day (MGD) and an organic loading of 2,772 pounds of biochemical oxygen demand (BOD) per day. The facility was originally constructed in the early 1960's. Recent improvements to the plant included outfall improvements in 1997, a raw water irrigation system in 2000, and a new grit removal unit in 2003. The basic processes for the Evans WWTF lagoon system are as follows;

- Influent 9 IN Parshall Flume with flow transmitter.
- Headworks Building with one 0.25 IN opening cylindrical fine screen and screenings screw washer/compactor/conveyor and dumpster.
- One vortex grit chamber with grit pump, concentrator and conveyor.
- Influent pumping with three 1,540 gpm, 40 hp, centrifugal pumps.
- One 12.30 million gallon partially mixed Aeration Lagoon No. 1, equipped with (8) 15 HP Aqua-Aerobics aerators.
- One 8.25 million gallon partially mixed Aeration Lagoon No. 2, equipped with (5) 15 HP Aqua-Aerobics aerators.
- One 1.63 million gallon Polishing Cell.
- One 29,000 gallon Chlorine Contact Channel with liquid sodium hypochlorite disinfection and sodium bisulfite dechlorination.
- Effluent 6 IN Parshall Flume with flow transmitter.
- 24 IN outfall pipe to the South Platte River.

A process flow schematic for the current facility is shown on Figure 4-1 and a photo of the site is shown on Figure 4-2. For the purposes of discussion, facilities can be separated into liquid stream and solids stream processes. Liquid stream facilities are those designated for wastewater treatment, while solids stream processes are those dedicated for handling the sludge (biosolids) produced during wastewater treatment.

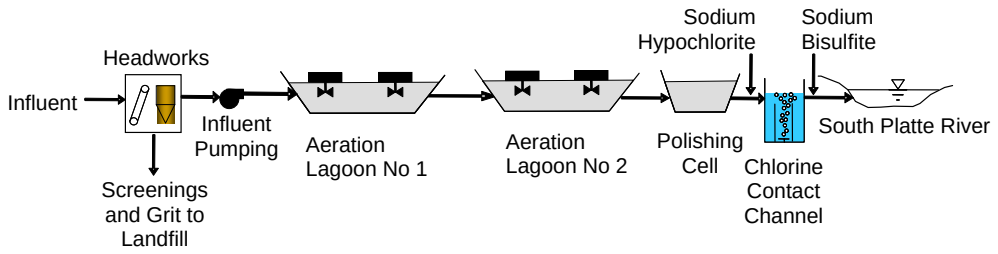


Figure 4-1. Evans WWTF Process Flow Schematic

Figure 4-2. Evans WWTF Site

4.2.1. Liquid Stream

Raw wastewater is conveyed by gravity from the wastewater collection system to the Headworks Building through a 24 IN influent pipeline. Flow is measured in a 9 IN Parshall flume before entering the Headworks Building. After the flume, the flow travels through a channel in to the cylindrical screen and then through a vortex grit separator before entering the lift station wet well. There are isolation gates to the grit separator that allow for bypass of the grit removal system; however this is only done when maintenance is required. Three pumps, rated for a total of 150 percent of max flow, convey the liquid from the Headworks Building to Aeration Lagoon No. 1.

Aeration Lagoon No. 1 is a 12.30 MG pond with a surface area of approximately 289,238 SQ FT and depth of 6 FT. The basin is mixed by eight (8) 15 HP mixers resulting in approximately 9 HP per MG of volume. The lagoon has a detention time of 10.25 days at the design flow of 1.2 MGD. Flow then travels by gravity through a 12 IN pipe to Aeration Lagoon No. 2.

Aeration Lagoon No. 2 is an 8.25 MG pond with a surface area of approximately 199,069 SQ FT and depth of 6 FT. It is operated similar to Lagoon No. 1, but is designed for a detention time of 6.9 days at design flow. Five (5) 15 HP mixers are installed in this lagoon resulting in approximately 9 HP per MG of volume. After traveling through Aeration Lagoon No. 2, the flow is gravity fed through a 12 IN pipe to the final lagoon,

the Polishing Cell. Additionally, a 12 IN bypass pipe is available to take effluent from Aeration Lagoon No. 2 directly to the Chlorine Contact Channel. The bypass is used during periods of high algae growth in the Polishing Cell.

The Polishing Cell has a volume of 1.63 MG with a surface area of 34,483 SQ FT and a depth of 6 FT. This final pond is not aerated and is designed for final clarification of the liquid stream prior to disinfection and discharge to the South Platte River. After a short detention time of 1.4 days at design flow, the treated effluent continues to the Chlorine Contact Channel.

The treated wastewater is then disinfected with liquid sodium hypochlorite, after which flow passes through the 29,000 gallon Chlorine Contact Channel to provide the required 30 minute detention time. The Chlorine Contact Channel is approximately 31 FT long and 12 FT wide with seven serpentine passes and provides a detention time of approximately 34 minutes at design flow of 1.2 MGD. Given the storage volume available upstream, peak flows do not impact the Chlorine Contact Channel detention time. Following chlorination, liquid sodium bisulfite is used for dechlorination. Effluent then leaves the contact basin in a 12 IN pipe and flows to the 6 IN Parshall flume. Discharge out of the flume to the intersection with 37th Street is through a 12 IN pipe. Effluent then enters a diversion box, makes a 90 degree turn and heads to the South Platte River in a 24 IN pipe. Effluent is discharged to the river through a valve which prevents flows from backing up into the plant. Figure 4-3 provides a site plan of the facility.

4.2.2. Solids Stream

Solid stream facilities are limited to the material removed from Headworks Facility and biosolids stabilization in the lagoons. Since the facility utilizes a basic aerated lagoon design, there is no recycling of sludge and the waste solids are stored and digested in the lagoon system. Inert solids removed by the cylindrical screen or grit separator are collected in a trash bin and sent to a landfill for disposal. Sludge is removed from the lagoons approximately every 5 to 10 years, at which time it is dredged and disposed of in accordance with local, State and Federal regulations.

4.2.3. Capacity Evaluation

4.2.3.1. Current Conditions

Currently, the Evans WWTF receives an average monthly influent flow of approximately 1.2 MGD. Summer average for 2012 was 1.3 MGD and winter average for 2012 was 1.15 MGD. Organic and solids loadings are relatively consistent throughout the year. Based on data collected from the year 2005 through the present, the average influent flow has generally increased. Summer and winter flows vary by approximately 5 percent over the four year period indicating the collection system is minimally influenced by infiltration and inflow (I/I). As noted by the 2012 flows and loads, the Evans WWTF is above the rated capacity. As such, this utility plan update will be used to select technologies for increasing capacity.

The organic loading is measured by an indirect analytical procedure called 5-day biochemical oxygen demand (BOD_5). The influent BOD_5 loading concentrations show

an increase over time, indicating a trend toward the use of lower flow fixtures and the removal of I/I from the system. Total suspended solids (TSS) concentrations also show a slight increase over time, with a large spike in 2005 which was the result of scheduled cleaning of the sewer system. However, the concentrations of both parameters generally remain in the range of 250-300 mg/L, which reflects normal levels of BOD_5 and TSS.

4.2.3.1. Overall Facility Performance

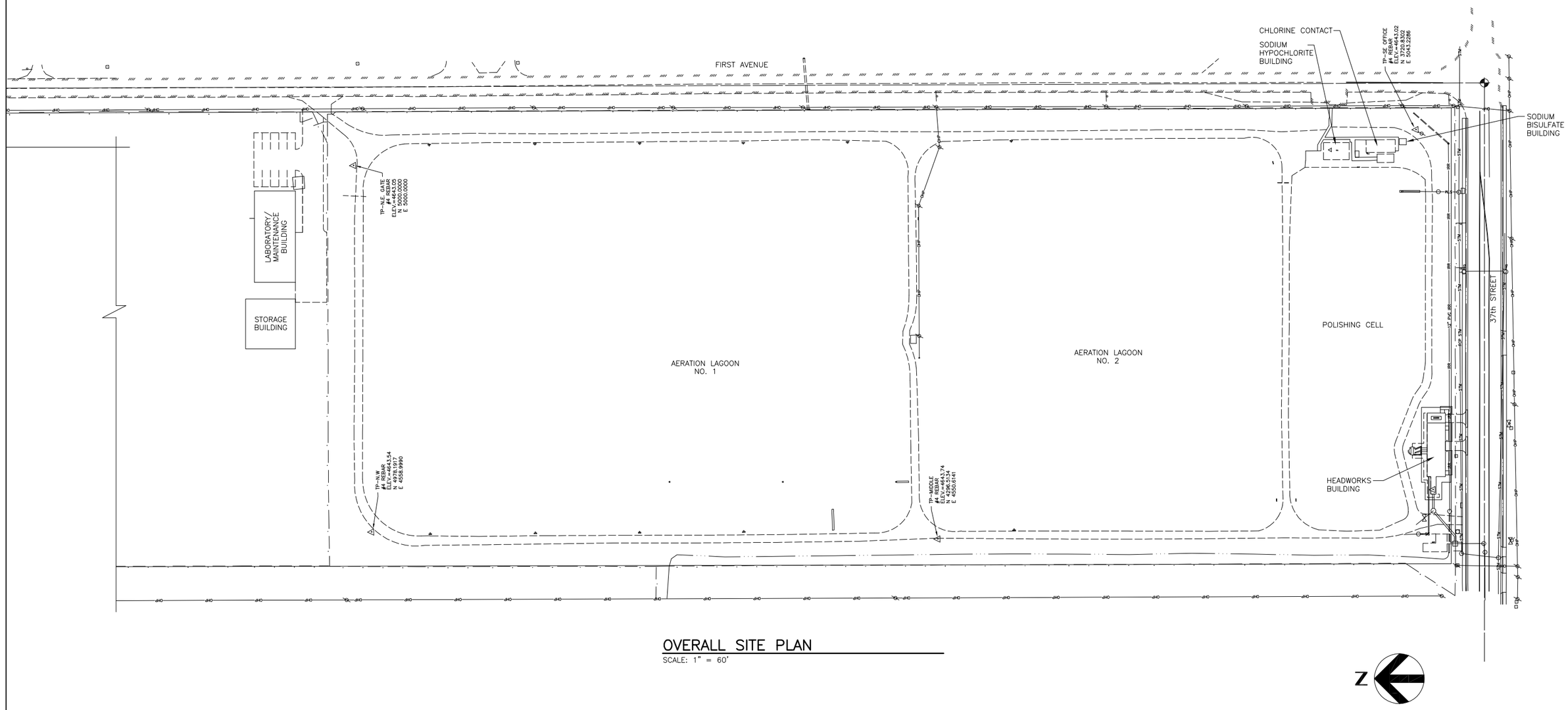
Aeration Lagoon No. 1 is designed to reduce the organic loading by 2,342 pounds of BOD per day with a detention time of 10.25 days at peak flow. Aeration Lagoon No. 2 is designed to reduce the organic loading by an additional 430 pounds of BOD per day with a detention time of 6.9 days at peak flow.

The performance of the Evans WWTF is provided in Table 2-6 of Chapter 2, summarized herein as Table 4-1. Treatment plant performance is generally better during the summer months (July-September). Higher summer temperatures are expected to be a factor in the better performance achieved as a result of higher biological activity in the lagoons.

Table 4-1 shows the current plant performance is generally in compliance with the current Colorado Department of Public Health and Environment (CDPHE) permit requirements. The permit dictates maximum effluent BOD_5 and TSS concentrations of 45 mg/L and 110 mg/L or less, respectively, with at least 85 percent removal of BOD_5 for 7-day average. Thirty-day average concentration limits are 30 mg/L and 75 mg/L for BOD_5 and TSS, respectively. As noted in Table 4-1, both BOD and TSS near the thirty-day average limits at times. Some of this is attributed to algae growth the last two ponds.

Influent BOD, TSS, and ammonia influent loading appears to be remaining relatively consistent on a concentration basis with the four year average being 281 mg/L, 312 mg/L, and 30 mg/L respectively.

Effluent BOD is consistently below permit limits; however, in 2008 the effluent TSS concentration was at or above the 7-day permitted limit of 110 mg/L for five of the twelve months. Effluent ammonia has always been below the required permit limit of 44 mg/L and 43 mg/L for April and July, respectively. However, there have been a few instances where the facility has not been able to meet permit for the whole effluent toxicity (WET) test.



OVERALL SITE PLAN

SCALE: 1" = 60'

HDR

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
–	–	–

PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

EVANS WWTF
Overall Existing Facility Site Plan

0 1" 2"

FILENAME	FIG_4-03.DWG
SCALE	AS NOTED

SHEET

FIG. 4-3

C:\pwworking\omaid\0437481\FIG_4-03.DWG, Plot, 10/7/2010 2:52:03 PM, joknight

Table 4-1. Average Yearly Recorded Flow and Loading Data for the Evans WWTF

Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Influent NH_3-N, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
2005 Average	0.999	279	467	30	24	47	16
2006 Average	0.995	285	320	32	25	55	19
2007 Average	1.048	277	255	27	26	45	19
2008 Average	1.132	265	252	28	28	68	23
2009 Average	1.187	279	257	32	24	43.5	22
2010 Average	1.21	273	250	31	25	41	20
2011 Average	1.20	272	259	32	27	44	20
Sum 2005 Ave[1]	1.032	283	521	30	26	49	8
Sum 2006 Ave[1]	1.022	282	331	31	25	63	9
Sum 2007 Ave[1]	1.080	257	220	24	26	49	9
Sum 2008 Ave[1]	1.153	254	240	27	25	68	17
Sum 2010 Ave[1]	1.31	236	221	28	22	31	14
Sum 2011 Ave[1]	1.29	243	244	30	27	40	12
Wint 2005 Ave[2]	0.966	275	417	33	22	45	24
Wint 2006 Ave[2]	0.968	288	309	34	25	48	28
Wint 2007 Ave[2]	1.015	297	290	31	26	41	29
Wint 2008 Ave[2]	1.111	275	262	29	30	68	26
Wint 2010 Ave[2]	1.13	302	270	30	26	46	25
Wint 2011 Ave[2]	1.13	294	274	33	27	43	26

[1] Average of April through September
[2] Average of January, February, March, October, November, and December

4.2.3.2. Nutrient Removal Performance

Nutrient discharges, particularly phosphorus and nitrogen, are a growing concern to water bodies throughout the United States. Excessive nutrient levels have been found to stimulate the growth of nuisance algae. Treated municipal wastewater discharges have been found to be a significant contributor to elevating nutrient levels in the South Platte River.

Nitrogen in untreated domestic wastewater is normally in the form of ammonia (Table 4-1 summarizes the effluent ammonia concentrations from the Evans WWTF during the period of 2005 to 2011). The Evans WWTF current CDPHE permit (No. CO-20508) does not include effluent requirements for phosphorus, nitrate, or total nitrogen levels, however, as discussed in Chapter 3, it is expected that limits will be enforced in the next

10 to 15 years. The permit does limit ammonia during the months of April and July. The new permit will become even more stringent with regard to ammonia.

Based upon the above, Evans has shown they have been able to consistently meet current nutrient permit limits. However, even though the Evans WWTF has consistently met the discharge ammonia limits, on occasion effluent from the facility has failed the WET test (see Chapter 3) for reasons mainly due to high ammonia concentrations.

4.2.3.3. Liquid Stream

In assessing the capacity of a unit treatment process, one must first determine the critical design or performance criterion for the process. The critical design criterion for some processes such as pretreatment, influent pumping, and the plant outfall is the ability to convey peak flows without causing uncontrolled bypasses. The key performance criterion for other unit processes is typically the ability to provide sufficient treatment performance to ensure reliable compliance with effluent quality requirements.

Preliminary Treatment

Preliminary treatment facilities are located at the Headworks Building and include the influent pipeline, flow measurement, screening, grit removal, and influent pumping. The performance of each process is described below.

Influent Pipeline - The 24 IN influent pipeline is constructed at a slope of 0.08 percent resulting in a full pipe capacity of approximately 4.14 MGD (assuming free discharge downstream). Based on the available capacity, the influent pipeline is adequately sized to handle flows for the next 20 years.

Influent Screen - Typical capacity criterion for the influent screen and screenings handling is its ability to handle peak hour hydraulic flows without surcharging the collection system upstream. The cylindrical screen unit capacity is reported as 4.8 MGD which is adequately sized for the next 20 years. The existing screen has a 0.25 IN screen spacing and performs well to eliminate debris from entering the lagoon system. The screenings are collected in a dumpster and periodically disposed of to a landfill. A bypass channel with manually raked screen is provided as a backup. The screening equipment was installed in 1998 and was designed for a 20 year life. The City will need to plan for funding replacement of the equipment in the next 10 years. Additionally, to provide for adequate redundancy, a second screen should be installed in the bypass channel.

Grit Removal– The existing vortex grit chamber was installed in 2003. The grit chamber has a diameter of 8 FT and depth of 12 FT with provision for a bypass of the basin. The system has a peak flow capacity of approximately 3 MGD. One 15 hp vacuum primed grit removal pump delivers grit to a single grit concentrator unit. Separated grit is then conveyed through a grit dewatering screw conveyor to a dumpster. Unit process redundancy is not provided at the vortex grit chamber and grit cyclone. The grit chamber has adequate capacity to meet flows for the next 10 years. To meet future flow requirements and provide redundancy, the City should plan on constructing a second unit in the next 10 to 15 years.

Flow Measurement - The existing 9 IN Parshall flume is rated for a capacity up to 3.69 MGD. The typical recommended maximum flow for a free flowing flume of this size is

approximately 5.75 MGD. Higher flows can be accommodated; however, careful consideration to downstream submergence conditions must be used. The existing Parshall flume and ultrasonic flow rate transmitter is adequately sized to meet flows for the next 20 years.

Influent Pumping – The screened and de-gritted influent is then pumped from the Headworks Building to Aeration Lagoon No. 1 by three 40 HP centrifugal pumps (2 duty and 1 standby). Each pump has a capacity of 1,540 gpm resulting in a firm capacity with one pump in standby of 4.4 MGD. HDR has not performed a detailed hydraulic analysis of the existing equipment, however, based on conversations with City staff; it appears the system is adequately sized to meet flows for the next 20 years. The pumps discharge to a common 12 IN PVC forcemain which brings the flow to Aeration Lagoon No. 1. Based on recent inspections, City staff have determined the 12 IN forcemain is partially plugged. Either repair or replacement of the plugged section is required. The pumps were installed in 1998 and are more than 10 years old. Again, equipment life expectancy is approximately 20 years, therefore, the City should plan on replacing the pumps within the next 10 years.

Secondary Treatment

The core of the secondary treatment system is the two Aerated Lagoons and Polishing Cell. Organic matter in the wastewater is consumed by suspended growth microorganisms in the aerated lagoons. These microorganisms and inert solids settle in the Aerated Lagoons and Polishing Cell where the solids are digested and then periodically dredged and removed from the system. An objective of the secondary treatment system is to maintain a large inventory of organic-consuming microorganisms, which are properly conditioned to settle in the Polishing Cell.

Aerated Lagoons - The aerated lagoons are the key process that establishes the organic capacity of the treatment facility. The design of the Evans WWTF was based on a total capacity of 2,772 pounds of BOD_5 per day in the two lagoon system, under average conditions (from CDPHE CDPS No. CO-20508). Aeration Lagoon No. 1 has eight (8) 15-HP mixers which were installed in the spring of 2009. Aeration Lagoon No. 2 has five (5) 15-HP mixers which were installed in 1976 and are nearing the end of their useful life. The lagoons have approximately 9 HP/MG of mixing power, thus defining the lagoons as partially-mixed (approximately 30 HP/MG is required for complete mix conditions).

A partially mixed lagoon system is designed only to provide the necessary oxygen levels for BOD removal and not to provide complete solids suspension. Because lagoon treatment systems do not have a solids recycle, the solids retention time is the same as the hydraulic retention time. At the average day rated capacity of the Evans WWTF and a working depth of 6 FT, Aerated Lagoons No. 1 and No. 2 have an SRT/HRT of approximately 10 days and 6.8 days, respectively.

The performance of partially mixed aerated lagoons is directly linked to the following parameters:

- Number of cells
- Temperature

- Lagoon configuration
- Mixing and aeration

Research has shown that increasing the number of cells or creating a circuitous flow decreases the overall required detention time by creating a system more similar to a plug flow reactor. Ideally, two or more cells are provided. For the Evans WWTF, two aerated and one polishing cells are provided.

Due to the large surface area of lagoon treatment systems, temperature of the water during the winter months can be significantly lower than a comparable conventional activated sludge treatment system. Temperature affects the reaction rate with lower temperatures resulting in decreased BOD removal. Organisms which nitrify ammonia to nitrate are also inhibited by water temperatures below 15 degC. Evans WWTF data shows winter lagoon effluent temperatures near 0 degC. Temperatures this low significantly inhibit microorganism activity.

To maximize BOD removal, lagoons are typically designed as rectangular basins with a length to width ratio of 3:1 or 4:1. Aeration Lagoon No. 1 has a length of 600 FT and width of 467 FT resulting in a length to width ratio of 1.3:1. Aeration Lagoon No. 2 is similar with a length to width ratio of 1.2:1 (length of 467 FT and width of 390 FT). Finally, the Polishing Cell provides a length to width ratio of approximately 3.8:1 (length of 467 FT and width of 120 FT).

The amount of mixing provided for partially mixed lagoons is dependent upon the amount of oxygen required for BOD removal. Typically, a minimum of 2.5 lbs of oxygen is required for each pound of BOD removed. This results in a mixing energy of approximately 5 to 10 HP per million gallons of lagoon volume. To create a complete mix system, more than three times the input horsepower is required. For Aeration Lagoon No. 1, approximately 7,200 lbs oxygen per day is provided resulting in 2.6 lbs of oxygen per pound of BOD removed at a depth of 6 FT.

Based on the influent BOD loading over the past 3 years, the existing lagoon treatment system is either at or above the rated capacity. As such, improvements are needed. Additionally, there are currently no limits for total phosphorus or nitrogen. Should full nitrification be required, as a result of more stringent nutrient removal limits in future permits, the existing lagoons would not be able to meet the discharge limits. Installation of additional mixing in the lagoons would increase the capacity for nitrification, but at lower temperatures the lagoon retention time still limits the amount of nitrification possible.

Polishing Cell - The volume of the polishing cell is approximately 1.63 MG and provides 1.35 days of detention at the rated capacity. This cell is designed to settle and remove any remaining solids prior to discharge to the disinfection system. However, due to the relatively shallow depth (6 FT) and retention time, algae tend to grow in this cell resulting in periods of high effluent TSS concentration. During these periods, the Polishing Cell is bypassed and effluent in Aeration Lagoon No. 2 is directly discharged to the Chlorine Contact Channel. Additionally, the anaerobic conditions created in the Polishing Cell due to the settled solids can cause a release of ammonia making it very difficult to meet effluent ammonia limits during the summer months.

Disinfection

The purpose of disinfection is to reduce pathogenic (disease causing) organisms in the wastewater. The current Evans WWTF discharge permit limits the concentration of *E.Coli* which can be discharged to no more than 986 per 100 mL on a 30 day average and 1,972 per 100 mL on a 7 day average. Disinfection is accomplished by adding liquid sodium hypochlorite to the wastewater and allowing time for contact prior to discharge. Chlorine is a toxic material and, at moderate concentrations, is hazardous to aquatic organisms. As such, sodium bisulfite is added after the Chlorine Contact Channel to reduce the concentration of chlorine in the effluent. The discharge permit currently requires the City to limit the effluent residual chlorine concentration to no more than 0.22 mg/L on an average monthly basis and no more than 0.17 mg/L on any given day. Control of the disinfection and dechlorination systems are manually adjusted based on effluent flow rate.

Chlorination Feed - The Evans WWTF originally disinfected through the use of gaseous chlorine but converted to liquid sodium hypochlorite in 2006. Liquid sodium hypochlorite is dosed to either the effluent from the Polishing Cell or the bypass from Aeration Lagoon No. 2. Sodium hypochlorite is stored in a 500 gallon tank located in a building near the channel. The solution is pumped from the tank by peristaltic pumps to the injector located approximately 5 FT upstream of the Chlorine Contact Channel. The facility typically uses between 200 and 300 gallons of sodium hypochlorite per month. To maintain a consistent working volume, the storage tank is normally refilled twice a month.

Chlorine Contact Channel - The Chlorine Contact Channel is simply a serpentine tank that allows time for the chlorine to contact with the wastewater and inactivate pathogenic organisms. Design criteria dictate the Chlorine Contact Channel provide 30 minutes of detention at the peak effluent flow rate. Based on this criterion, the chlorine contact channel, with an effective volume of 29,000 gallons has a peak flow capacity of 1.39 MGD. To meet future flow rates, the capacity of the disinfection system must be increased.

Dechlorination Feed – Effluent from the Chlorine Contact Channels must be dechlorinated prior to discharge to the South Platte River. Dechlorination is accomplished through injection of liquid sodium bisulfite at the exit of the contact channel. The liquid is delivered and stored in 55 gallon drums and metered using peristaltic pumps. Approximately 100 to 200 gallons of sodium bisulfite is used on a monthly basis.

Effluent Outfall

Final effluent from the Chlorine Contact Channel is discharged through a 12 IN pipe to a 6 IN Parshall Flume. Effluent then flows by gravity south to 37th Street through a 12 IN pipe where it enters a diversion structure and then flows east to the South Platte River through a 24 IN pipe. Effluent is discharged to the South Platte River through a tide-flex valve. The outfall does not include a diffuser to promote mixing of the wastewater effluent with the river. Based on a high water elevation in the South Platte River of approximately 4,636 FT above sea level, the outfall has a maximum capacity of

approximately 2 MGD. Based on a peak hour future flow projection of approximately 4 MGD, the outfall does not have adequate capacity. To meet the future flow condition, the following outfall improvements are required.

- Replace existing 6 IN Parshall Flume with a closed pipe flow meter.
- Construct a parallel 24 IN outfall pipe.

4.2.4. Solids Removal and Disposal

During biological wastewater treatment, microorganisms grow using the wastewater organics as food. In addition, inert solids accumulate in the treatment process. Excess microorganisms and inert solids settle and create a layer of sludge in the bottom of the aeration lagoons and polishing cell. These solids must be periodically removed from the system through dredging, dewatering, and land application.

The City of Evans WWTF makes use of a lagoon treatment system. As such, biosolids accumulate and are stabilized in Aeration Lagoons No. 1 and 2 and the final Polishing Cell. Biosolids must be removed from the treatment system every 5 to 7 years. This is accomplished by dredging and hauling the material by tanker truck to a land application site. The biosolids removed typically have a concentration between 2 and 11 percent solids depending on the removal location and depth in the lagoon.

The City contracts this work to a licensed hauler and applier with the removal and disposal fee based on the quantity of dry tons. The licensed hauler/applier is responsible for the testing and required permitting for land application. Prior to application, samples are sent to a licensed laboratory for testing of the following:

- Total solids
- pH
- Total ammonia
- Nitrate nitrogen
- Total phosphorus
- Conductivity
- Organic matter
- Available phosphorus
- Metals (arsenic, cadmium, chromium, copper, lead, mercury, molybdenum, nickel, selenium, zinc)

Additionally, the biosolids are tested for fecal coliform to confirm the material meets the CDPHE requirements for Class B biosolids which requires seven individual samples be taken at the time of removal with a geometric mean of the density of fecal coliform less than 2,000,000 Most Probable Number (MPN) per gram of total solids (dry weight basis). Finally, the biosolids must also meet biosolids stability vector attraction reduction requirements through one of ten methods provided in CDPHE Regulation No. 64.12.C.(1). For the City of Evans WWTF, this requirement is normally met by applying it to the surface and incorporating it into the soil within six hours.

The Evans WWTF was last dredged in 2004. Approximately 5,785,000 gallons, at an average 3 percent solids, were removed from the lagoons resulting in 720 dry tons of

biosolids. The biosolids were applied to a non-food producing corn crop on approximately 160 acres. Table 4-2 provides a summary of the biosolids parameters. Testing indicated the biosolids parameters were all below the permit limits noted in Table 3-11. Given the service area is comprised of primarily residential customers, and no significant industrial dischargers are connected to the system, it would not be expected the Evans biosolids would exceed any of the metals limits.

Table 4-2. Evans WWTF - 2004 Biosolids Sampling Results

Parameter	Units	Evans 2004 Sample
Average Percent Solids	%	3
Total Nitrogen	lbs N/DT	11.67
Total Phosphorus	lbs P/DT	24.36
Potassium	lbs K/DT	4.7
Fecal Coliform (Geometric Mean-9 samples)	#/gram	10,306
Arsenic (As)	mg/kg	4
Cadmium (Cd)	mg/kg	6.2
Chromium (Cr)	mg/kg	46.7
Copper (Cu)	mg/kg	427.3
Lead (Pb)	mg/kg	80.9
Mercury (Hg)	mg/kg	0.2
Molybdenum (Mo)	mg/kg	7.7
Nickel (Ni)	mg/kg	13.8
Selenium (Se)	mg/kg	6.7
Zinc (Zn)	mg/kg	669.6

4.2.5. Support Facilities

4.2.5.1. Standby Power

Backup power is provided to the Evans WWTF Headworks Building by a 125 kW diesel standby generator. The main purpose of the generator is to maintain power to the influent pumps. The other treatment processes do not currently have backup power.

4.2.5.2. Operations/Maintenance/Lab Facility

The operations/maintenance/lab facility was constructed in 1998 and consists of offices, a lab, and a dedicated maintenance area. Control, operation, and maintenance for both the Evans WWTF and the Hill-n-Park WWTF is conducted at this facility.

4.2.5.3. Odor Control

The Evans WWTF makes use of a partially mixed lagoon treatment system. Currently, no odor control is provided. The City has received a minimal number of odor complaints

over the past 5 years. The proximity to housing and commercial sites in the future may increase the potential for odor impacts on the community. Odor control for the proposed improvements will be evaluated during predesign.

4.2.5.4. Air Quality Permits

The Evans WWTF does not have an air emissions permit. As noted previously, the facility makes use of partially mixed lagoon treatment system. Normal setback for this type of treatment system is 1,000 feet. The facility is situated away from housing and commercial development. The onsite emergency generator is in compliance with air emission regulations for the date of its manufacture.

4.2.5.5. Stormwater Management

There is no formal stormwater management plan for the facility as the site is presently exempted. The Evans WWTF site is flat with the exception of the treatment lagoon dikes. Stormwater near the treatment system is directed to the lagoons. The existing drainage pattern will be maintained for proposed upgrade alternatives at the Evans WWTF site. If a formal site drainage plan and permit are required when expansion is implemented, one will be developed.

4.2.6. Summary

The Evans WWTF capacity rating evaluation is summarized in Table 4-3.

Table 4-3: Evans WWTF Capacity Rating Evaluation Summary

Unit Process	Capacity Criteria	Rated Capacity	Year 2020 Ave/Peak Anticipated Flows or Loading	Year 2030 Ave/Peak Anticipated Flows or Loading	Comment
Influent Pipeline	Hydraulic	4.14 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future.
Influent Flow Measurement	Hydraulic	3.69 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity through 2020.
Influent Screen	Hydraulic	4.8 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future. Need to add second unit for redundancy
Grit Removal	Hydraulic	4.0 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future, however, a second unit is required for redundancy.
Influent Pumping	Hydraulic	4.4 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for future. Firm capacity=4.4 MGD.

Unit Process	Capacity Criteria	Rated Capacity	Year 2020 Ave/Peak Anticipated Flows or Loading	Year 2030 Ave/Peak Anticipated Flows or Loading	Comment
Treatment Lagoons	Solids Loading (BOD/TSS)	2,772 lb/day	2,984 lb/day	3,346 lb/day	Improvements required to meet future flow and loading requirements.
Chlorine Disinfection Equipment and Channel	Hydraulic	1.4 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Adequate capacity for current conditions. System should be expanded or replaced to meet future flow requirements based on proposed treatment alternatives.
Outfall	Hydraulic	2 MGD	1.277 MGD/ 3.516 MGD	1.432 MGD/ 4.087 MGD	Parallel outfall should be constructed to meet future flow requirements.

4.3. Existing Facilities Evaluation – Hill-n-Park WWTF

Similar to the Evans WWTF, the Hill-n-Park WWTF also employs a lagoon treatment system and is currently permitted for a 30-day average flow of 0.5 MGD and an organic loading of 1,000 pounds of BOD per day. The following sections provide a description of the Hill-n-Park WWTF. Due to the similarity of the treatment systems, only specific information related to the Hill-n-Park WWTF is described.

The facility was originally constructed as part of the Hill-n-Park residential development in the late 1970's. Recent improvements to the plant include the following:

- General improvements in 2000 including construction of headworks building, chlorine contact channel, installation of two new mixers in Aeration Lagoon No. 1 and three new mixers in Aeration Lagoon No. 2, and construction of Polishing Cell.
- Lining of Aeration Lagoon No. 2 and Polishing Cell in 2001.
- Headworks building improvements in 2003 including new step screen, screenings washer compactor, new influent flow building, and new non-potable wash water system.
- Construction of new outfall in 2004.
- Installation of two additional mixers in Aeration Lagoon No. 1 in 2009.

The basic processes for the Hill-n-Park WWTF lagoon system is as follows:

- Influent 9 IN Parshall flume with flow transmitter.

- Influent Flow Building with one 6 MM opening step screen and screenings screw washer/compactor/conveyor and dumpster, manual bar screen with 1/2 IN opening.
- One 9.07 million gallon partially mixed Aeration Lagoon No. 1 with (6) 15 HP Aqua-Aerobics aerators.
- One 5.06 million gallon partially mixed Aeration Lagoon No. 2 with (3) 15 HP Aqua-Aerobics aerators.
- One 2.03 million gallon Polishing Cell.
- One 40,000 gallon Chlorine Contact Basin.
- Liquid sodium hypochlorite disinfection and sodium bisulfite dechlorination systems.
- Effluent 9 IN Parshall flume with flow transmitter.
- 21 IN outfall to South Platte River.

A process flow schematic for the current facility is shown on Figure 4-4 and a photo of the site is shown on Figure 4-5.

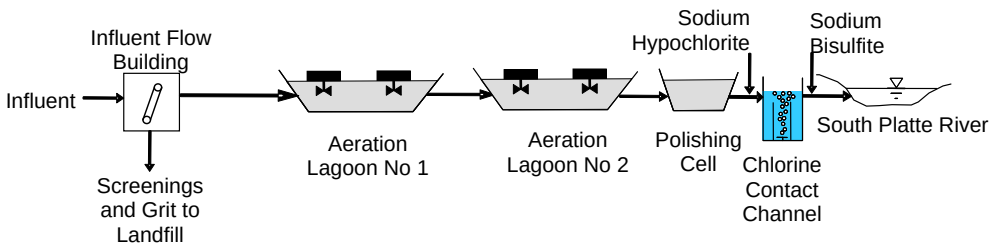


Figure 4-4. Hill-n-Park WWTF Process Flow Schematic

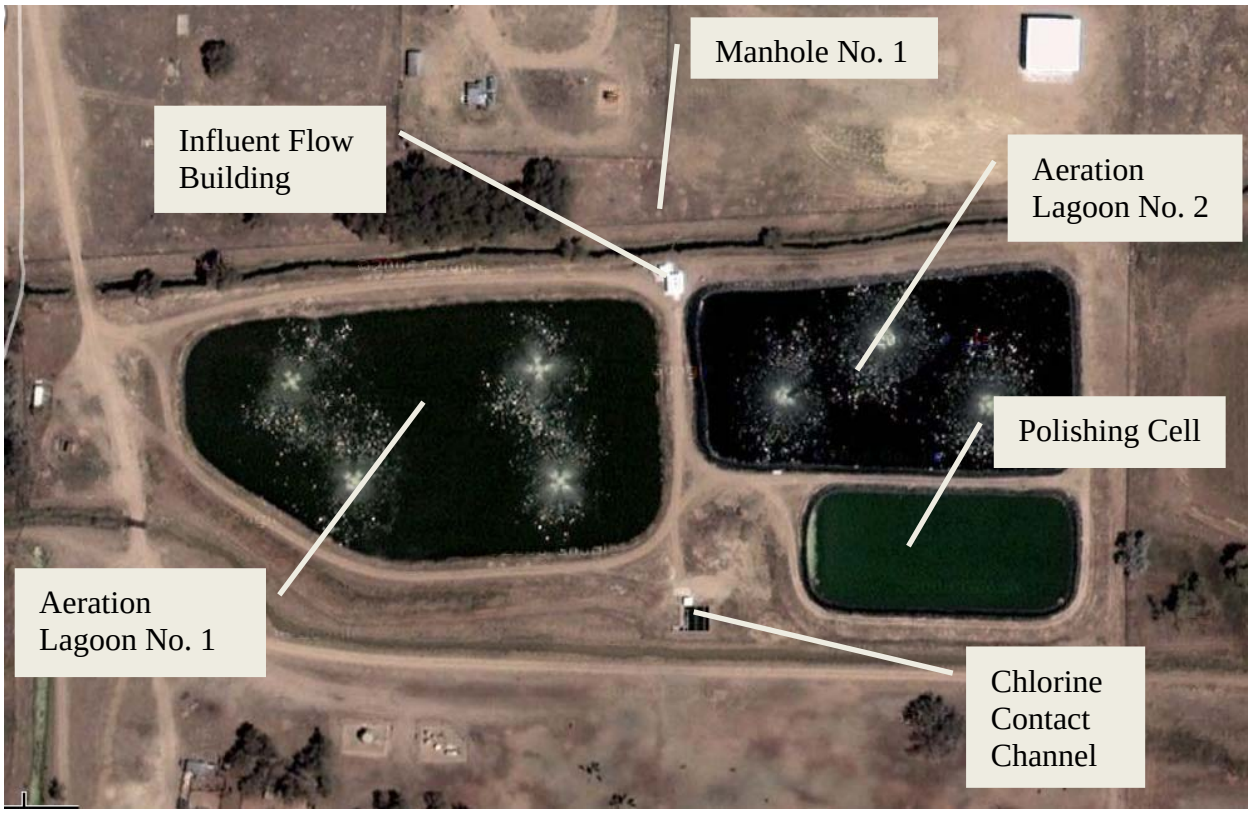


Figure 4-5. Hill-n-Park WWTF Site

4.3.1. Liquid Stream

The wastewater flow is collected in gravity branch lines and then flows to the 30 IN influent sewer which conveys these joined flows to Manhole No 1 located at the north end of the facility. At the Influent Flow Building, the flow travels through a 9 IN Parshall flume and then the water is screened through a 6 MM step screen. A series of ductile iron gravity pipes and control structures connect the three lagoon cells. Each control structure is equipped with slide gates that provide the ability to bypass a given cell. During regular operation, flow travels through a 16 IN pipe from the Influent Flow Building to Aeration Lagoon No 1.

Aeration Lagoon No 1 has volume of approximately 9.07 MG, a surface area of 147,000 SQ FT and a depth of 10 FT. The detention time is approximately18.1 days at the design flow of 0.5 MGD. The basin is mixed by six (6) 15 HP mixers resulting in approximately 10 HP per MG of volume. Flow then travels through a 12 IN gravity pipe to Aeration Lagoon No. 2.

Aeration Lagoon No. 2 has a volume of approximately 5.06 MG, a surface area of 100,125 SQ FT, and depth of 8 FT. Aeration Lagoon No. 2 has three (3) 15 HP mixers resulting in approximately 9 HP per MG of volume and is operated similar to Aeration Lagoon No. 1, but is designed for a detention time of 10.1 days. After traveling through Aeration Lagoon No. 2, a 12 IN gravity pipe conveys the flow to the final lagoon, the Polishing Cell.

The Polishing Cell has a volume of 2.03 MG with a surface area of 44,000 SQ FT and a depth of 8 FT. This final pond is not aerated and is designed for final clarification of the liquid stream. After a short detention time of 2.03 days at design flow, the stream continues through a 12 IN gravity line to the Chlorine Contact Basin.

The treated wastewater is disinfected with sodium hypochlorite, after which flow passes through the 40,000 gallon Chlorine Contact Basin to provide the required 30 minute detention time. The Contact Basin has a detention time of more than 2 hours at design flow of 0.5 MGD. Following chlorination, liquid sodium bisulfite is used for dechlorination. Before final discharge to South Platte River, the flow is measured through a second 9 IN Parshall flume. A 12 IN bypass pipe and gate valve provides the ability to divert flow around the chlorination process. The outlet of the bypass is just upstream of the effluent Parshall flume. Final effluent is conveyed to the South Platte River though a 21 IN pipe. Figure 4-6 provides a site plan of the facility.

4.3.2. Solids Stream

Solids stream facilities are limited to the material removed from the Influent Flow Building and stabilized biosolids from the lagoons. The waste biosolids are stored and digested in the lagoon system. Inert solids removed by the step screen are collected in a trash bin and sent to a landfill for disposal. Biosolids are removed from the lagoons approximately every 5 to 10 years, at which time it is dredged and disposed of in accordance with local, State and Federal regulations.

4.3.3. Capacity Evaluation

4.3.3.1. Current Conditions

The Hill-n-Park WWTF is designed to receive an average monthly influent flow of 0.5 MGD, with the actual average flow typically around 0.4 MGD. Organic and solids loadings are relatively consistent throughout the year, with average BOD_5 and TSS values of 251 mg/L and 245 mg/L, respectively. As with the Evans WWTF, the average flow being treated by the Hill-n-Park WWTF has increased with time as well as during major rain and snow melt periods.

4.3.3.2. Overall Facility Performance

Based on CDPHE CDPS Permit No. CO-47287, Aeration Lagoon No. 1 is designed to reduce the organic loading by 1,000 pounds of BOD per day with a detention time of 18.1 days at peak flow. Aeration Lagoon No. 2 is designed to reduce the organic loading by an additional 335 pounds of BOD per day with a detention time of 10.1 days at peak flow.

The performance of the Hill-n-Park WWTF from January 2005 through March 2009 is provided in Table 2-6 of Chapter 2, summarized herein as Table 4-4 and shows the current plant performance is in compliance with the CDPHE permit requirements. The permit dictates maximum effluent BOD_5 and TSS concentrations of 45 mg/L and 110 mg/L or less, respectively, with at least 85 percent removal of BOD_5 for 7-day average.

Thirty-day average concentration limits are 30 mg/L and 75 mg/L for BOD_5 and TSS, respectively.

Table 4-4. Historical Recorded Flow and Loading Data for the Hill-n-Park WWTF

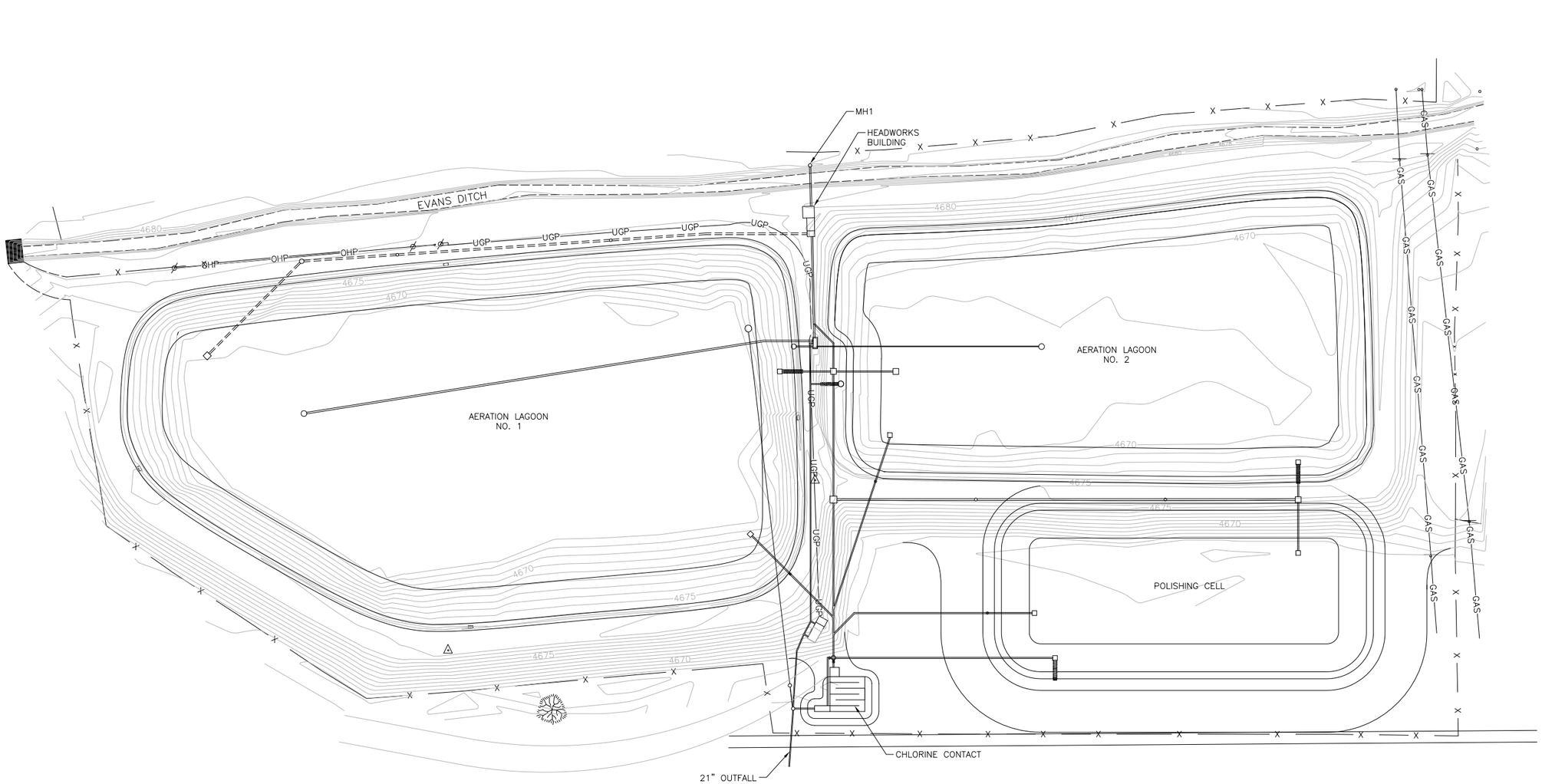
Month	Flow, MGD	Influent BOD_5, mg/L	Influent TSS, mg/L	Effluent BOD_5, mg/L	Effluent TSS, mg/L	Effluent NH_3-N, mg/L
2005 Average	0.346	273	283	20	31	-
2006 Average	0.399	292	296	20	31	-
2007 Average	0.419	270	238	18	28	-
2008 Average	0.422	251	245	18	28	24.25
2009 Average	0.484	269	224	20	32	22
Sum 2005 Ave[1]	0.363	271	309	19	29	-
Sum 2006 Ave[1]	0.411	294	299	21	34	-
Sum 2007 Ave[1]	0.422	263	238	18	30	-
Sum 2008 Ave[1]	0.419	253	241	19	36	-
Wint 2005 Ave[2]	0.333	275	256	21	33	-
Wint 2006 Ave[2]	0.387	290	293	19	29	-
Wint 2007 Ave[2]	0.415	278	231	18	26	-
Wint 2008 Ave[2]	0.424	247	239	17	19	-

[1] Average of April through September
[2] Average of January, February, March, October, November, and December

4.3.3.1. Nutrient Removal Performance

The discharge permit issued in June 2008 requires the facility to meet an effluent ammonia limit of 44 mg/L and 43 mg/L for the months of April and July, respectively. Effluent ammonia samples have consistently shown the facility meets the required limits. Because the facility is permitted for a flow rate below 1 MGD, WET testing is not required as compared with the Evans WWTF.

As with the Evans WWTF, there are no effluent limits for phosphorus, nitrate or nitrogen for the Hill-n-Park WWTF, though it is expected that limits will be imposed in the future.



OVERALL SITE PLAN

SCALE: 1" = 50'

C:\pwworking\oma\d0437481\FIG_4-06.DWG, Plot, 5/23/2011 11:09:09 AM, kbrewste

HDR

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
–	–	–

PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF
Overall Existing Facility Site Plan

0 1" 2"

FILENAME	FIG_4–06.DWG
SCALE	AS NOTED

SHEET **FIG. 4-6**

4.3.3.2. Liquid Stream

The treatment processes used at the Hill-n-Park WWTF is similar to the Evans WWTF. Future projections show the Hill-n-Park WWTF will need to increase capacity significantly. In the interim, the City has begun discussions with CDPHE to increase the capacity of the Hill-n-Park WWTF from 0.5 MGD to 0.99 MGD. CDPHE has issued preliminary effluent limits (PEL) for the re-rating which are summarized in Chapter 3. The most significant change is with regard to effluent ammonia. The PELs include a maximum day ammonia limit for the months of April and May, and 30-day average limits for June through September. Because the facility is still below 1 MGD, WET testing is not required.

To assess the capacity of the Hill-n-Park WWTF, a comparison has been made between the capacities of individual unit treatment processes and the flow and loading projections associated with the 0.99 MGD capacity rating.

Preliminary Treatment

Preliminary treatment facilities are located at the Influent Flow Building and include the influent pipeline, flow measurement and screening. The performance of each process is described below.

Influent Pipeline - The 30 IN influent pipeline is constructed at a slope of 1.4 percent resulting in a half-full pipe capacity of approximately 2.35 MGD. The pipeline has adequate capacity to meet short term flow projections but will require improvements to meet 20 year projections.

Influent Screen - Typical capacity criterion for step screens and screenings handling is its ability to handle peak hydraulic flows without surcharging the collection system upstream. The unit capacity is reported as 1.75 MGD, provided the mechanism's automatic cleaning operation is maintained in proper working order. The existing screen has a 6 MM screen spacing and performs well to eliminate debris from entering the lagoon system. The screenings are collected in a dumpster and periodically disposed of to a landfill. The screening equipment was installed in 2004 and was designed for a 20 year life. The screen has adequate capacity to meet short term flow projections (up to 0.99 MGD) but will require improvements to meet 20 year projections.

Grit Removal– No grit removal is currently provided. It is expected a majority of the grit removal occurs through settling in Aeration Lagoon No. 1.

Flow Measurement - The existing 9 IN Parshall flume is rated for a capacity up to 5.73 MGD. The flume has adequate capacity to meet both short and long term flow projections.

Secondary Treatment

Aerated Lagoons - As noted in CDPHE CDPS Permit No. CO-47287, the design of the Hill-n-Park WWTF was based on a total capacity of 1,335 pounds of BOD_5 per day in the three lagoon system (two aerated and one polishing), under average conditions. The lagoons use mechanical aerators, supplying approximately 10 HP/MG and 9 HP/MG of mixing power to Aeration Lagoon No. 1 and No.2, respectively. Aeration Lagoon No. 1

contains six (6) 15 HP mixers. Four original mixers were installed in 2000 with the additional two installed in the summer of 2009. Aeration Lagoon No. 2 has three (3) 15 HP mixer which were installed in 2000.

At the current average day rated capacity of 0.5 MGD and a working depth of 10 FT, Aeration Lagoons No. 1 and No. 2 have an SRT/HRT of approximately 18.1 days and 10.1 days, respectively. If flow is increased to 0.99 MGD in the near future, the SRT/HRT for Aeration Lagoons No. 1 and 2 decreases to 9.2 and 5.1 days respectively, which are very similar to the detention time provided at the Evans WWTF.

Similar to the Evans WWTF, the Hill-n-Park WWTF data shows winter lagoon effluent temperatures near 0 degC and as a result, temperatures this low significantly inhibit microorganism activity.

Based on results from kinetic modeling of the lagoon system with an influent BOD and TSS loadings of 251 mg/L and 245 mg/L and a winter water temperature of 9 degC, the Hill-n-Park WWTF has adequate capacity to treat up to 0.68 MGD for average daily flow conditions (model did not take into account the loss of lagoon volume to sludge/grit deposition). During the summer months when the average water temperature is approximately 23 degC, the facility has adequate capacity to treat up to 1.1 MGD. However, this calculation does not take into account ammonia removal. Because the system is only partially mixed, it is not feasible to model the treatment process with regard to nitrogen removal. Therefore, an understanding of how well the existing system removes ammonia at the higher flow rates is not known.

There are currently no limits for total phosphorus or nitrogen. Should full nitrification be required, as a result of more stringent nutrient removal limits in future permits, the existing lagoons would not be able to meet the discharge limits. Additionally, based on CDPHE PEL-200090, the re-rated Hill-n-Park WWTF will be required to remove ammonia during the months of April through September. Installation of additional surface mixing to the lagoons would increase the capacity for nitrification but may not provide the level of process control to meet lower ammonia limits. Additionally, due to slower growth rates for nitrifying bacteria, at low temperatures, hydraulic retention time becomes the limiting factor.

Polishing Cell - The volume of the polishing cell is approximately 2.03 MG and provides 4.1 days and 2 days of detention at the rated and proposed capacity, respectively. As mentioned previously for the Evans WWTF, the anaerobic conditions which occur in the Polishing Cell can cause the release of ammonia during the warmer months.

The lagoon treatment system has adequate capacity to meet short term flow projections but will require improvements to meet 20 year projections.

Disinfection

The current Hill-n-Park WWTF discharge permit limits the concentration of *E.Coli* which can be discharged to no more than 986 per 100 mL on a 30 day average and 1,972 per 100 mL on a 7 day average. Disinfection is accomplished by injecting liquid sodium hypochlorite to the wastewater and allowing time for contact prior to discharge. The

discharge permit does not currently require dechlorination; however it is required prior to June 2010.

Chlorination Feed - The Hill-n-Park WWTF disinfects by dosing liquid sodium hypochlorite to the effluent from the Polishing Cell. The hypochlorite system was installed in 2000 to replace a gaseous chlorine system. Sodium hypochlorite is stored in a 500 gallon tank located in a building near the channel. The solution is pumped from the tank by peristaltic pumps to the injector located approximately 5 FT upstream of the Chlorine Contact Basin. The system is flow paced off of the effluent flow meter. The facility typically uses between 200 and 300 gallons of sodium hypochlorite per month. To maintain a consistent working volume, the storage tank is normally refilled once a month.

Chlorine Contact Basin - The Chlorine Contact Basin has a volume of approximately 40,000 gallons and provides upwards of 115 minutes of detention at 0.5 MGD. Based on a minimum detention of 30 minutes, the chlorine contact basin has an average flow capacity of approximately 1.92 MGD.

The disinfection system has adequate capacity to meet short term flow projections but will require improvements to meet 20 year projections.

Effluent Outfall

Final effluent is discharged to the South Platte River through a 21 IN pipeline from the Chlorine Contact Basin to the river channel. The outfall was constructed in 2004 and does not include a diffuser to promote mixing of the wastewater effluent with the river. The outfall pipe has a slope of 0.12 percent which results in a full pipe capacity of approximately 3.5 MGD. To meet the future flow condition, a parallel 30 IN outfall pipe is required.

4.3.3.3. Summary of Capacity Analysis

The Hill-n-Park WWTF has an overall treatment and conveyance capacity of approximately 1 MGD based on current permit conditions (i.e. no ammonia limit). Based on a future peak hour wet weather flow rate of approximately 14 MGD, capacity expansions will require construction of new facilities. Improvements to the Hill-n-Park WWTF are discussed later in this chapter.

4.3.4. Solids Removal and Disposal

Similar to the Evans WWTF, biosolids accumulate and are stabilized in Aeration Lagoons No. 1 and 2 and the final Polishing Cell. Biosolids must be removed from the treatment system every 5 to 7 years.

The Hill-n-Park WWTF was last dredged in 2001 by Parker AG Services. Approximately 1,714 dry tons were removed from the lagoons and applied to a non-food producing winter wheat crop. Table 4-5 provides a summary of the biosolids parameters.

Table 4-5. Hill-n-Park WWTF - 2001 Biosolids Sampling Results

Parameter	Units	Hill-n-Park WWTF 2001 Sample
Average Percent Solids	%	7
Total Phosphorus	%	1
Potassium	%	0.23
Fecal Coliform (Geometric Mean-9 samples)	#/gram	6,000
Arsenic (As)	mg/kg	0.82
Cadmium (Cd)	mg/kg	0.50
Chromium (Cr)	mg/kg	11.8
Copper (Cu)	mg/kg	18.9
Lead (Pb)	mg/kg	11.6
Mercury (Hg)	mg/kg	0.05
Molybdenum (Mo)	mg/kg	1.0
Nickel (Ni)	mg/kg	7.5
Selenium (Se)	mg/kg	1.9
Zinc (Zn)	mg/kg	62

4.3.5. Support Facilities

4.3.5.1. Odor Control

The Hill-n-Park WWTF makes use of a partially mixed lagoon treatment system. Currently, no odor control is provided. The City has only received a minimal number of odor complaints over the past 5 years. The proximity to housing and commercial sites in the future may increase the potential for odor impacts on the community. Odor control for the proposed improvements will be evaluated during predesign.

4.3.5.2. Air Quality Permits

The Hill-n-Park WWTF does not have an air emissions permit. As noted previously, the facility makes use of partially mixed lagoon treatment system. Normal setback for this type of treatment system is 1,000 feet. The facility is situated away from housing and commercial development. The site currently does not have backup standby power generation.

4.3.5.3. Stormwater Management

There is no formal stormwater management plan for the facility as the site is presently exempted. The Hill-n-Park WWTF site slopes from north to south. Drainage from the site is conveyed off the site to the South Platte River through overland flow. If a formal site drainage plan and permit are required when expansion is implemented, one will be developed.

4.3.6. Summary

The Hill-n-Park WWTF capacity rating evaluation is summarized in Table 4-6.

Table 4-6. Hill-n-Park WWTF Capacity Rating Evaluation Summary

Unit Process	Capacity Criteria	Rated Capacity	Year 2020 Ave/Peak Anticipated Flows or Loading	Year 2030 Ave/Peak Anticipated Flows or Loading	Comment
Influent Pipeline	Hydraulic	2.35 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD.
Influent Flow Measurement	Hydraulic	5.73 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD.
Influent Screen	Hydraulic	1.75 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. Expansion required to meet future flow requirements.
Treatment Lagoons	Solids Loading (BOD/TSS)	1,335 lb/day	4,554 lb/day	6,780 lb/day	Adequate capacity for rerated 0.99 MGD. Expansion required to meet future flow and permit requirements.
Chlorine Disinfection Equipment and Channel	Hydraulic	1.92 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. System should be expanded or replaced to meet future flow requirements.
Outfall	Hydraulic	3.5 MGD	1.949 MGD/ 7.968 MGD	2.902 MGD/ 13.982 MGD	Adequate capacity for rerated 0.99 MGD. Parallel outfall should be constructed to meet future flow requirements.

4.4. Wastewater Treatment Alternatives Evaluation

This section presents the evaluation of alternatives for improving or expanding the wastewater treatment facilities. The evaluation process included the following basic steps:

- Summarize the existing and future limitations of the treatment systems and identify process improvement or expansion that will be required within the next ten to twenty years.
- List reasonable alternatives for detailed evaluation.

- Identify criteria to be used for evaluation.
- Develop alternatives in sufficient detail to permit a reasoned evaluation of their advantages and disadvantages.
- Develop capital and 20-year life cycle costs for reasonable alternatives.
- Identify a recommended plan.

4.4.1. Evaluation Process

Alternatives were identified and evaluated through an interactive process involving City and consultant staff. Major elements of the process are described below.

- **Brainstorm and Screen Ideas -** Any and all potential alternatives for expanding or improving the Evans wastewater treatment facilities were identified. An initial screening step was conducted to eliminate ideas that were fatally flawed, technically unproven, excessively expensive or otherwise unworthy of detailed evaluation.
- **Detailed Development and Evaluation -** Alternatives surviving the initial screening step were developed in detail. Facility sizing and cost estimating were conducted for modular expansion of facility capacity for 2020, 2030 and theoretical build-out conditions. Alternatives were compared based on cost and non-economic criteria. Based on this analysis, preliminary recommendations for facility improvements were made.
- **Decision Workshop -** Based on the results of the evaluation process, the project team developed final alternatives and recommendations for consideration by City Staff. One workshop was held to select the elements of the recommended plan.

4.4.2. Driving Forces

Improvements to the Evans wastewater treatment facilities are needed to provide reliable treatment capacity, to comply with regulatory requirements, to improve operational efficiency and to serve residential and commercial development. The key driving forces behind the needed improvements are summarized below.

- **Permit Revisions –** It is expected CDPHE will establish more stringent effluent quality requirements for the Evans WWTF and Hill-n-Park WWTF within the next 5 to 10 years. Establishment of more stringent limits for effluent nitrate, total inorganic nitrogen, and total phosphorus is particularly important since the current facilities are not designed to remove these nutrients.
- **Service Area Growth -** The City of Evans's service population is expected to grow at an annual rate of 4 percent, increasing wastewater loading especially to the Hill-n-Park WWTF (see Chapter 2).
- **Good Neighbor Considerations -** Encroachment of residential and commercial development is not an immediate concern, however; the increased value of property could result in development of the lands surrounding the Hill-n-Park WWTF. To ensure the treatment facilities do

not hinder potential growth, consideration must be made with regard to odor control and visual impacts.

4.4.3. Define Process Methodology and Evaluation Criteria

To provide a consistent planning basis, HDR developed an evaluation methodology for the wastewater facilities. This process defined evaluation criteria, outlined the decision-making process, and prescribed opinion of probable cost estimating procedures. The evaluation criteria are listed in Table 4-7. Except for the opinion of probable cost, these criteria were applied on a non-weighted qualitative basis when evaluation of alternatives was performed.

Table 4-7. Alternatives Evaluation Criteria

Regulatory Compliance	Implementation Criteria
• Meets current NPDES requirements	• Ability to logically phase expansion
• Flexible – Allows for potential future NPDES requirements	• Ease of construction
• Meets current and anticipated biosolids regulations	• Ability to maintain operation during construction
Operations/Technology	• Permit/approval requirements
• Proven performance/proven treatment process	**Community/Environmental Criteria**
• Low complexity	• Odor potential
• Operational ease	• Noise potential
• Ease of automation	• Vector attraction potential
• Reasonable maintenance	• Air quality impacts (non-odor)
• Reliability	• Truck traffic
• Longevity	• Hazardous chemicals
• Flexible – allow for future growth	• Public safety
• Compatible with existing facilities	**Risk**
• Safe/low use of hazardous chemicals	• Potential to fail due to changes in future regulations, public perception or land use
Cost	
• Construction cost/cash flow	**Compatibility with Site**
• Operations cost	• Ability to fit on site
• Life cycle cost	• Compatibility with surrounding land uses

4.4.4. Alternatives Brainstorming and Screening

Potential alternatives for expanding or improving the Evans and Hill-n-Park WWTF's were identified by HDR staff. Following the initial alternative development, an initial screening step was conducted to eliminate ideas which were fatally flawed, technically unproven, excessively expensive, or otherwise unworthy of detailed evaluation. The screening criteria used for evaluation are as follows:

- **Potentially Unacceptable to Regulatory Agencies -** If it is clear an idea would not be approved by CDPHE or another key regulatory agency, then the component failed the screening.
- **Technically Unworkable -** If the alternative idea does not perform as required, then it failed the screening.
- **Potentially Acceptable to Relevant Land Use -** If it was known an alternative idea would have no chance of being approved to NRPWQPA, it failed the screening.
- **Does Not Involve Extraordinary Costs -** An alternative idea failed the screening process if, in the opinion of the team members, the water quality and secondary benefits were expected to be much lower than the costs of comparable approaches which could accomplish the same results.
- **Community Impact Anticipated to be Acceptable -** This criterion was used to ensure the impacts on the community are acceptable. Examples of community impacts include odor, traffic, recreation, aesthetics and historical/cultural.
- **Environmental Impact Anticipated Being Acceptable -** An alternative idea failed the screening process if it was expected to cause adverse impacts on fisheries, wildlife habitat, air quality, water quality, wetland, rare/endangered species, or natural history that were impossible or difficult to mitigate.

The initial screening labeled each idea as “retain,” “fail,” or “feature.” These labels are defined as:

- **Retain, In-Scope:** Carry idea forward to detail alternative analysis as part of this facilities plan.
- **Retain, Not-in-Scope:** Valid idea, but outside the scope of this study. Address in concurrent or future studies.
- **Fail:** Idea is fatally flawed. Do not carry forward to detailed alternative analysis.
- **Feature:** Idea should be considered as a component of other ideas generated, or as a component of the pre-design.

Table 4-8. Evans WWTF – Alternative Development Ideas and Initial Screening Results

Idea		Initial Screening Result
Biological Process Improvements – Short Term to Meet Future Ammonia Limits:		
Alternative EB1	Construct submerged attached growth reactor after lagoon treatment system	Retain for evaluation.
Alternative EB2	Construct MBBR treatment system	Retain for evaluation.
Alternative EB3	Convert Polishing Pond to an aerated complete mix reactor	Retain for evaluation.
Biological Process Improvements – Long Term to Meet Nitrate and/or Total Nitrogen and Phosphorus Limits:		
Alternative EB4	Add denitrification filter to Alternative EB1	Retain for evaluation.
Alternative EB5	Convert Alternative EB2 MBBR to IFAS through addition of anaerobic and anoxic cells and the construction of secondary clarification	Retain for evaluation.
Alternative EB6	Expand aerated complete mix reactor from Alternative EB3	Retain for evaluation.
Alternative EB7	Demolish existing lagoon treatment system and construct conventional activated sludge system	Fail. Too expensive.
Alternative EB8	Demolish existing lagoon treatment system and construct membrane bioreactor system	Fail. Too expensive.
Alternative EB9	Decommission the Hill-n-Park WWTF, construct pump station and forcemain to convey all flow to the Evans WWTF, demolish existing Evans WWTF lagoon treatment system and construct Conventional Activated Sludge System	Fail. Too expensive.
Alternative EB10	Phosphorus removal through chemical addition and effluent filtration – Addition to all alternatives above	Retain for evaluation.
Disinfection Improvements		
Alternative ED1	Maintain existing liquid sodium hypochlorite disinfection system	Retain for evaluation.
Alternative ED2	Ultraviolet light disinfection	Retain for evaluation.
Alternative ED3	Irradiation	Fail. Not proven technology.
Alternative ED4	Ozone	Fail. Too expensive.
Biosolids Management		
Alternative EBM1	No Action - Maintain liquid application to agricultural land	Retain for evaluation. Existing system.
Alternative EBM2	Dewater and create Class A product	Retain for evaluation.
Alternative EBM3	Landfill	Fail. Too expensive and does not promote use of valuable product.

Idea		Initial Screening Result
Support Facilities		
Alternative ESF1	Replacement of standby power with larger unit	Retain for evaluation.

Table 4-9. Hill-n-Park WWTF – Alternative Development Ideas and Initial Screening Results

Idea		Initial Screening Result
Biological Process Improvements – Short Term to Increase Capacity and Meet Future Ammonia Limits:		
Alternative HB1	Convert Aeration Lagoons No. 1 to complete mix through addition of mixers	Retain for evaluation.
Alternative HB2	Construct MBBR nitrification cell into Aeration Lagoon No. 1	Retain for evaluation.
Biological Process Improvements – Long Term to Increase Capacity and Meet Nitrate and/or Total Nitrogen and Phosphorus Limits:		
Alternative HB3	Compartmentalize Aeration Lagoon No. 1, convert to complete mix through addition of mixers, add clarification and return flow	Retain for evaluation.
Alternative HB4	Construct MBBR nitrification and denitrification cells into Aeration Lagoon No. 1	Retain for evaluation.
Alternative HB5	Convert portion of Aeration Lagoon No. 2 to an aerated and complete mix reactor, add clarification and return flow	Retain for evaluation.
Alternative HB6	Demolish existing lagoon treatment system and construct Medium Rate Activated Sludge (MRAS) system	Retain for evaluation.
Alternative HB7	Demolish existing lagoon treatment system and construct Conventional Activated Sludge System	Retain for evaluation.
Alternative HB8	Decommission the Evans WWTF, construct pump station and forcemain to convey all flow to the Hill-n-Park WWTF, demolish existing Hill-n-Park WWTF lagoon treatment system and construct Conventional Activated Sludge System	Retain for evaluation.
Alternative HB9	Demolish existing lagoon treatment system and construct membrane bioreactor system	Fail. Too expensive.
Alternative HB10	Phosphorus removal through chemical addition and effluent filtration – Addition to all alternatives above	Retain for evaluation.
Disinfection Improvements		
Alternative HD1	Maintain existing liquid sodium hypochlorite disinfection system	Retain for evaluation.
Alternative HD2	Ultraviolet light disinfection	Retain for evaluation.
Alternative HD3	Irradiation	Fail. Not proven technology.
Alternative HD4	Ozone	Fail. Too expensive.

Idea		Initial Screening Result
Biosolids Management		
Alternative HBM1	No Action - Maintain liquid application to agricultural land	Retain for evaluation. Existing system.
Alternative HBM2	Dewater and create Class A product	Retain for evaluation.
Alternative HBM3	Landfill	Fail. Too expensive and does not promote use of valuable product.
Support Facilities		
Alternative HSF1	Installation of standby power	Retain for evaluation.
Alternative HSF2	Construction of administration/operations building	Retain for evaluation.

4.5. Detailed Evaluation of Alternatives

Following the initial brainstorm and screening steps, the remaining alternatives were developed in detail and compared against evaluation criteria. This section identifies the alternatives evaluated, presents major design criteria used in development of the alternatives, and describes the cost estimating methodology.

4.5.1. Design Criteria

An array of design criteria was established to guide development of the treatment alternatives.

4.5.2. Planning Horizon

Rather than selecting a specific timeframe for a planning horizon, alternatives were examined for two incremental increases in influent flows and loads at years 2020 and 2030. Tables 4-10 and 4-11 summarize the design flow conditions associated with these years for each treatment facility.

Table 4-10. Evans WWTF –Design Flows for Planning Horizon (MGD)

	2020	2030
Average Dry-Weather, MGD	1.277	1.432
Peak Hour Dry-Weather, MGD	2.043	2.291
Peak Hour Wet-Weather, MGD	3.516	4.087

Table 4-11. Hill-n-Park WWTF – Design Flows for Planning Horizon (MGD)

	2020	2030
Average Dry-Weather, MGD	1.949	2.902
Peak Hour Dry-Weather, MGD	2.729	4.063
Peak Hour Wet-Weather, MGD	7.968	13.982

4.5.3. Effluent Quality Requirements

Development of all unit processes was based on meeting the projected flow and loading conditions presented in Chapter 3. Flexibility was provided to meet potential future changes to effluent quality requirements such as limitation on total nitrogen or more stringent limits on phosphorus.

4.5.4. Design Criteria

Specific design criteria were established for each unit process. Design criteria were based on the following elements:

- **Process Sizing.** These criteria specify design loading rates and operating parameters for critical unit treatment processes. Examples include clarifier overflow rates, aeration basin mixed liquor concentrations, and chlorine contact basin detention times.
- **Reliability/Redundancy.** These criteria define reliability and redundancy requirements for unit processes and critical equipment. Reliability/redundancy is discussed in detail in each section of this chapter.
- **Short Term Water Quality Parameters.** The following effluent quality targets will be used for planning in the short term:

Biochemical Oxygen Demand (BOD_5)	<30 mg/L
Total Suspended Solids (TSS)	<30 mg/L
Ammonia Nitrogen	<1 mg/L

- **Long Term Water Quality Parameters.** It is expected the total nitrogen and a total phosphorus limit will be included in permits within the next 15 years. All alternatives discussed in the following sections must have flexibility to meet the following effluent quality targets in the future:

Biochemical Oxygen Demand (BOD_5)	<25 mg/L
Total Suspended Solids (TSS)	<25 mg/L
Ammonia Nitrogen	<1 mg/L
Total Nitrogen	<5 mg/L
Total Phosphorus	<0.5 mg/L

4.5.5. Solids Handling

For all alternatives, it is assumed solids from the biological process will continue to be stored and aerobically digested in the existing lagoon system. The City has indicated a desire to keep the solids handling process similar to the existing system. Solids will be dredged and land applied as required.

4.6. Evans WWTF Improvements

While effluent flow from the existing Evans WWTF typically meets or exceeds current permit requirements, the facility struggles to remove ammonia during cold weather

months (applicable to WET test). In addition, it is expected regulations on effluent ammonia, nitrate, and/or total nitrogen will be included in future permits. Finally, the facility is operating above 80 percent of the permitted flow and load capacity. In order to provide adequate capacity and meet regulatory requirements, short and long term alternatives were developed for the Evans WWTF. These include:

- Short Term – Do Nothing – The Evans WWTF is currently operating above 80 percent of the permitted flow and load. CDPHE requirements require improvements be implemented. As such, the "do nothing" alternative is not viable and was eliminated from further consideration.
- Short Term – Increase Capacity and Meet New Discharge Permit Ammonia Limits – For this alternative, improvements to the Evans WWTF are implemented to increase capacity and meet more stringent effluent ammonia limits.
- Long Term –Meet Nitrate and/or Total Nitrogen and Phosphorus Limits – For this alternative, the facility improvements include processes able remove nitrate and/or total inorganic nitrogen and phosphorus to meet the requirements of Regulation 31.

4.6.1. Biological Process Improvements – Short Term to Increase Capacity and Meet New Discharge Permit Ammonia Limits

Driving Forces

- **Capacity.** The Evans WWTF is currently operating at or above 80 percent of the permitted capacity. As noted in Chapter 3, the facility requires re-rating to increase the permitted flow and load. The lagoon treatment system is limited in the amount of influent BOD which it can treat.
- **Regulations.** Nitrogen in untreated domestic wastewater is normally in the form of ammonia (NH_3) with typical concentrations ranging from 30 to 45 mg/L. Ammonia is removed from wastewater by a microbial process which sequentially oxidizes to nitrite (NO_2^-) and then to nitrate (NO_3^-). The process requires excess amounts of dissolved oxygen and alkalinity and is termed nitrification. For a majority of wastewater treatment systems, the conversion of ammonia to nitrate is all that is required. As noted, for the Evans WWTF, the new discharge permit associated with the re-rating will include more stringent effluent ammonia limits. Additionally, it is expected the recently released Revised Federal Ammonia Criteria will reduce the effluent ammonia concentration limit in the Evans WWTF discharge permit.

Alternatives Considered

In order to increase the Evans WWTF's ability to remove ammonia and treat a higher influent flow and load, three alternatives were evaluated. All alternatives are based on a 2020 maximum monthly flow of approximately 1.66 MGD and average influent BOD_5, TSS, and NH_3 concentrations of 265 mg/L, 252 mg/L, and 32 mg/L, respectively. The alternatives included:

- Alternative EB1 – Construct submerged attached growth reactor after lagoon treatment system
- Alternative EB2 – Construct MBBR treatment system
- Alternative EB3 – Convert Polishing Pond to an aerated complete mix reactor

4.6.1.1. Alternative EB1 – Construct Submerged Attached Growth Reactor After Lagoon Treatment System

A submerged attached grown reactor (SAGR) is typically a post lagoon treatment process that can be installed to provide additional BOD, ammonia, and TSS polishing. The SAGR system utilizes a fully aerated attached growth system typically making use of dense rock media. Due to the attached growth, the system is able to nitrify at temperatures close to freezing. The SAGR is a clean gravel bed with evenly distributed wastewater flow across the width of the cell, and a horizontal collection chamber at the back end of the system. Aeration throughout the floor of the SAGR provides aerobic conditions that are required for nitrification. The gravel bed is often covered with a layer of peat or mulch for insulation. The reactor can be constructed of earth or concrete.

Figure 4-7. Submerged Attached Growth Reactor

As previously discussed, ammonia removal is significantly reduced during colder months, and the facility has failed WET testing multiple times, most likely due to ammonia levels. By utilizing a SAGR reactor after the lagoon system, ammonia levels can be reduced to meet anticipated permit requirements. Additionally, the SAGR reactor removes additional TSS from the lagoon effluent eliminating the need for the polishing cell.

For this alternative, the existing lagoon treatment system would be maintained for primary and secondary treatment. The improvements would include the following:

- Existing lagoon cells would provide BOD/TSS levels generally not exceeding 30/45 mg/l.
- Effluent from second treatment lagoon would flow through a 4-cell aerated horizontal flow SAGR for nitrification (ammonia removal). The new SAGR will have dimensions of approximately 600 ft wide by 300 ft long.
- Construct a new blower system and distribution piping to provide aeration for the SAGR at a flow of approximately 6,000 scfm.

A new UV disinfection system would be installed to increase capacity and reduce chemical costs. To provide backup power for the UV disinfection system, the standby

generator will be replaced with a larger unit. Finally, a new 24 IN parallel outfall pipeline would be constructed to meet future flow requirements.

Additionally, no changes are required to the influent screening, grit removal, or influent pumping.

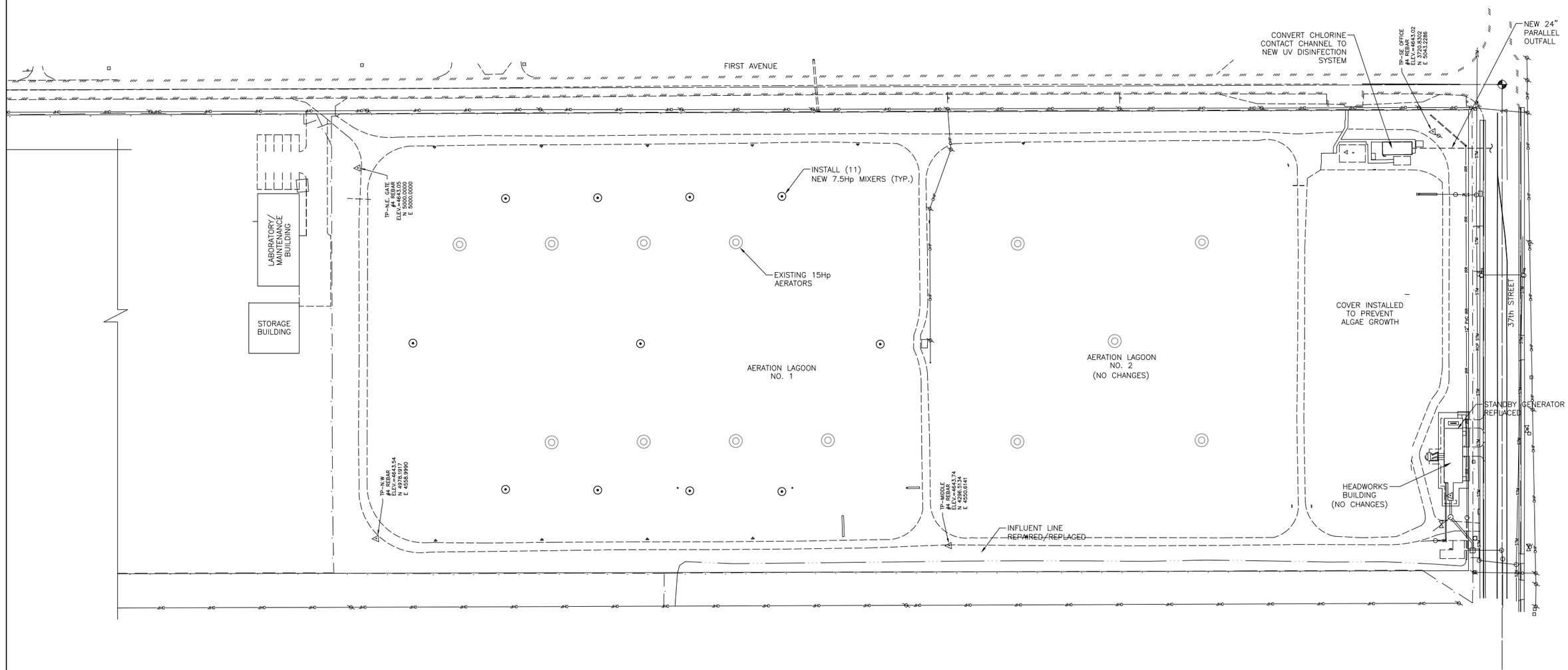
Advantages with this alternative include the following:

- Requires minimal operator attention (no real change from existing operations).
- Addresses ammonia removal during low winter time temperatures.
- Provides adequate ammonia removal to near term future permit conditions.
- Easy retrofit into existing treatment system.

Disadvantages include:

- Low process control – no guarantee as to quality of effluent. Can not meet more stringent effluent ammonia limits.
- Does not address nitrate removal.
- Does not address phosphorus removal.
- High construction cost.
- Due to the large area required for the SAGR system, additional land west of the existing facility will need to be obtained.

For this alternative, no changes would be made to the Headworks Building, Aeration Lagoon No. 2, Polishing Pond, or Chlorine Contact Channel. However, the influent line from the Headworks Building will need to be repaired/replaced to remove the plugging. Figure 4-8 provides a preliminary layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 60'

N

HDR

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
–	–	–

PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

The City of Evans COLORADO

EVANS WWTF - ALTERNATIVE EB1
Convert Aeration Lagoon No. 1 To Complete Mix Through Addition Of Mixers

0 1" 2"

FILENAME	FIG_4-08.DWG
SCALE	AS NOTED

SHEET **FIG. 4-8**

C:\pwworking\oma\d0437481\FIG_4-08.DWG, Plot: 5/19/2014 6:37:45 AM, eolson

4.6.1.2. Alternative EB2 – Construct MBBR Treatment System

Moving bed bioreactors (MBBR) fixed film treatment systems have been around for more than fifty years in the form of trickling filters and solids contact basins, however, only recently has the technology advanced to allow for floating and submerged attached growth media. Fixed film processes are inherently stable and resistant to organic and hydraulic shock loadings. In MBBR systems, the aeration basin biomass inventory is attached to either fixed or dispersed media and thus physically retained within the aeration basin. This allows a much higher solids inventory compared to a conventional lagoon treatment system. The fixed biomass also contributes to the ability of the process to respond to organic or hydraulic shock loads and to recover from upsets.

There are several types of media used to fix the biomass in the activated sludge basin. They include "dispersed media" entrapped in the aeration basin, and "fixed media," such as structured sheet media or knitted fabric media, that is placed in the aeration basin. The amount of biomass growing on the media is controlled by media design and fill ratio. Figure 4-9 shows a typical dispersed media and Figure 4-10 shows media retention screens.

Figure 4-9. MBBR Media

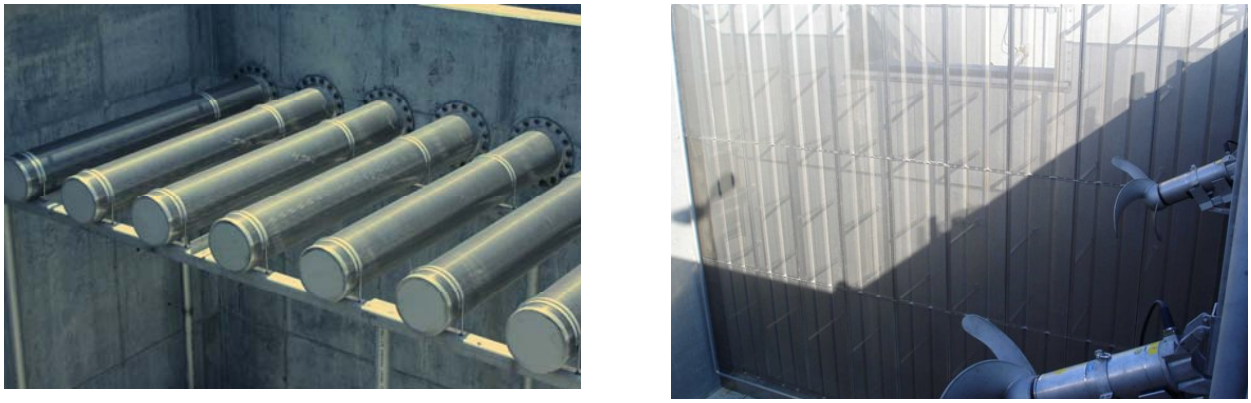


Figure 4-10. MBBR Media Retention Screens

Coarse bubble diffusers are necessary to eliminate diffuser maintenance due to the diffuser access limitations and are preferable to fine bubble diffusers, which tend to float the media. The media break up larger air bubbles and increase the retention time. The

oxygen uptake rates are similar to fine bubble diffusion giving the advantage of lower pressure aeration. Figure 4-11 shows a coarse bubble aeration system.

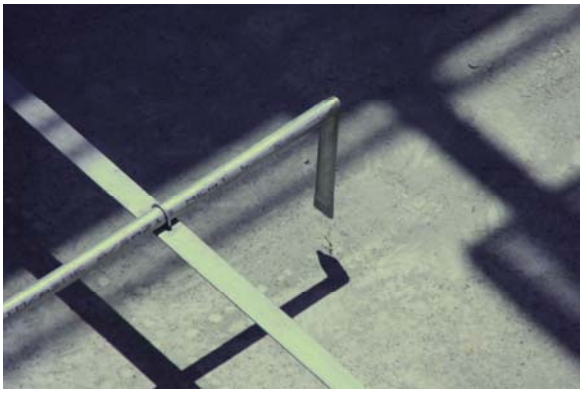


Figure 4-11. Coarse Bubble Aeration System

For this Alternative the existing lagoon treatment system would be taken offline and replaced with a two cell MBBR system. The first cell is for oxidizing BOD and the second for nitrifying ammonia. The basin would be constructed of concrete have overall dimensions of approximately 64 FT long by 64 FT wide by 18 FT deep resulting in a volume of approximately 552,000 gal. The tank is separated into two trains each with a 32 FT by 32 FT BOD carbon removal basin and a 32 FT by 32 FT ammonia removal basin.

Coarse air diffusers provide the required oxygen and maintain solids suspension. Retention screens keep the media within the basin. Effluent flows to the Polishing Cell for any final solids settling and then to a new UV disinfection system. A small blower system with two blowers is required for this alternative. The blowers would be housed in a 20 FT wide by 25 FT long building constructed near the existing maintenance building. To provide backup power for the UV disinfection system, the standby generator will be replaced with a larger unit. Solids collected in the Polishing Cell would be pumped to one of the existing treatment lagoons for stabilization prior to final land disposal. Due to the small size of the MBBR media and the required retention screens, fine screening is required upstream. The existing 0.25 IN opening influent screen system would be replaced with one having openings no greater than 3 MM. Finally, a new 24 IN parallel outfall pipeline would be constructed to meet future flow requirements.

A number of manufacturers supply MBBR media including the Siemens, Veolia, and Entex. In Colorado, the technology has been implemented successfully at a number of facilities including Johnstown and Broomfield.

Advantages for this alternative include the following:

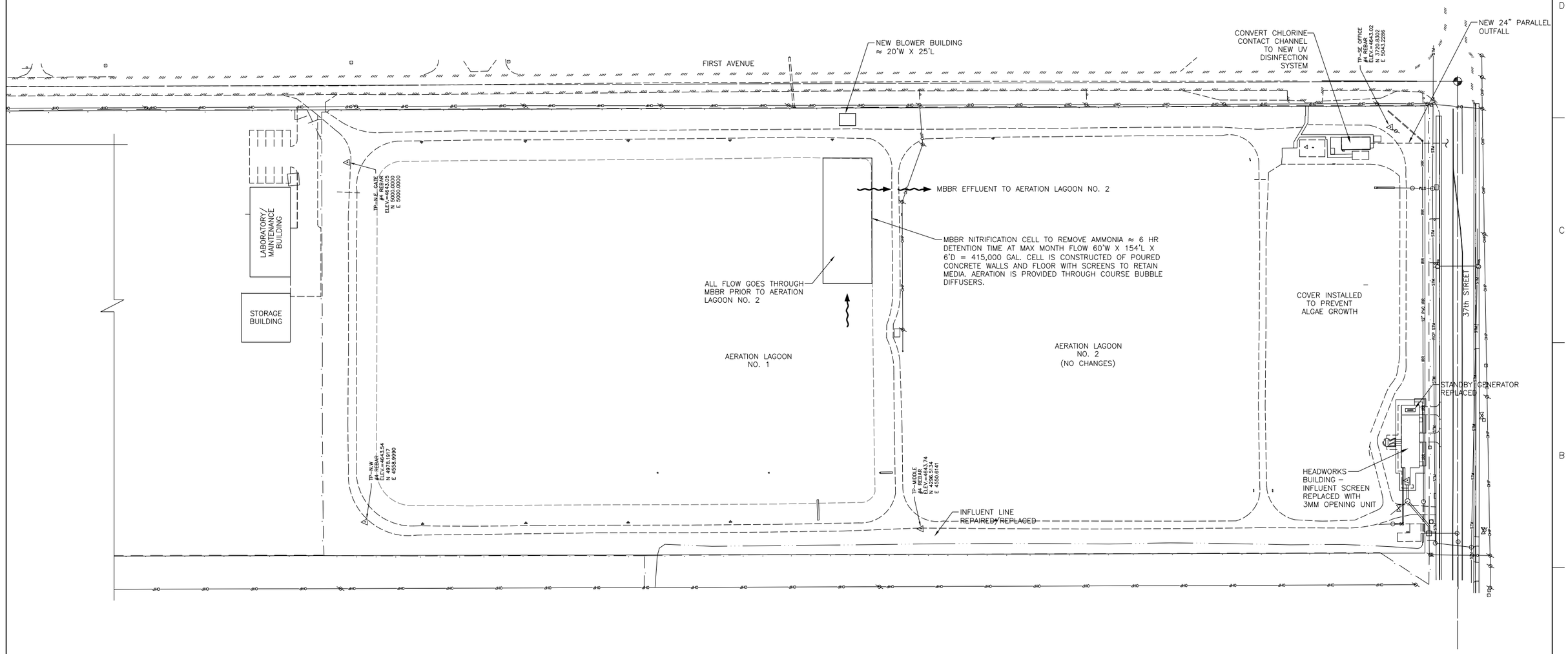
- Minimal operator attention. The media allows for greater bacteria population without the additional solids loading. The higher population provides for a more robust treatment process.
- Consistent ammonia removal to very low levels.
- Better process control than a standard lagoon treatment system.
- Relatively simple retrofit into the existing process.

- Good mixing through low energy input.
- Allows for future phasing of additional zones for removal of nitrate and phosphorus.
- MBBR system not as susceptible to cold temperatures as standard activated sludge system or lagoon systems.
- Allows phasing through the addition of media as needed to meet flow/load increases.
- Small footprint.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- MBBR media requires fine screening upstream.
- Increased operations and maintenance due to fine screening and new blower system.

For this alternative, no changes would be made to Aeration Lagoon No. 2, the Polishing Lagoon, or the Chlorine Contact Channel. Similar to Alternative EB1, the influent line from the Headworks Building will need to be repaired/replaced to remove the plugging. Figure 4-12 provides a preliminary layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 60'

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ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

The City of Evans Colorado

EVANS WWTF - ALTERNATIVE EB2
Construct MBBR Nitrification Cell In Aeration Lagoon No. 1

0 1" 2"

FILENAME	FIG_4–12.DWG
SCALE	AS NOTED

SHEET **FIG. 4-12**

C:\pwworking\oms\d0437451\FIG_4-12.DWG, Plot: 5/19/2014 6:39:05 AM, eelson

4.6.1.3. Alternative EB3 – Convert Polishing Pond to an Aerated Complete Mix Reactor

This alternative converts the existing lagoon system into a more traditional biological nutrient removal process through the use of a bioreactor, solids clarification, and mixed liquor and return activated sludge recycle. The aerated complete mix system is an extended retention biological treatment process similar to a lagoon. However, it differs from a lagoon with its use of aeration diffusers, which not only control oxygen input but also provide additional mixing. The air is diffused through a series of aeration chains which consist of tethered aeration floating headers and submerged membrane diffusers. As air is introduced into the system, the chain and diffusers move back and forth providing for enhanced mixing. The additional oxygen delivery allows for efficient BOD and ammonia removal. Figure 4-13 shows a photo of the general layout for this type of system. The aeration chains can be seen running across the lagoon.

Figure 4-13. Aerated Complete Mix System

There are a number of manufacturers who supply systems to convert treatment ponds into this style of bioreactor including the Parkson Biolac®, and EDI ATLAS-IS. Both systems make use of aeration chains which consist of tethered aeration floating headers and submerged membrane diffusers.

Based on a maximum month flow rate of 1.86 MGD, the new bioreactor would have a volume of approximately 2.465 MG and be constructed in the existing Polishing Pond area. Assuming a 10 FT side water depth (the existing lagoon requires additional excavation); the new bioreactor would have dimensions of approximately 274 FT long by 120 FT wide. Effluent from the new bioreactor flows to a new Secondary Clarifier also constructed within the existing Polishing Pond footprint. Solids wasted from the system are stored and stabilized in either Aeration Lagoon.

The bioreactor would be constructed of earth and lined with an HDPE liner to prevent leakage. The clarifier would be constructed of concrete and have a side water depth of 16 FT requiring excavation of the Polishing Pond. A pump located near the clarifier removes solids from the clarifier and recycle a portion (approximately 0.5 to 1 times the influent flow rate) back to the beginning of the process. Periodically (about once per week) solids are wasted to the remaining volume of Aeration Lagoon No. 1 for final stabilization through aerobic or facultative digestion.

Effluent from the Secondary Clarifier flows to the new UV disinfection system for final disinfection prior to discharge to the South Platte River. To provide backup power for the UV disinfection system, the standby generator will be replaced with a larger unit. Finally,

a new 24 IN parallel outfall pipeline would be constructed to meet future flow requirements.

Three blowers would be installed in a new blower building to supply air for the biological process. For this alternative, no changes are required to the influent screening, grit removal, or influent pumping systems. To achieve ammonia and nitrate removal, the air can be cycled on and off to provide simultaneous nitrification and denitrification. This type of operation has been used successfully in cold weather climates and has been shown to achieve total nitrogen levels as low as 8 mg/L.

The following is required to convert the existing Evans WWTF lagoon system to an aerated and complete mix system:

- Polishing Pond must be dredged, excavated and lined.
- A portion of the lagoon would be partitioned for use as the aerated complete mix system and Secondary Clarifier.
- Construct Bioreactor.
- Construct Secondary Clarifier.
- Construct blower building and air distribution header.
- Construct air delivery piping.
- Install RAS and WAS pumping and piping systems.
- Construct new influent pipeline.
- Construct new 24 IN parallel outfall pipeline.
- Replace standby generator.
- Convert either Aeration Lagoon No. 1 or No. 2 to sludge storage and stabilization lagoon. Relocate unused existing surface aerators to lagoon or convert to a facultative sludge lagoon.

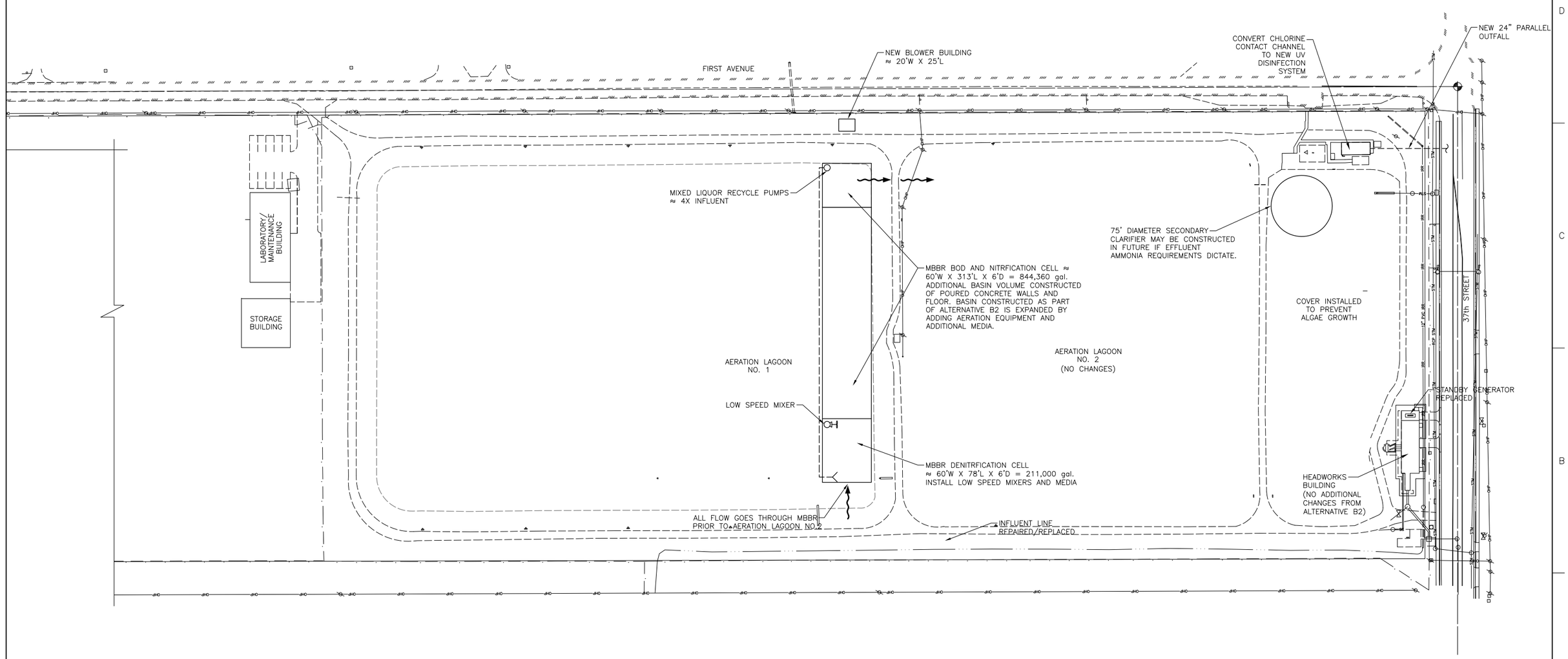
Advantages for this option include the following:

- Minimal operator attention.
- Very good ammonia and nitrate removal. Can be modified in the future to provide phosphorus removal.
- Better process control than a standard lagoon treatment system.
- Relatively simple construction and retrofit into the existing process- can be constructed in earthen basins. Requires the shutdown of only the Polishing Pond with the remaining treatment process staying in service. Does not require a modification to the existing permit as the City regularly operates the system without the Polishing Pond during certain periods of the year to minimize algae growth.
- Good mixing through low energy input.
- Lower energy input than complete mix alternative.
- Compact footprint.
- Ammonia released from anaerobically digested solids does not enter effluent.
- Smaller treatment system surface area decreases heat loss.
- Increased algae control.

Disadvantages include:

- Greater equipment complexity compared to a standard pond treatment system.
- Requires mixed liquor return pumping system.
- Requires return activated sludge pumping system and solids wasting.
- Requires construction of secondary clarifier.

Surface aerators which are no longer used in Aeration Lagoon No. 1 would be relocated to Aeration Lagoon No. 2. No changes would be made to the Headworks or Chlorine Contact Channel. Figure 4-14 provides the general layout for this alternative.



OVERALL SITE PLAN

SCALE: 1" = 60'

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

The City of Evans Colorado

EVANS WWTF - ALTERNATIVE EB4
Construct MBBR Nitrification and Denitrification Cells In Aeration Lagoon No. 1

0 1" 2"

FILENAME	FIG_4–14.DWG
SCALE	AS NOTED

SHEET **FIG. 4-14**

C:\pwworking\oma\d0437481\FIG_4-14.DWG, Plot: 5/19/2014 6:39:48 AM, edson

4.6.2. Biological Process Improvements – Long Term to Meet Nitrate and/or Total Inorganic Nitrogen and Phosphorus Limits

For the long term alternatives, the 2030 maximum monthly flow of 1.86 MGD was used. Additionally, for this alternative it is anticipated effluent permit limits will require the removal of nitrate and/or total inorganic nitrogen.

Driving Forces

- **Capacity.** Based on future growth, additional capacity at the Evans WWTF is required to meet the 2030 flow condition.
- **Future Regulations.** It is anticipated that future regulations will require the removal of total nitrogen from wastewater effluent to less than 5 mg/L, requiring a denitrification process and thus reducing nitrate to nitrogen gas that can be discharged to the atmosphere. This process can only be accomplished in anoxic (without dissolved oxygen) conditions. The anoxic atmosphere allows the bacteria to metabolize biodegradable substrate using nitrate as the electron acceptor. This process also returns some alkalinity and oxygen to the system which is consumed during nitrification. In order to provide the ability for nitrate removal in the future, the ammonia removal alternatives discussed in the previous section should be adaptable to one of the following alternatives.
- **Future Regulations.** It is anticipated that future regulations will require the removal of total phosphorus. This process will be accomplished through the use of a combination of biological and chemical addition/ effluent filtration.

Alternatives Considered

In order to increase capacity, add denitrification, and remove phosphorus, a number of alternatives were evaluated. The following alternatives consider the requirements previously described for ammonia, BOD and TSS removal as well as removing nitrogen and phosphorus from the system. The removal efficiencies vary slightly based on the technology and is discussed in the description of each alternative. Alternatives considered included:

- Alternative EB4 – Add denitrification filter to Alternative EB1
- Alternative EB5 – Convert Alternative EB2 MBBR to IFAS through addition of anaerobic and anoxic cells and the construction of secondary clarification
- Alternative EB6 – Expand aerated complete mix reactor from Alternative EB3
- Alternative EB7 – Demolish existing Lagoon Treatment System and construct a conventional activated sludge treatment system

- Alternative EB8 – Demolish existing Lagoon Treatment System and construct a membrane bioreactor treatment system
- Alternative EB9 - Decommission the Hill-n-Park WWTF, construct pump station and forcemain to convey all flow to the Evans WWTF, Demolish existing Evans WWTF Lagoon Treatment System and construct conventional activated sludge system
- Alternative EB10 – Phosphorus removal through chemical addition and effluent filtration – Addition to all alternatives above

4.6.2.1. Alternative EB4 – Add Denitrification Filter to Alternative EB1

This alternative is a modification of Alternative EB1 and can be constructed in the future if a total nitrogen and total phosphorus limit is implemented as a result of Regulation 31. For this alternative, the following improvements are required:

- Construction of denitrification filters after the SAGR reactor. The filters would convert nitrate to nitrogen gas and remove any remaining particulates.
- To denitrify, carbon addition is required. This would be accomplished through addition of methanol or similar. A new chemical storage and feed facility is required.
- To meet the phosphorus limit, a tertiary filter would be added. Alternative EB9 provides a discussion of the technologies.

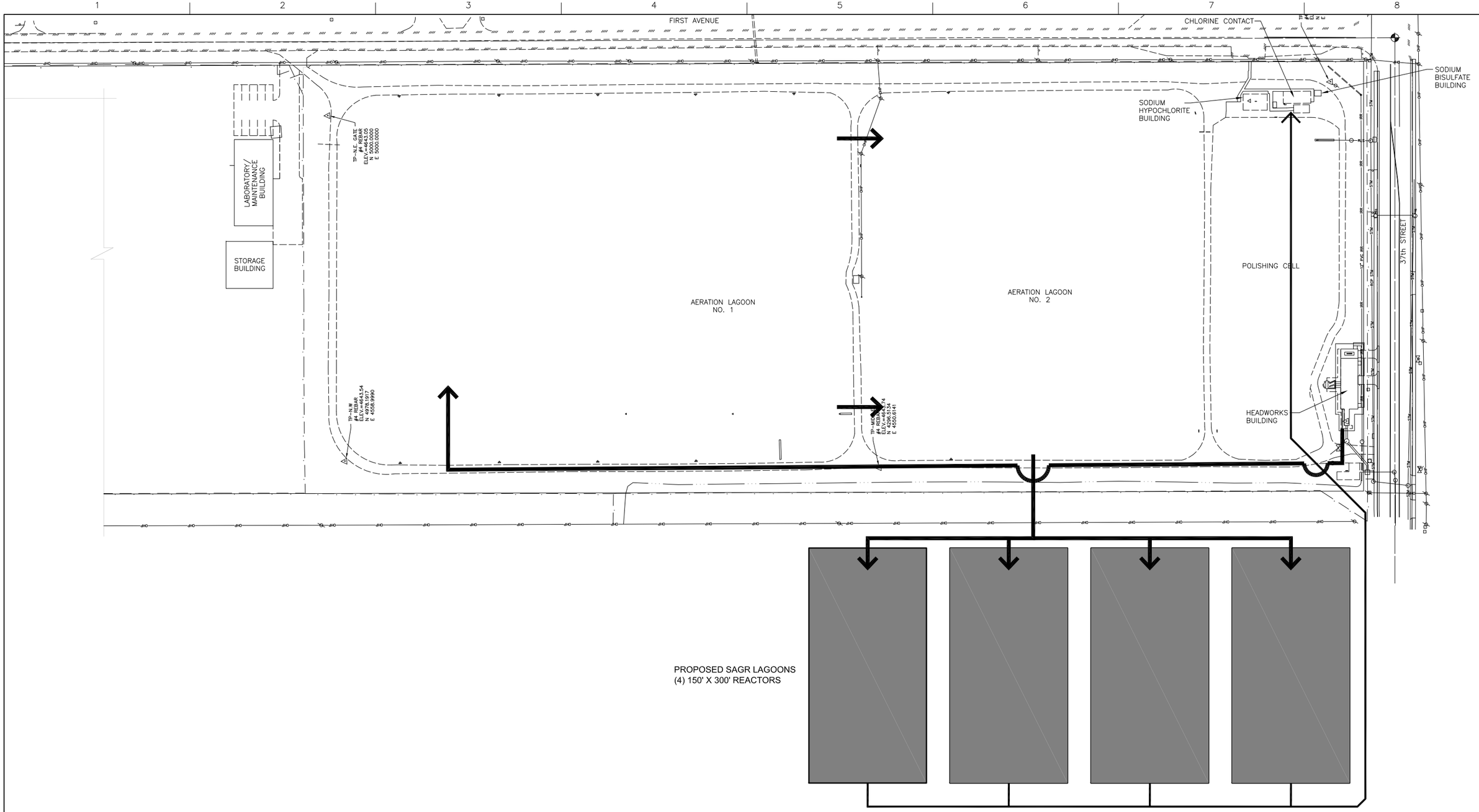
Advantages for this alternative include the following:

- Separate denitrification and phosphorus removal system provide precise control.

Disadvantages include:

- Greater equipment complexity.
- Large footprint.
- High capital cost.

For this alternative, no changes would be made to the Headworks Building, Aeration Lagoon No. 2, Polishing Lagoon, or Chlorine Contact Channel. It is assumed the influent line from the Headworks Building will be repaired/replaced as part of Alternative EB1. Figure 4-15 provides a layout of the proposed system.



OVERALL SITE PLAN

SCALE: 1" = 60'

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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

The City of Evans Colorado

EVANS WWTF - ALTERNATIVE EB4
Add Denitrification Filter to Alternative EB1

0 1" 2"

FILENAME	FIG_4-15.DWG
SCALE	AS NOTED

SHEET **FIG. 4-15**

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4.6.2.2. Alternative EB5 – Convert Alternative EB2 MBBR to IFAS Through Addition of Anaerobic and Anoxic Cells and the Construction of Secondary Clarification

The MBBR system, as previously discussed under Alternative EB2, has the capability to increase ammonia removal by providing media for nitrifying organisms to grow on. While a minimal amount of denitrification occurs with this system, it requires modification in order to meet future nitrate limits. For this alternative, anoxic and anaerobic basins would be constructed onto the basin constructed under Alternative EB2. Additionally, a secondary clarifier would be constructed to retain suspended growth bacteria. With these changes, the system operates as an integrated fixed film activated sludge system (IFAS). Mixed liquor from the secondary clarifier would be recycled to the tanks where it would be mixed and come in contact with floating dispersed media. Coarse air diffusers provide the required oxygen and maintain solids suspension. Retention screens keep the media within the basin. Mixed liquor will be recycled from the end of the aerobic zone to the anoxic zone. Final effluent from the secondary clarifier flows to the UV disinfection system prior to discharge to the South Platte River.

Conversion of the system constructed as part of Alternative EB2 for nitrate and phosphorus removal requires the following:

- Construct a 12 FT long by 32 FT wide anaerobic selector for each train.
- Construct a 32 FT long by 32 Ft wide anoxic zone for each train.
- Construct a secondary clarifier and RAS/WAS pumping.
- Install mixed liquor recycle pump at the end of the aerobic zone to return flow to the beginning of the anoxic zone.
-

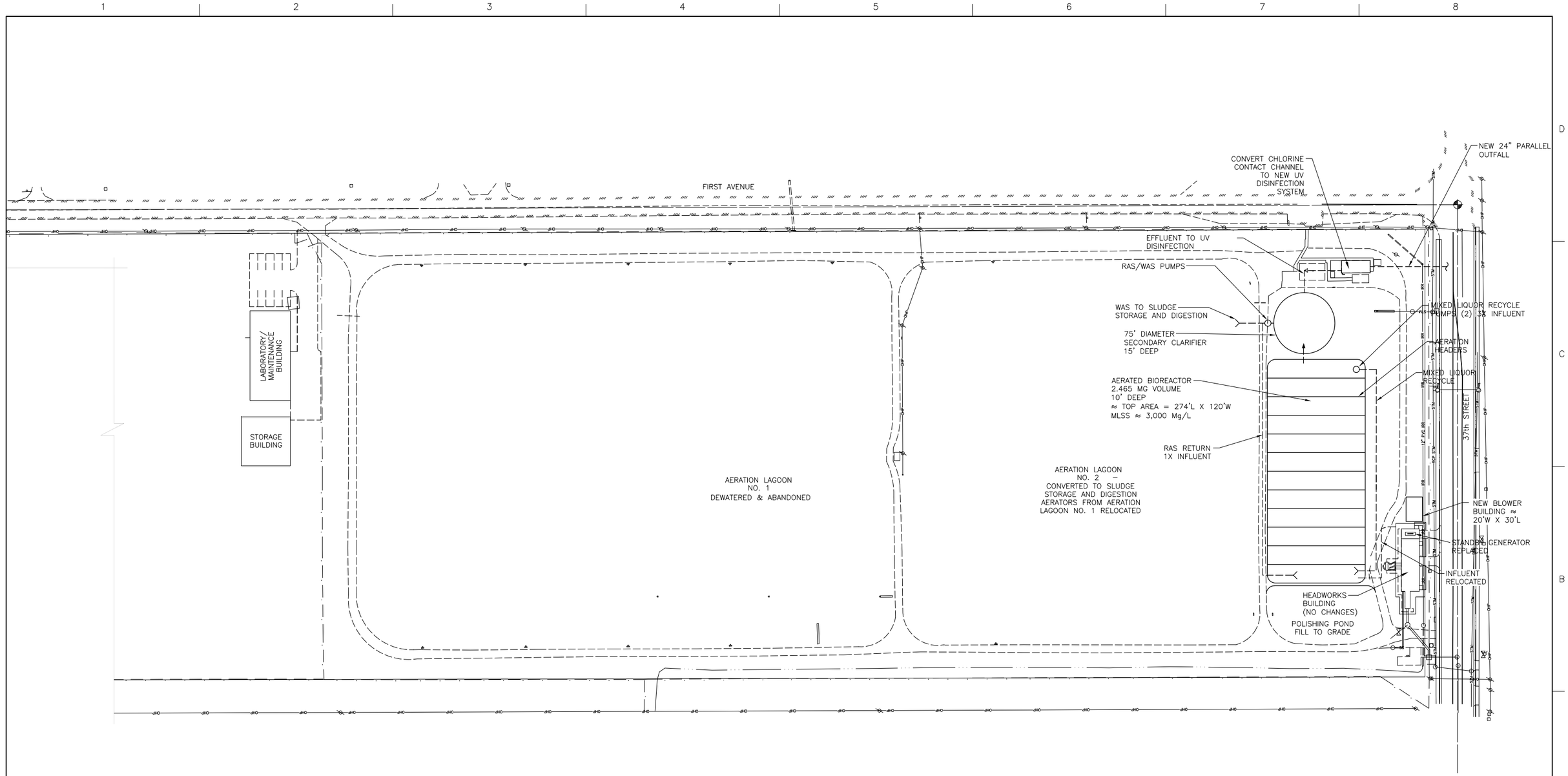
Advantages for this alternative include the following:

- Reduced operator attention (however, greater than lagoon treatment option).
- Very good nitrogen and phosphorus removal.
- Very good process control.
- Relatively simple expansion of Alternative EB2.
- IFAS system not as susceptible to cold temperatures as standard activated sludge system.
- Benefits of fixed film systems into the suspended growth activated sludge process – resistant to shock loads.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- MBBR/IFAS media requires fine screening upstream.
- May require construction of secondary clarifier in the future if effluent must meet very low ammonia limits.

Grit removal, influent pumping, and solids stabilization systems remain the same. This alternative assumes the influent line from the Headworks Building will be repaired/replaced. Figure 4-16 provides a layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 60'

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ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

The City of Evans Colorado

EVANS WWTF - ALTERNATIVE EB5
Convert Portion Of Polishing Pond To An Aerated And Complete Mix Reacter, Add Clarification And Return Flow

0 1" 2"

FILENAME	FIG_4–16.DWG
SCALE	AS NOTED

SHEET **FIG. 4-16**

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4.6.2.3. Alternative EB6 - Expand Aerated Complete Mix Reactor from Alternative EB3

This alternative expands the capacity of the aerated complete mix reactor from Alternative EB3 by increasing the size of the reactor and providing separated anoxic and anaerobic zones. The new zones would be constructed at the inlet to the reactor and can be constructed using earth basins or out of concrete. Additionally, mixed liquor recycle pumps would be needed to return activated sludge to the anaerobic zone for biological phosphorus removal.

4.6.2.4. Alternative EB7 – Demolish Existing Lagoon Treatment System and Construct a Conventional Activated Sludge Treatment System

For this alternative, the existing Evans WWTF lagoon treatment system would be demolished and a new conventional activated sludge system with anaerobic, anoxic and aerobic zones would be constructed in the footprint of the existing Polishing Pond. To meet low effluent total nitrogen limits, mixed liquor is recycled from the end of the aeration basin and combined with influent upstream of the anoxic zone. A new secondary clarifier would be installed next to the new bioreactor. Similar to the other long term alternatives, a new blower building would be constructed, a parallel outfall constructed, and the current sodium hypochlorite disinfection system would be expanded or replaced with a UV system. Waste activated sludge is pumped from the secondary clarifier to Aeration Lagoon No. 2 which would be converted to a sludge storage and stabilization basin (either aerobic or facultative).

Due to the cost associated with constructing this alternative compared to the others included in this section, it has been removed from further consideration.

4.6.2.5. Alternative EB8 – Demolish Existing Lagoon Treatment System and Construct a Membrane Bioreactor Treatment System

The membrane bioreactor (MBR) treatment system consists of a suspended growth biological reactor integrated with a membrane system that replaces the solids separation function of the secondary clarifier. The membranes are submerged and in direct contact with the mixed liquor. Through the use of a suction duty pump, a vacuum is applied to a header connecting the membranes. The vacuum draws the treated water through the hollow fiber membrane and into the pump where it is discharged. Air is introduced to the bottom of the membrane modules to produce turbulence to scour the external surface of the hollow fibers transferring rejected solids away from the membrane surface. The MBR technology overcomes the problem of poor settling associated with conventional activated sludge processes and allows for much higher mixed liquor concentrations. This results in an activated sludge system with a very compact footprint. Additionally, this allows for highly effective removal of both soluble and particulate biodegradable material in the waste stream. All particles larger than the membrane pore size are excluded from the effluent and wasted to the solids handling process, which would occur in Aeration Lagoon No. 2.

Process components for this option include fine screening, activated sludge system with anoxic and aerobic zones, and membrane filtration. Additional provisions include filtrate pumps, blowers, and chemical systems. Typical chemicals used in a MBR process include sodium hypochlorite, sodium hydroxide, and citric acid.

Membrane treatment also provides a nearly absolute barrier to solids carry-over, producing a clarified effluent with a turbidity of 0.1 NTU or less allowing for the effluent to be beneficially reused. The MBR process does require a higher volume of air for scouring of the membranes which results in more energy consumption for aeration when compared to conventional activated sludge processes (typically 50 percent more). However, recent advances in blower technology and the operation of the air scour system have decreased the total power compensation of an MBR by more than 30 percent.

Due to the cost associated with constructing this alternative and the additional energy required compared to the others included in this section, it has been removed from further consideration.

4.6.2.6. Alternative EB9 - Decommission the Hill-n-Park WWTF, Construct Pump Station and Forcemain to Convey All Flow to the Evans WWTF, Demolish Existing Evans WWTF Lagoon Treatment System and Construct Conventional Activated Sludge System

For this alternative, treatment facilities would be consolidated at the Evans WWTF. The Hill-n-Park WWTF would be decommissioned and a pump station constructed to convey all current and future wastewater to the Evans WWTF. As noted in Table 2-37, average daily and peak hour 2030 flows which must be pumped from the Hill-n-Park WWTF are approximately 4.3 MGD and 18.1 MGD respectively. The forcemain between the two facilities is approximately 3.5 miles and could be constructed along the South Platte River alignment.

As discussed in previous alternatives, the existing Evans WWTF treatment system must be replaced with an advanced system able to remove nitrogen and phosphorus. For the purposes of this discussion, it is assumed a conventional activated sludge biological nutrient removal system would be constructed in the footprint of the existing Polishing Pond.

Due to high cost associated with constructing the required pump station, forcemain, and additional treatment capacity at the Evans WWTF, this alternative has been removed from further consideration.

4.6.2.7. Alternative EB10 – Phosphorus Removal Through Chemical Addition and Effluent Filtration – Addition to Alternatives EB4-EB6

Effluent filtration is used to remove suspended solids which do not settle in the clarification process. Filtration will be required to meet the effluent anticipated Regulation 31 phosphorus limit. This alternative would be added to Alternatives EB4 through EB6 discussed previously.

Effluent filtration provides phosphorus removal by removing filterable particulate matter containing phosphorus. Effluent filtration also enhances downstream disinfection processes by increasing UV transmittance properties and clarity of the effluent.

There are many types of filters available for consideration. The alternatives listed below are discussed in more detail and provide a representation of the major types of filter technology available.

- Traveling Bridge Sand Media Filters
- Traveling Bridge Fabric Media Filters
- Compressible Media Filters
- Continuous Backwash Sand Filters
- Cloth Media Disc Filters
- Deep Bed Mono-Media Filters

Filtration basins and equipment must be housed in a covered and heated space to prevent freezing and facilities for polymer and alum/ferric addition will be required. These facilities will be incorporated into the site based on the final biological process selected.

Traveling Bridge Sand Media Filters

Traveling bridge filters utilize shallow (approximately 16-IN) granular media beds configured in long, narrow basins, typically 16-FT x 100-FT maximum width and length. A motorized bridge, located above the water in each filter basin, is equipped with backwash pumps. As the bridge travels the length of the filter, the filter is backwashed. The recommended hydraulic loading rate for this type of filter is 2 gpm/sf at average flow and not over 5 gpm/sf at maximum day flow. Approximately 645-SF of filter bed, or two beds, each 12-FT by approximately 12-FT wide would be required. A third filter would be needed, for reliability, when one filter is out of service.

Traveling Bridge Fabric Filters

Traveling bridge fabric media filters are also long and narrow. Their filtering concept is entirely different. The filter media consists of large hollow tubes covered with fabric media. The tubes are submerged and run parallel to each other over the long dimension of the filter basin. Hydraulic loading rates can be as much as 3.2 gpm/sf at annual average flow. This equates to approximately 404-SF of filter area.

Compressible Media Filters

A unique filter media is used in compressible media filters, 1-1/4-IN synthetic fiber balls. A moveable plate with a motor actuator compresses the media to a depth of 30-IN in preparation for operation. Flow is upward at hydraulic loading rates up to 30 gpm/sf. Because the hydraulic loading rate is over six times higher than granular media filters or fabric media filters, the footprint space is smaller than for other filtration alternatives. Maximum capacity is 2.8 MGD per tank; however, two smaller tanks will be required allowing one filter to be out of service. When backwashing is required, the moveable plate is raised to decompress the media. Low pressure air is used to scour the media.

Continuous Backwash Filters

Continuous backwash filters use a deep (generally 40-IN or more) granular media bed. Upflow and downflow configurations are available from various manufacturers.

Continuous backwashing is accomplished by pumping media from the bottom of the filter to the top with an air lift pump. The pumping action is used to scrub the media. The hydraulic loading rate at annual average flow would be 3.4 gpm/sf. At this loading rate, approximately 380-SF of filter is needed.

Cloth Media Disc Filters

Cloth media filters consist of several disc shaped frames, approximately 7-FT in diameter by 4-IN thickness, covered with cloth filter media and mounted on a hollow shaft. The entire disc assembly is submerged and backwash on each disc occurs by rotating the discs with reversed flow. Loading rates of up to 6 gpm/sf have been accepted nationally resulting in a total area of 215-SF. At that rate, a minimum of two filters are recommended, with one unit out of service.

Deep Bed Mono Media Filters

Deep bed mono media filters are similar in configuration to many of the filters used in potable water treatment plants. Granular media is supported on an underdrain collection system and the direction of flow is downward. Filter media is typically about 72-IN. The filters are backwashed by pumping water from a clearwell or contact basin into the underdrain, reversing the direction of flow. Compared with other filtration alternatives, backwash flows are considerably higher for this technology.

Effluent Filtration Evaluation

The final effluent filtration alternatives above will be necessary to meet the phosphorus limits in Regulation 31. Each of the filtration options presented above is technically capable of meeting treatment performance requirements. Cloth media disc filters and traveling bridge sand media filters typically have capital costs 15 percent less than the costs of the other options. This is due to the lower cost of the equipment for the traveling bridge sand filters and smaller building footprint requirement for cloth media disc filters for housing the equipment. It is recommended that once final effluent filtration becomes necessary to meet total phosphorus requirements, a more detailed evaluation of the effluent filtration alternatives be conducted during preliminary design.

Since continuous backwash sand media filters typically have the lowest capital and present worth costs, the estimated capital cost for this effluent filtration option is presented in the following section. To provide redundancy, three filtration units will be installed, each with a minimum capacity of 1.43 MGD for a total capacity of 4.29 MGD. To keep the system from freezing in the winter, the equipment will be stored in a temperature controlled prefabricated building.

Additional operations and maintenance requirements for this alternative include the following:

- Effluent testing for turbidity,
- Maintenance of effluent filters,
- Additional chemical storage and use (coagulant for filtration and hypochlorite for cleaning).

4.6.3. Evans WWTF Biological Process Improvements Evaluation

4.6.3.1. Cost Evaluation

This section summarizes the assumptions related to cost items and presents an opinion of probable project cost for each alternative. The comparison is concluded with an analysis of the probable cost of each alternative related to the overall benefit associated with each modification.

Assumptions used in the probable cost estimate for each alternative include:

- The cost of conveyance to and from the wastewater facility is not included.
- Excavated material can be disposed of on site and incurs no additional cost.
- All alternatives include the cost for replacing the standby generator and construction of the parallel 24 IN outfall pipeline.
- Conversion of the chlorine disinfection system to UV disinfection is not included in the cost and is detailed later in Section 4.6.4.
- Percentage values are included for miscellaneous mechanical and electrical needs.

Capital costs were estimated for the biological treatment alternatives evaluated. A summary of the costs are shown in Table 4-12. When comparing Alternatives, note that Alternatives EB1, EB2 and EB3 are short term only to increase capacity to meet 2020 conditions and addressing ammonia removal. Alternatives EB4, EB5, and EB6 address long term effluent water quality requirements including the removal of nitrate while increasing capacity to meet 2030 conditions. Alternative EB10 addresses removal of total phosphorus.

Table 4-12. Evans WWTF – Biological Process Cost Comparison

Description	Short Term Improvements			Long Term Improvements			
	Alternative EB1	Alternative EB2	Alternative EB3	Alternative EB4[1]	Alternative EB5[1]	Alternative EB6[1]	Alternative EB10
Base Construction Cost	$6,963,000	$2,107,000	$1,982,000	$588,000	$1,258,000	$821,000	$917,000
Electrical & Controls (20% of Equipment)	$304,000	$145,000	$216,000	$25,000	$116,000	$70,000	$64,000
Subtotal A	**$7,267,000**	**$2,252,000**	**$2,198,000**	**$613,000**	**$1,374,000**	**$891,000**	**$981,000**
Mobilization & Bonds (5%)	$363,000	$113,000	$110,000	$31,000	$69,000	$45,000	$49,000
Contractor's Overhead & Profit (10%)	$727,000	$225,000	$220,000	$618,000	$137,000	$89,000	$98,000
Subtotal B	**$8,357,000**	**$2,590,000**	**$2,528,000**	**$705,000**	**$1,580,000**	**$1,025,000**	**$1,128,000**
Miscellaneous Costs Not Itemized (30%)	$2,507,000	$777,000	$758,000	$212,000	$474,000	$308,000	$338,000
Subtotal C	**$10,864,000**	**$3,367,000**	**$3,286,000**	**$917,000**	**$2,054,000**	**$1,333,000**	**$1,466,000**
Engineering, Legal, Administration (20%)	$2,173,000	$673,000	$657,000	$183,000	$411,000	$267,000	$293,000
Total Project Cost	**$13,037,000**	**$4,040,000**	**$3,943,000**	**$1,100,000**	**$2,465,000**	**$1,600,000**	**$1,759,000**
Estimated Annual Operations Cost[3]	**$173,000**	**$117,000**	**$150,000**	**-**	**-**	**-**	**-**
20 Year Capital and Operations Present Worth[2]	**$10,314,000**	**$3,446,000**	**$3,609,000**	**-**	**-**	**-**	**-**

[1]Cost assumes equipment installed as part of short term improvements is used for long term improvement scenario and is not included a second time.
[2]Assumes electrical operations only at $0.08 per kWh. Present worth includes 4% interest rate, and 2% escalation rate. Present worth for Alternatives EB1, EB2 and EB3 based on the year 2020.
[3]Operations cost includes electrical cost only for operation of blowers.

The short term and long term combination of Alternatives EB1 and EB4 have the highest capital expenditure with a combined cost of approximately $14,137,000. This alternative also requires the purchase of land for the SAGR reactor which is not included in the cost.

Alternatives EB2 and EB5 make use of MBBR/IFAS technology and result in a combined short and long term capital expenditure of approximately $6,505,000. This alternative provides the greatest long term flexibility to meet future regulatory requirements.

The short term and long term combination of Alternatives EB3 and EB6 provide the lowest capital expenditure with a combined cost of approximately $5,543,000. However, selection of this alternative may result in difficulty meeting future more stringent effluent nitrogen and phosphorus limits because the initial reactor constructed is not as effective at meeting low nutrient limits.

Finally, Alternative EB10 has a capital cost of approximately $1,759,000. This alternative would be added to the selected biological improvement alternative to remove phosphorus once the Regulation 31 criteria are implemented.

4.6.3.2. Criteria Evaluation

In addition to cost, the treatment alternatives were evaluated using the criteria outlined at the beginning of this report. The criteria are weighted from 1 to 3 based on importance to the City and each alternative is then rated on a scale from 1 to 5. Alternatives which meet both short and long term requirements are included together. Table 4-13 summarizes this evaluation.

Table 4-13. Evans WWTF – Biological Process Alternatives Evaluation

Description	Weighting Value	Alternatives EB1 and EB4	Alternatives EB2 and EB5	Alternative EB3 and EB6
Regulatory Coordination	3	3/9	5/15	4/12
Operations/ Technology	2	5/10	4/8	4/8
Compatibility with Site	2	1/2	5/10	4/8
Implementation	3	2/6	4/12	3/9
Community/ Environmental	1	3/3	4/4	4/4
Risk	1	2/2	4/4	3/3
Cost (Capital Only)	3	1/3	3/9	4/12
TOTAL		**35**	**62**	**56**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.6.3.3. Evans WWTF Biological Process Improvements Recommendation

Based on capital cost, long term energy use, overall constructability, and non-economic criteria, the combination of Alternative EB2 and EB5-MBBR/IFAS is recommended for implementation to meet both the short and long term requirements. This alternative requires a greater initial investment to meet the short term ammonia limits, however, it

provides a lower overall operations cost and can be constructed without taking the existing treatment facility offline. Additionally, this alternative provides a higher degree of operational control and can be retrofitted in the future with Alternative EB10 meet effluent phosphorus limits.

4.6.4. Evans WWTF Disinfection Improvements

The Evans WWTF disinfection system is designed to utilize liquid chlorine for effluent disinfection. The current permit requirements for disinfection require an *E. Coli* bacteria count of no greater than 986 organisms per 100 mL (30-day average). Recent discussions indicate CDPHE will likely be modifying future permit limits to require *E. Coli* bacteria counts of no greater than 200 organisms per 100 mL. In addition, costs for chemical disinfection continue to increase.

Because permit requirements will most likely require more stringent disinfection practices, and due to the health hazard and cost associated with chlorine disinfection, the City has decided to evaluate alternatives for upgrade of their disinfection facilities. An analysis of the alternatives for disinfection is provided below.

Driving Forces

- **Improve Process.** Improved controls will be needed to provide reliable dechlorination if liquid chlorination is continued as the disinfection process used at the facility.
- **Permit Change.** The existing discharge permit limits effluent *e. coli* concentrations to 986 no/100 ml and 1,972 no/100 ml for the 30-day and 7-day average, respectively. New permit requirements are likely to be more stringent for bacteriological indicators.

Alternatives Considered

Four alternatives were considered for effluent disinfection to meet the more stringent effluent permit limits expected from CDPHE. Two of the four alternatives were retained for detailed evaluation because of their technical feasibility and cost effectiveness. The following effluent disinfection alternatives were evaluated for improving effluent disinfection capability:

- Alternative ED1 – Retain Liquid Sodium Hypochlorite Disinfection and Sodium Bisulfite Dechlorination.
- Alternative ED2 – Install Ultraviolet Light Disinfection System.

4.6.4.1. Alternative ED1 – Retain Liquid Sodium Hypochlorite Disinfection System

This alternative retains the existing liquid sodium hypochlorite disinfection system and sodium bisulfite dechlorination. This system has worked effectively for the City with current chemical costs total approximately $33,550 per year. Improvement of the existing system can be accomplished through the use of an effluent residual analyzer to automate the disinfection and dechlorination process.

4.6.4.2. Alternative ED2 – Ultraviolet Light Disinfection

Ultraviolet (UV) disinfection has been used for disinfecting primary, secondary or tertiary treatment effluents through out North America since 1980s. Recently, many wastewater treatment facilities utilize UV disinfection or are considering upgrading to it due to the regulatory restrictions on chlorine use and chlorination byproducts released to the environment from municipal wastewater systems.

Figure 4-17. Open Channel Low Pressure

There are several types of UV systems that could be installed at the Evans WWTF. These include the following:

Open Channel Low Pressure UV Disinfection. Open channel low pressure systems include UV lamps submerged in the contact channel in both vertical and horizontal arrangements. Low pressure ultraviolet bulbs have the ability to treat between 10 and 180 gallons per minute of wastewater. The existing chlorine contact channel would be modified to house the UV system. Two UV channels are required, each with a capacity of peak monthly flow. The UV system comes prepackaged in a steel channel with inlet/outlet transition boxes to connect to existing piping. Lamp replacement costs are typically $30 to $40 per unit. Power consumption is expected to be less than 2500 watts per day. Level control facilities, lamp removal hoists and monorails, and lamp cleaning systems would be installed. A portion of the existing chlorine storage room would be used for UV lamp maintenance and cleaning purposes, and for installation of the UV electrical service equipment. It is recommended a building be constructed over the UV channels to provide for weather protection and equipment maintenance.

Closed Medium Pressure UV Disinfection. This alternative includes removal of the permanent chemical disinfection systems and installing a closed channel UV technology. The closed channel technology requires directing wastewater flows via pipeline through closed chambers. This technology is readily available in modules up to 5 mgd. To accommodate the closed channel technology, it is anticipated two parallel flow pipelines, each capable of the peak monthly flow, would be installed. The pipelines would be installed in an ultraviolet light lamp gallery and would be accessible from ground level. The gallery would be configured with drainage systems to maintain the gallery dry, overhead hoist, and access stairs would be included for personnel access. A protective roof would be installed over the gallery to protect the UV equipment from the weather. Once again, the existing chlorine feed and storage

Figure 4-18. Closed Medium Pressure

rooms could be used for required electrical service equipment associated with the UV system. Control valves and system automation would be installed to bring the multiple parallel systems on line as required by plant flow conditions.

4.6.4.3. Disinfection Alternatives Analysis Summary

Key advantages and disadvantages of the alternatives for disinfection are presented in Table 4-14.

Table 4-14. Advantages and Disadvantages of Disinfection Alternatives

Disinfection Option	Advantages	Disadvantages
Retain Hypochlorite Disinfection and Dechlorination (ED1)	• Lower initial capital cost • Familiar Systems • Simple implementation	• Complicated controls with chlorine residual analyzers • Safety concerns with chemical handling • High chemical costs
Ultraviolet Light Disinfection (ED2)	• Small footprint for future facilities • Minimizes toxic gas and chemicals stored on-site • More reliable to meet effluent disinfection limits • Less environmental impact	• Higher initial capital cost • Less familiar technology • Greater mechanical complexity • Smaller chemical system required for in-plant uses and possible reclaimed water needs

Based upon the projected flow and loadings presented in Chapter 2, the existing chlorine feed system and chlorine contact channels only have sufficient capacity for existing conditions. Evaluation of the disinfection alternatives considers expansion of the facilities to 2030 flow conditions with expandability to theoretical build out conditions.

Estimated capital and present worth costs for the disinfection alternatives are presented in Table 4-15. Alternative ED1 maintains the existing liquid chlorine disinfection process, but expands capacity and adds residual control. This alternative offers the lowest initial capital cost, yet maintains larger quantities of chemicals on-site and carries a greater risk to toxicity concerns than the UV light alternative. The liquid hypochlorite alternative has a much lower initial capital cost than the UV alternative. The UV alternative requires greater initial capital cost, yet provides a more environmentally acceptable solution. The City would improve site safety considerably by minimizing the quantity of chlorine handled and transported to the site.

Table 4-15. Evans WWTF – Disinfection Cost Comparison

Description	Retain Hypochlorite Disinfection and Dechlorination (ED1)	Install UV Light Disinfection (ED2)
Base Construction Cost	$72,000	$304,000
Electrical & Controls (20% of Equipment for Alt D1 and 30% of Equipment for D2)	$14,000	$54,000
Subtotal A	**$86,000**	**$358,000**
Mobilization & Bonds (5%)	$4,000	$18,000
Contractor's Overhead & Profit (10%)	$9,000	$36,000
Subtotal B	**$99,000**	**$412,000**
Miscellaneous Costs Not Itemized (30%)	$30,000	$124,000
Subtotal C	**$129,000**	**$536,000**
Engineering, Legal, Administration (20%)	$26,000	$107,000
Total Project Cost	**$155,000**	**$643,000**
Total Operations Cost/Year	$42,000	$33,000
Total Present Worth	**$637,000**	**$912,000**

The disinfection alternatives were also evaluated using the established evaluation criteria. Table 4-16 summarizes the evaluation. The alternative to retain liquid chlorine disinfection, although it offers less initial capital costs, has significant shortfalls when community/environmental and regulatory considerations are taken into account. Installation of the ultraviolet light open channel low pressure alternative offers cost, implementation and community/environmental benefits.

Table 4-16. Evans WWTF – Disinfection Alternatives Evaluation Summary

Evaluation Criteria	Weighting Value	Retain Hypochlorite Disinfection and Dechlorination (ED1)	Install UV Light Disinfection (ED2)
Regulatory Coordination	3	2/6	4/12
Operations/ Technology	2	3/6	4/8
Compatibility with Site	2	4/8	4/8
Implementation	3	3/9	2/6
Community/ Environmental	1	1/1	4/4
Risk	1	2/2	5/5
Cost (Capital Only)	3	4/12	3/9
TOTAL		**44**	**52**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.6.4.4. Disinfection Alternatives Preliminary Recommendations

Although capital costs for installation of UV light disinfection are greater than upgrading the chemical disinfection system, there are considerable advantages for the City of Evans to consider installation of UV light disinfection. UV light is less susceptible to market fluctuations for chemicals, carries less risk for toxic chemical spill and provides greater assurance that future regulatory requirements are met. Since the existing chlorination system would need to be modified, it is recommended the City install the UV disinfection system (Alternative ED2) to alleviate capacity limitations and to meet future, more stringent, permit requirements. Both UV disinfection technologies are applicable to the Evans WWTF. Selection of a final type shall be performed during predesign.

4.6.5. Biosolids Management

Evans currently disposes of biosolids from both facilities through contract dredging and land application. This method has been cost effective over the years due to the lower flow rates. The purpose of this section is to explore options for biosolids disposal from the Evans WWTF assuming one of the advanced biological treatment alternatives discussed previously is constructed in the future.

Driving Forces

- **Cost.** The current cost of biosolids disposal is relatively inexpensive. In the future, available land required may not be available to meet the solids loading demand resulting in the City paying significantly more for removal and disposal.

Alternatives Considered

Alternatives considered for improvements to the Biosolids Management facilities are:

- Alternative EBM1 - Maintain Liquid Application to Agricultural Land.
- Alternative EBM2 - Dewater and Compost.

4.6.5.1. Alternative EBM1 – No Action - Maintain Liquid Application to Agricultural Land

Biosolids from the Evans WWTF are currently dredged and land applied on agricultural fields every 5 or so years. This type of solids stabilization and biosolids management works very well for small lagoon treatment systems. With the construction of the new biological nutrient removal treatment system, additional biosolids production is anticipated which may result in dredging the solids stabilization basin on a more regular basis.

4.6.5.2. Alternative EBM2 – Dewater and Create Class A Product

As discussed in Chapter 3, regulations for the land application of Class B biosolids are expected to become more restricted. Additionally, due to the cost associated with dredging and land application, maintaining the current method for biosolids management may not be feasible. For this alternative, sludge dredged from the storage and stabilization lagoon is dewatered and then either pasteurized or composted to produce

biosolids that meet Class A requirements. There are several technologies available for achieving this including lime addition, pasteurization at 160 DegF, and composting.

For the purposes of this evaluation we have considered the use of a screw press in which steam could be injected for heating and pasteurizing biosolids. The equipment would be housed in a new dewatering building. Stabilized solids would be pumped from the lagoon to the dewatering building and then stored prior to disposal.

The screw press is a mechanical device used for liquid/solid separation. A cross section of the press is shown in Figure 4-19. Liquid/solid separation is accomplished by gradually reducing the volume available for the solids as they are conveyed from the inlet to the outlet end of the screw press. The reduction in volume is achieved by using a tapered shaft that is larger in diameter at the discharge end than the inlet end. The shaft is surrounded by a screen system that contains small (less than 1/8-inch diameter) holes.

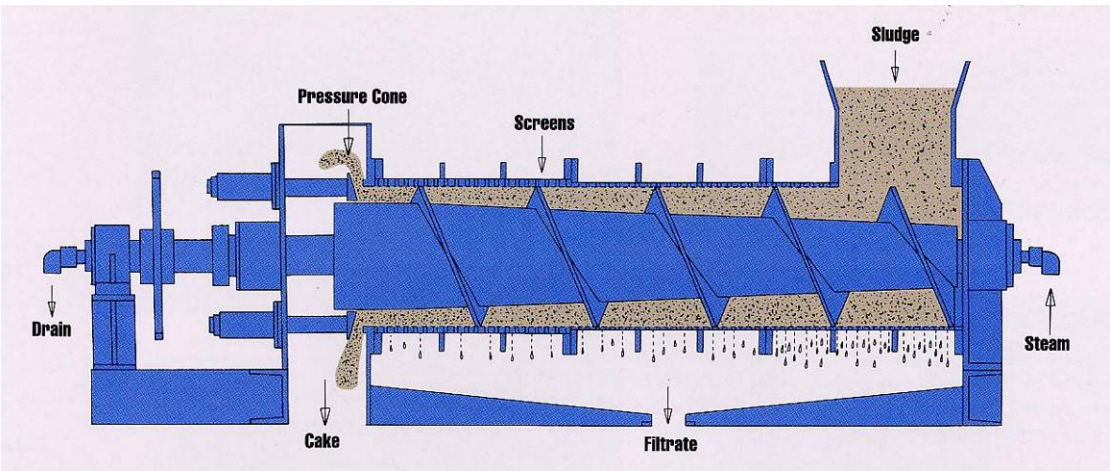


Figure 4-19. Cross-Section of Screw Press

A key advantage to this technology is the ability to inject steam into the center shaft to heat and pasteurize the solids. Screw press manufacturers claim that the equipment can provide a cake solids concentration of approximately 20-25 percent, similar to typical results from a centrifuge.

Advantages of this alternative include much greater flexibility in the use of the biosolids for agricultural applications. Disadvantages include high capital expenditures to construct the new facilities. Additionally, dewatering processes typically are operations intensive and will require an additional full time employee.

4.6.5.3. Cost Evaluation

This section summarizes the assumptions and possible unknowns related to cost items and present an opinion of probable project cost for each alternative. The comparison is concluded with an analysis of the probable cost of each alternative related to the overall benefit associated with each modification.

Assumptions used in the probable cost estimate for each alternative include:

- Excavated material can be disposed of on site and incurs no additional cost.
- Percentage values are included for miscellaneous mechanical and electrical needs.

Capital costs were estimated for the biosolids management alternatives evaluated. A summary of the costs are shown in Table 4-17.

Table 4-17. Evans WWTF – Biosolids Management Cost Comparison

Description	Alternative HBM2
Base Construction Cost	$595,000
Electrical & Controls	$47,000
Subtotal A	**$642,000**
Mobilization & Bonds (5%)	$32,000
Contractor's Overhead & Profit (10%)	$64,000
Subtotal B	**$738,000**
Miscellaneous Costs Not Itemized (30%)	$221,000
Subtotal C	**$959,000**
Engineering, Legal, Administration (20%)	$192,000
Total Project Cost	**$1,151,000**

4.6.5.4. Criteria Evaluation

In addition to cost, the alternatives were evaluated using the criteria outlined at the beginning of this report. The criteria are weighted from 1 to 3 based on importance to the City and each alternative is then rated on a scale from 1 to 5. Alternatives which meet both short and long term requirements are included together. Table 4-18 summarizes this evaluation.

Table 4-18. Evans WWTF – Biosolids Management Alternatives Evaluation

Description	Weighting Value	Alternative HBM1	Alternative HBM2
Regulatory Coordination	3	2/6	4/12
Operations/Technology	2	4/8	3/6
Compatibility with Site	2	4/8	2/4
Implementation	3	4/12	2/12
Community/Environmental	1	2/2	4/4
Risk	1	2/2	4/4
Cost	3	5/15	2/6
TOTAL		**53**	**47**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.6.5.5. Recommendations

At this point in time, it is recommended the current biosolids management (Alternative EBM1) be maintained. However, in the future, regulations or growth may dictate looking at an alternative method for disposal. Biosolids management should be reviewed every 5 years.

4.6.6. Evans WWTF Support Facilities

Support Facilities, in this evaluation, comprise electrical power supply and standby power. Additionally, the Evans WWTF does not have an operations and maintenance building.

Driving Forces

- **Improve Process.** Biological nutrient removal processes require a stable environment. Loss of power and resultant loss of air supply can result in process upsets. Additionally, standby power is required to provide for continuous disinfection during a power outage.

4.6.6.1. Standby Power

Installation of a standby power generator is recommended for critical equipment required to maintain the biological process and to power the UV disinfection system. The existing generator is only sized for the Influent Pumping system. It is recommended the existing unit be replaced with an in-place engine generator with an automatic transfer. The standby generator should be capable of operating all primary equipment (influent pumping, aeration, process pumping, clarifier mechanism, and UV disinfection) and be provided with an automatic transfer switch to provide for un-manned restart of equipment in the event of a power outage. It would be installed in the same location as the existing. The diesel storage tank should have a minimum storage capacity to allow full load operation of the generator for 24 hours.

The standby power generator would likely not be used often. Therefore, it would need to be exercised approximately weekly. Exercising should include turning the generator on and running at full capacity for a period of time determined by the manufacturer. Size of the standby generator should be determined during pre-design but based on the size and loads in the facility will likely have a capacity between 600 and 1,000 kW. Estimated construction cost for this alternative is approximately $162,000.

4.7. Hill-n-Park WWTF Improvements

The existing Hill-n-Park WWTF was recently re-rated from 0.5 MGD to 0.99 MGD which required improvements including the addition of two new surface aerators in Aeration Lagoon No. 1 and baffles in both of the first two lagoons. A discussion of the Effluent Limits associated with the rerating is included in Chapter 3. Increasing the capacity requires the Hill-n-Park WWTF to remove ammonia and it is expected that regulations on effluent nitrogen will be imposed in the future. Similar to the Evans biological process improvements evaluation, the Hill-n-Park WWTF would require short term improvements to meet ammonia removal requirements and long term improvements

to further increase average daily flow capacity to approximately 3 MGD and remove nitrate/phosphorus. The sizing and costs for the long term improvements are for the full volume, however, most of the alternatives can be phased as necessary.

Similar to the Evans WWTF, all alternatives for improving the Hill-n-Park WWTF assume solids storage, digestion, and disposal should remain the same as existing with dredging and land application when necessary.

Finally, the Hill-n-Park WWTF does not currently have backup power. The addition of a standby generator is required for all alternatives.

4.7.1. Biological Process Improvements – Short Term To Meet Ammonia Removal Requirements

As discussed previously, the Hill-n-Park WWTF was recently expanded from 0.5 to 0.99 MGD. Effluent limits issued by CDPHE require ammonia removal during certain periods of the year. However, the EPA recently published the final Federal Ammonia Criteria which will result in more stringent effluent ammonia limits in the future.

Driving Forces

- **Future Regulations.** Based on recently published Revised Federal Ammonia Criteria, the treatment system will be required to remove ammonia at lower levels than the current permit.

Alternatives Considered

In order remove ammonia, two alternatives were evaluated. Both alternatives assume the maximum rated flow of 0.99 MGD and an average influent BOD, TSS, and ammonia concentration of 251 mg/L, 245 mg/L, and 32 mg/L (based on Hill-n-Park WWTF data), respectively. Alternatives considered included:

- Alternative HB1 – Convert Aeration Lagoon No. 1 to Complete Mix Through Addition of Mixers
- Alternative HB2 – Install MBBR Nitrification Cell into Aeration Lagoon No. 1

4.7.1.1. Alternative HB1 – Convert Aeration Lagoon No. 1 to Complete Mix Through Addition of Mixers

Preliminary kinetic modeling indicates the levels of dissolved oxygen achievable in the lagoon from the existing six (6) 15 HP surface aerators are sufficient to oxidize the BOD and ammonia, however incomplete mixing prevents this process from occurring consistently throughout the volume. In order to achieve an equivalent of 30 HP/MG of aeration mixing energy, an additional eleven (11) 5-HP mixers are necessary. Installing directional elbows on the mixers allows for placement around the perimeter of the lagoon while still achieving a complete mix. Again, concrete pads would be placed below the new mixers to prevent erosion of the membrane liner. To provide for backup power, a new standby generator would be installed.

Due to the higher TSS loading, it is expected the Polishing Cell would still need to be used to settle out any remaining particulate material. To minimize the amount of algae growth, the Polishing Cell should be covered. There are a number of floating cover products available including membranes to floating balls. Aeration Lagoon No. 2 would remain as is. Due to the age and condition of the existing influent screen, a new unit is required. However, no changes are required to the grit removal, influent pumping, site piping, or disinfection systems. Advantages and disadvantages for this alternative are noted below:

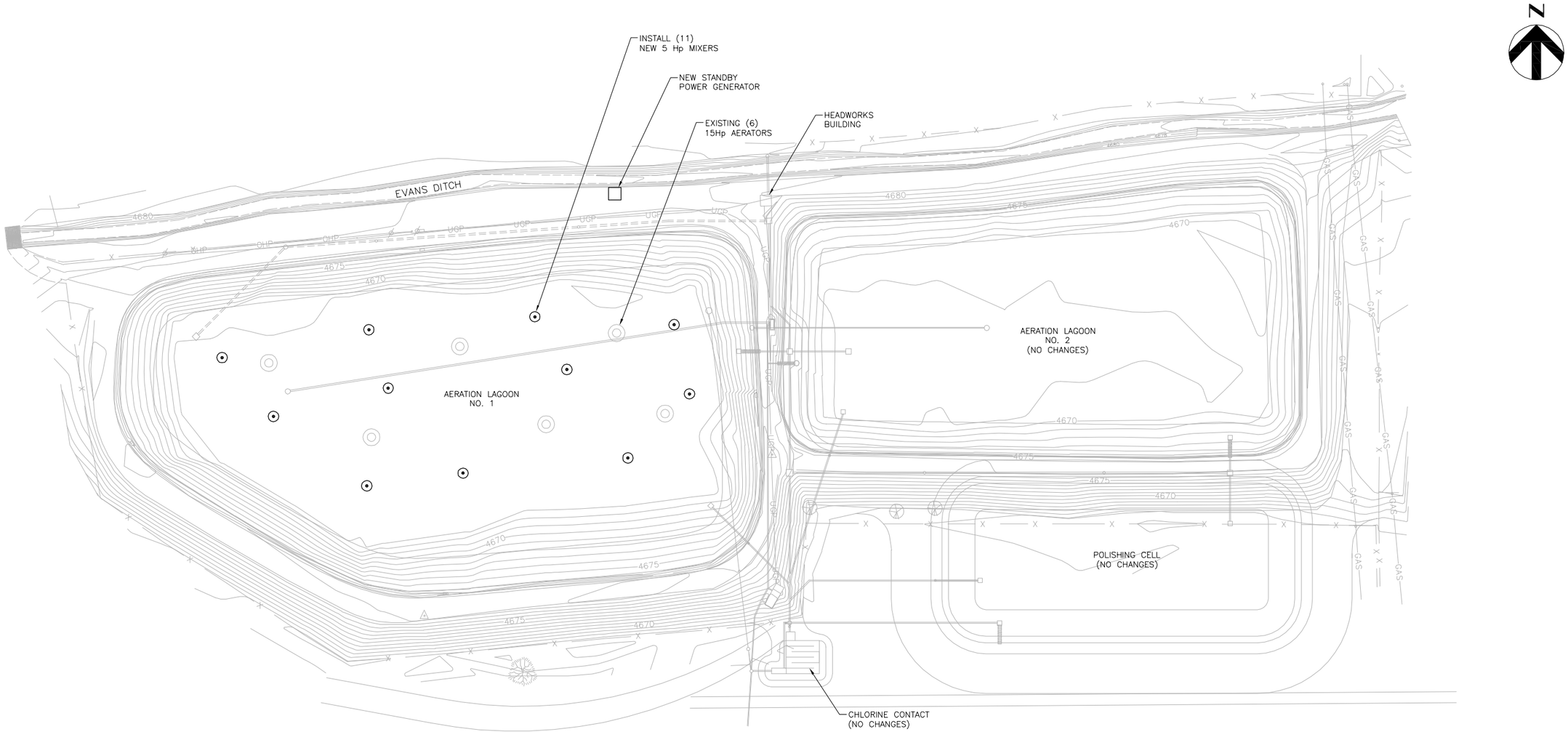
Advantages for this alternative include the following:

- Minimal operator attention.
- Provides adequate ammonia removal to meet current permit conditions.
- Low construction cost.
- Easy retrofit into existing treatment system. Installation of new mixers can be phased to determine exact number required to meet effluent ammonia limits.
- Staff familiarity with existing equipment and operations.

Disadvantages include:

- Low process control – no guarantee as to quality of effluent. Will not meet more stringent effluent ammonia limits.
- Will not assist in removal of nitrate if future permit requires.
- Does not address ammonia release in the Polishing Lagoon associated with anaerobically digested sludge.
- High energy use due to additional mixers.
- Difficult to phase to meet increased flows in the future.
- Due to the large surface area, this alternative does not address the low water temperatures experienced during the winter which leads to a reduction in ammonia removal and unreliable to meet effluent permit requirements.

Figure 4-20 provides a preliminary layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 50'

HDR

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB1
Convert Lagoon No. 1 to Complete
Mix Through Addition Of Mixers

0 1" 2"

FILENAME	FIG_4–20.DWG
SCALE	AS NOTED

SHEET

FIG. 4-20

4.7.1.2. Alternative HB2 – Install MBBR Nitrification Cell into Aeration Lagoon No. 1

This alternative includes constructing an aerated MBBR nitrification basin at the outlet of Aeration Lagoon No. 1. The basin would be constructed of concrete and measure approximately 64 FT long by 64 FT wide by 10 FT deep resulting in a volume of 325,000 gal. The volume provides approximately 6 hours of hydraulic detention time at the 2020 maximum month flow condition. Mixed liquor from the end of Aeration Lagoon No. 1 would flow into the basin where it is mixed and come in contact with floating dispersed media. Coarse air diffusers provide the required oxygen and maintain solids suspension. Retention screens keep the media within the basin. Effluent flows to Aeration Lagoon No. 2 for final treatment and settling. To provide for backup power, a standby generator would need to be installed.

A new air delivery system consisting of two blowers each with a capacity of approximately 900 scfm are required for this alternative. The blowers would be housed in a 20 FT wide by 30 FT long building constructed near the existing Headworks Building. To protect the MBBR media, a new fine screen would be installed in the existing Headworks Building. No retrofit of the existing disinfection system is required. Additionally, Aeration Lagoon No. 2 and the Polishing Cell would remain unchanged. Advantages and disadvantages are as described for the Evans WWTF Alternative EB2 with the addition of the following:

Advantages for this alternative include the following:

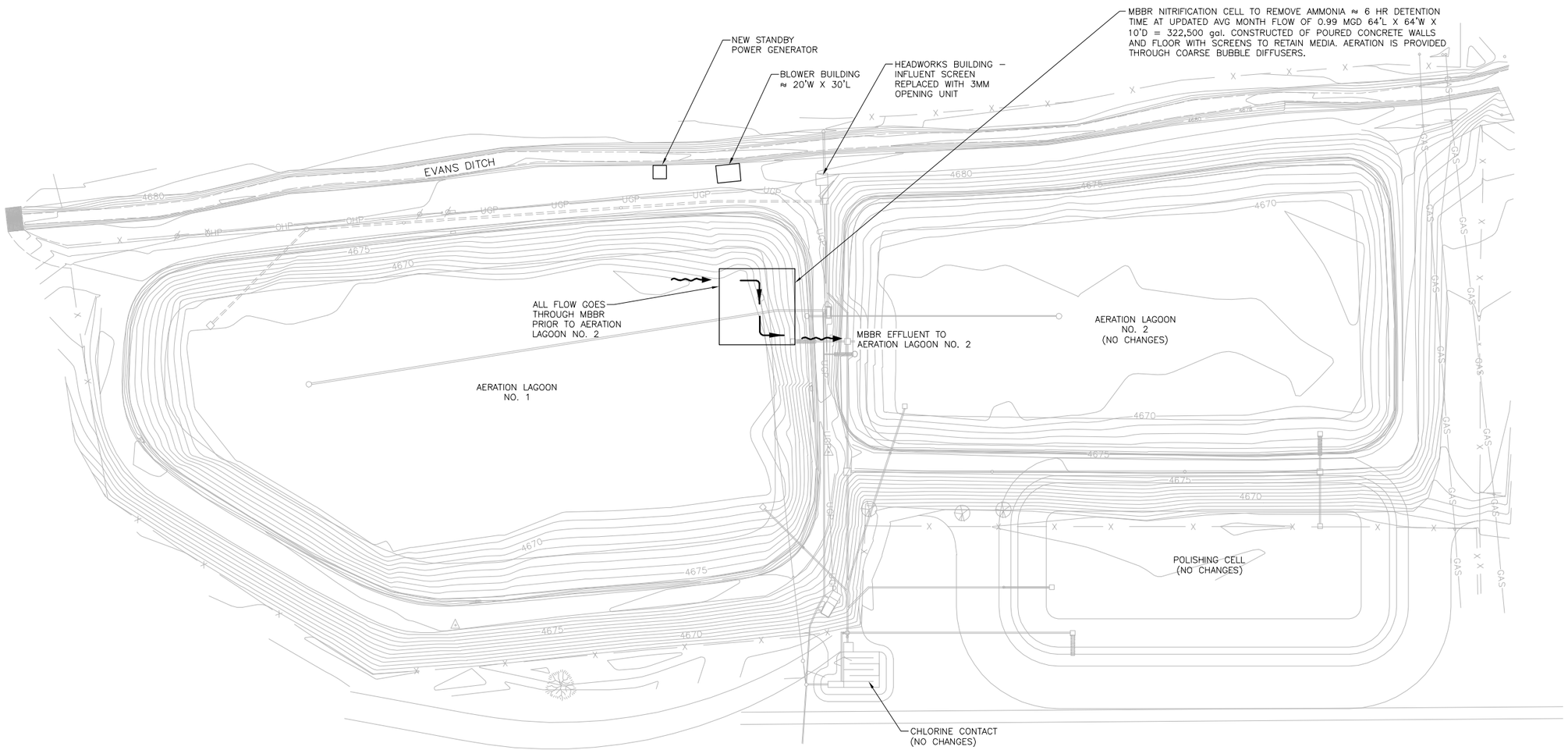
- Minimal operator attention. The media allows for greater bacteria population without the additional solids loading. The higher population provides for a more robust treatment process.
- The attached growth associated with MBBR technology allows for greater ammonia removal even at lower water temperatures.
- Better process control than a standard lagoon treatment system.
- Relatively simple retrofit into the existing process.
- Good mixing through low energy input.
- Allows for future phasing of additional zones for removal of BOD, ammonia and nitrate.
- MBBR system not as susceptible to cold temperatures as standard activated sludge system.
- Benefits of fixed film systems into the suspended growth activated sludge process – resistant to biomass washout during peak flow events.
- Straightforward phasing and expansion to meet increased flows in the future. The maximum fill volume, based on the future flow rate, of media is 60 percent, however, media can be added as needed based on flow.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- Will not assist in removal of nitrate.

- Requires installation of temporary baffles and dewatering for installation of tank.
- MBBR media requires fine screening upstream.
- Does not address ammonia release in the Polishing Lagoon associated with anaerobically digested sludge.

Figure 4-21 provides a preliminary layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 50'

HDR

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ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB2
Construct MBBR Nitrification Cell
Into Aeration Lagoon No. 1

0 1" 2"

FILENAME	FIG_4–21.DWG
SCALE	AS NOTED

SHEET **FIG. 4-21**

4.7.2. Biological Process Improvements – Long Term to Increase Capacity and Meet Total Nitrogen Removal Requirements

For these alternatives, the capacity of the Hill-n-Park WWTF is increased to the 2030 expected flow rate. Additionally, it is anticipated within the next 10 to 15 years effluent permit limits will require the removal of total nitrogen and phosphorus.

Driving Forces

- **Growth.** As discussed in Chapter 2, the Hill-n-Park WWTF must be expanded to handle the growth anticipated over the next 20 years. Based on expected growth rates and land uses, the Hill-n-Park WWTF will need to be expanded to treat an average daily flow rate of approximately 2 MGD by year 2020 and 3 MGD by the year 2030.
- **Future Regulations.** It is anticipated that future regulations will require the removal of total nitrogen from wastewater effluent to less than 5 mg/L, requiring a denitrification process and thus reducing nitrate to nitrogen gas that can be discharged to the atmosphere. This process can only be accomplished in anoxic (without dissolved oxygen) conditions. The anoxic atmosphere allows the bacteria to metabolize biodegradable substrate using nitrate as the electron acceptor. This process also returns some alkalinity and oxygen to the system which is consumed during nitrification. In order to provide the ability for nitrate removal in the future, the ammonia removal alternatives discussed in the previous section should be adaptable to one of the following alternatives.
- **Future Regulations.** It is anticipated that future regulations will require the removal of phosphorus. This process will be accomplished through the use of a combination of biological and chemical addition/ effluent filtration.

Alternatives Considered

In order to increase capacity and add denitrification, eight alternatives were initially evaluated. Three alternatives involve modifying the existing treatment system, four involve constructing a completely new facility, and the last involves decommissioning the facility and pumping flow to the Evans WWTF. All alternatives consider the requirements previously described for ammonia, BOD and TSS removal as well as removing total nitrogen. The removal capacity varies slightly based on the technology and is discussed in the description of each alternative. Alternatives evaluated included:

- Alternative HB3 – Convert Aeration Lagoon No. 1 to Complete Mix Through Addition of Mixers, Add Clarification and Return Flow
- Alternative HB4 – Construct MBBR/IFAS Nitrification and Denitrification Cells In Aeration Lagoon No. 1
- Alternative HB5 – Convert Aeration Lagoon No. 2 to An Aerated and Complete Mix Reactor, Add Clarification and Return Flow
- Alternative HB6 – Demolish Existing Lagoon Treatment System and Construct Medium Rate Activated Sludge (MRAS) System

- Alternative HB7 – Demolish Existing Lagoon Treatment System and Construct Conventional Activated Sludge System
- Alternative HB8 – Decommission the Evans WWTF, Modify Influent Pump Station and Construct Forcemain to Convey All Flow to the Hill-n-Park WWTF, Demolish Existing Hill-n-Park WWTF Lagoon Treatment System and Construct Conventional Activated Sludge System

4.7.2.1. Alternative HB3 – Convert Aeration Lagoon No. 1 to Complete Mix Through Addition of Mixers, Add Clarification and Return Flow

This alternative is an expansion of Alternative HB1 and can be constructed in the future to increase capacity and provide denitrification if a nitrate limit is implemented on the South Platte River. Aeration Lagoon No. 1 would be converted into a more traditional biological nutrient removal process through the use of an anoxic zone, aerobic zone, and mixed liquor and return activated sludge recycle. Additionally, a secondary clarifier would be constructed in the Polishing Cell and Aeration Lagoon No. 2 would be converted to sludge storage and stabilization. Through the inclusion of solids separation and return, the solids retention time can be disconnected from the hydraulic retention time.

For this alternative, the following would be constructed:

- Dredge and line Aeration Lagoon No. 1 (this lagoon is currently not lined).
- Convert approximately 2/3 of the area of Aeration Lagoon No. 1 to an aerobic zone with a volume of 5.4 MG. Install nine new (9) 75-HP aerators and two (2) mixed liquor recycle pumps with a capacity of four (4) times the influent flow.
- Convert approximately 1/3 of the area of Aeration Lagoon No. 1 to an anoxic zone with a volume of 1.35 MG. Relocate nine (9) 5-HP mixers installed as part of Alternative HB1.
- Install geomembrane baffle between the anoxic and the aerobic zones.
- Expand capacity of headworks and add grit removal.
- Construct a Secondary Clarifier with a diameter of approximately 130 FT and a working depth of 15 FT (this would require additional excavation of the Polishing Cell). The clarifier is sized for 100 percent design flow, with redundancy provided by the remaining lagoon volume. A pump located in the clarifier would remove solids and recycle a portion (ranging from half to the entire influent flow rate) back to the beginning of the process. This is referred to as ‘return activated sludge’ or RAS. Periodically (about once per week) solids are wasted to either the remaining volume of the Aeration Lagoon No. 1 or to Aeration Lagoon No. 2 for final stabilization (either aerobic or facultative). This is referred to as ‘waste activated sludge’ or WAS. Effluent from the Secondary Clarifier would flow to the Chlorine Contact Channel for final disinfection prior to discharge to the South Platte River.
- Expand capacity and convert to UV Disinfection System.

- Install standby power.
- Convert Aeration Lagoon No. 2 to a solids storage and stabilization. Relocate existing 15-HP aerators from Aeration Lagoon No. 1.

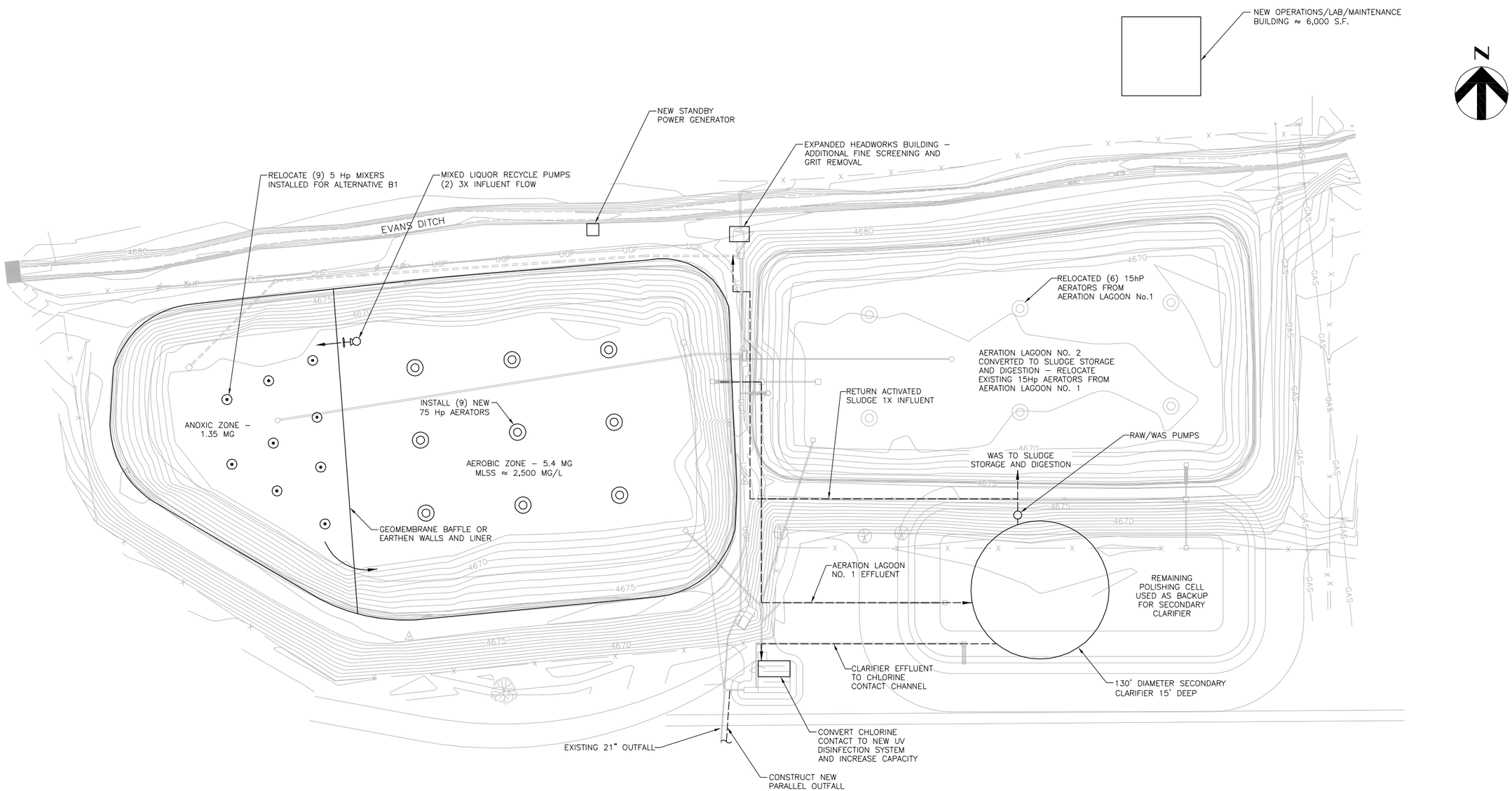
Advantages for this alternative include the following:

- Solids retention time is no longer tied to hydraulic retention time. Allows enough time for nitrifying bacteria growth.
- Very good ammonia and nitrate removal.
- Better process control than a standard lagoon treatment system.
- Makes use of mixers installed in Alternative EB1.
- Minimal operations.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- Does not address low winter time temperatures which reduces growth rate of nitrifying bacteria.
- Does not address ammonia release in the Polishing Lagoon associated with anaerobically digested sludge.
- Difficult to phase expansion based on flow. All nine aerators are required to provide complete mix conditions, however, lower flow rates initially would not make use of all oxygen provided. No way to vary the energy input based on flow rate.
- Difficult to construct while maintaining current treatment capacity. Alternative would require taking Aeration Lagoon No. 1 offline to dredge, line, and install baffles resulting in a temporary permit to discharge effluent which may not meet current NPDES permit.
- Difficult to incorporate biological treatment of phosphorus.
- Alternative makes use of large lagoons resulting is low water temperatures in the winter and inhibiting the growth of nitrifying bacteria.
- High energy cost due to additional mixing and aerators.

Figure 4-22 provides a layout of the proposed system.



OVERALL SITE PLAN

SCALE: 1" = 50'

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HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB3
Convert Aeration Lagoon No. 1 To Complete Mix Through Addition of Mixers, Add Clarification And Return Flow

0 1" 2"

FILENAME	FIG_4–22.DWG
SCALE	AS NOTED

SHEET **FIG. 4-22**

4.7.2.2. Alternative HB4 – Construct MBBR/IFAS Nitrification and Denitrification Cells In Aeration Lagoon No. 1

The MBBR system, as previously discussed under Alternative HB2, has the capability to increase ammonia removal by providing media for nitrifying organisms to grow on. While a minimal amount of denitrification occurs with the MBBR system constructed under Alternative HB2, the system requires modification in order to meet future nitrate limits. Alternative HB4 is the second phase to the system constructed as part of Alternative HB2 for nitrate removal and requires the following:

- Construct two aerobic MBBR/IFAS basins each with a length of 172 FT, width of 64 FT, and depth of 10 FT (each basin has a volume of 0.825 MG). The MBBR basin constructed as part of Alternative HB2 would be incorporated into the new basin. Install media and coarse bubble aeration system. Install mixed liquor recycle pump at the end of the aerobic zone to return flow to the beginning of the anoxic zone. (Size of basin assumes the use of high surface area media with a minimum surface area to volume of 300 FT^2/FT^3. Standard media has a surface area to volume of 150 FT^2/FT^3. If standard media is selected, the size of the basin would increase by a factor of two.)
- Construct two anoxic MBBR/IFAS basins each with a length of 94 FT, width of 64 FT, and depth of 10 FT (each basin has a volume of 0.45 MG). Install media and submerged low speed mixers.
- Install additional blowers in previously constructed blower building as part of Alternative HB2.
- Install floating cover on Polishing Pond to limit algae growth.
- Expand capacity of Headworks and add grit removal.
- Expand capacity and convert to UV Disinfection System.
- Install standby power.
- Construct Secondary Clarifier with a diameter of approximately 130 FT in the future if effluent ammonia limits require concentrations below 2 mg/L. This is due to the anaerobic digestion of the solids in the Polishing Pond and release of ammonia during the summer months.

This alternative can be phased to better match flow by constructing one basin initially with the second basin constructed when influent flow dictates. Additionally, the installation of media can be phased to match flow rate. This alternative can also be converted to an IFAS system as previously described for the Evans WWTF Alternative EB5. This would eliminate the need for the lagoon system for BOD removal. Anaerobic, anoxic, and aerobic zones would be constructed with a secondary clarifier to return suspended bacteria. This decision could be made at a later date based on the actual flows going to the Hill-n-Park WWTF and the regulatory requirements.

Advantages for this alternative include the following:

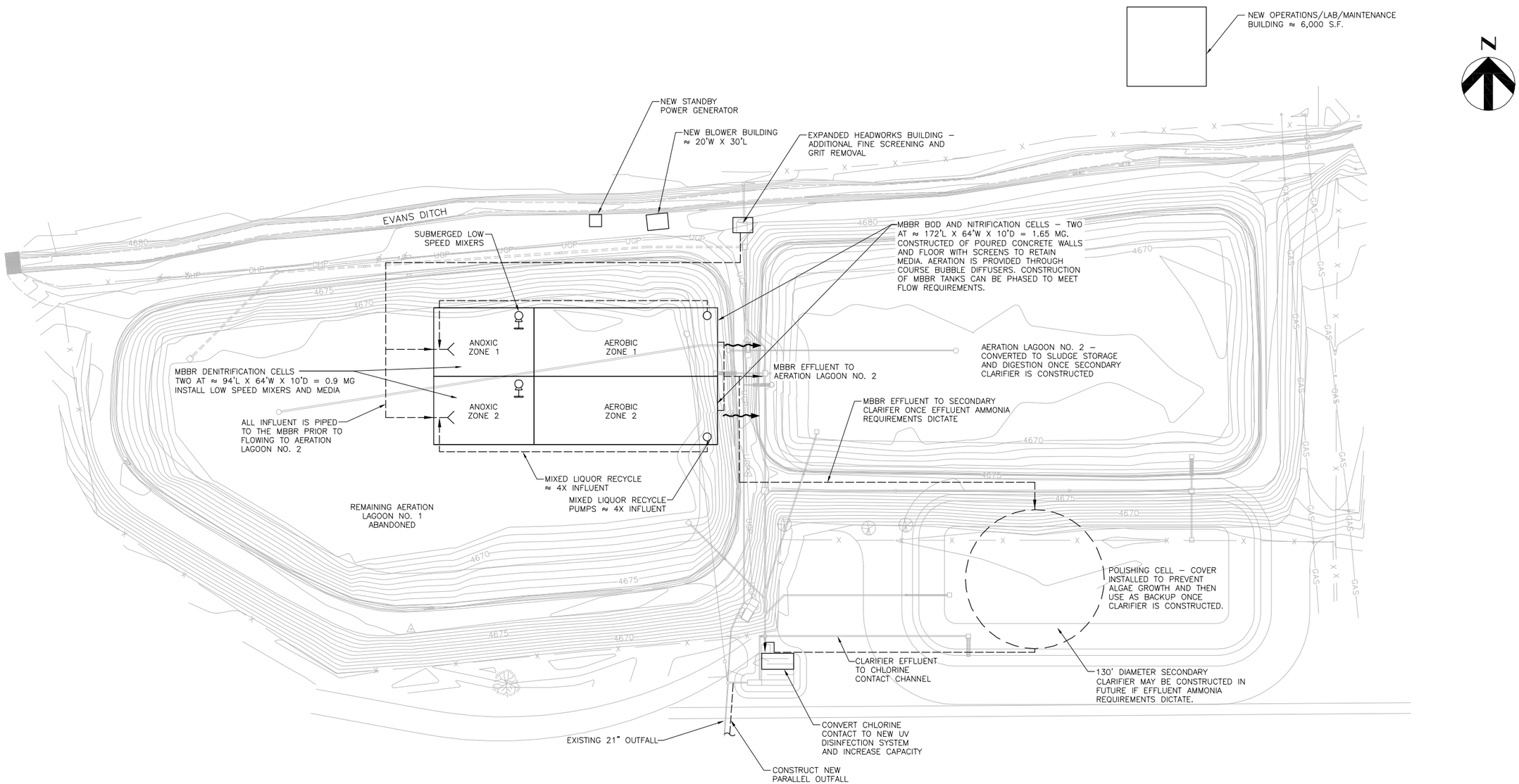
- Minimal operator attention.
- Very good ammonia and nitrate removal.
- Better process control than a standard lagoon treatment system.

- Relatively simple retrofit into the existing process during construction or the installation of a baffle and dewatering. Second phase to Alternative HB2.
- Good mixing through low energy input.
- MBBR/IFAS system not as susceptible to cold temperatures as standard activated sludge system.
- Benefits of fixed film systems into the suspended growth activated sludge process – resistant to shock loads.
- Straightforward phasing and expansion to meet increased flows in the future. The maximum fill volume of media is 60 percent, however, media can be added as needed based on flow.
- An anaerobic zone can be constructed in the future to allow for biological treatment of phosphorus.
- The attached growth associated with MBBR/IFAS technology allows for greater ammonia removal even at lower water temperatures. Additionally, tank size is reduced decreasing the amount of heat loss.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- MBBR/IFAS media requires fine screening upstream.
- May require construction of secondary clarifier in the future if effluent must meet very low ammonia limits.
- Difficult to construct while maintaining current treatment capacity. Alternative would require taking Aeration Lagoon No. 1 offline to dredge, line, and install baffles resulting in a temporary permit to discharge effluent which may not meet current NPDES permit.

Figure 4-23 provides a layout of this alternative.



OVERALL SITE PLAN

SCALE: 1" = 50'

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HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB4
Construct MBBR Nitrification And Denitrification Cells Into Aeration Lagoon No. 1

0 1" 2"

FILENAME	FIG_4–23.DWG
SCALE	AS NOTED

SHEET **FIG. 4-23**

4.7.2.3. Alternative HB5 – Convert Aeration Lagoon No. 2 to An Aerated and Complete Mix Reactor, Add Clarification and Return Flow

This alternative converts the existing Aeration Lagoon system into an aerated and complete mix bioreactor. Based on a future average annual flow rate of approximately 3 MGD, two new aerated bioreactors would be constructed within the footprint of Aeration Lagoon No. 2. Each bioreactor would have a volume of approximately 3.6 MG. Assuming a depth of 12 FT (this requires excavation of the existing lagoon), the bottom footprint should be approximately 200 FT long by 170 FT wide. Effluent from the new bioreactor would flow to a new 130 FT diameter Secondary Clarifier constructed within the existing Polishing Pond footprint. Solids wasted from the system would be stored and stabilized in Aeration Lagoon No. 1.

The bioreactor would be constructed of earth and lined with an HDPE liner to prevent leakage. The clarifier would be constructed of concrete and have a side water depth of 16 FT requiring excavation of the Polishing Cell. A pump located in the clarifier would remove solids from the clarifier and recycle a portion (approximately 0.5 to 1 times the influent flow rate) back to the beginning of the process. Periodically (about once per week) solids would be wasted to Aeration Lagoon No. 1 for final stabilization.

Effluent from the clarifier would flow by gravity to the Chlorine Contact Channel (converted to UV) for final disinfection. Three blowers would be installed in a new blower building to supply air for the biological process. To allow for the increased flow rate, the influent screening system should be expanded and a second channel and screen installed. Additionally, grit removal would be included. The sodium hypochlorite disinfection system also requires expansion.

The following would be required to convert the existing Hill-n-Park WWTF lagoon system to an aerated and complete mix system:

- Dredge, excavate, line, and construct two new bioreactors in Aeration Lagoon No. 2.
- Dredge and excavate Polishing Cell. Construct Secondary Clarifier (remaining volume of Polishing Cell will be used as a temporary backup unit).
- Construct blower building and air distribution header.
- Construct air delivery piping.
- Install RAS and WAS pumping and piping systems.
- Install standby power.
- Expand capacity of Headworks and add grit removal.
- Expand capacity and convert to UV Disinfection System.
- Convert Aeration Lagoon No. 1 to settling and sludge storage lagoon.

Advantages for this option include the following:

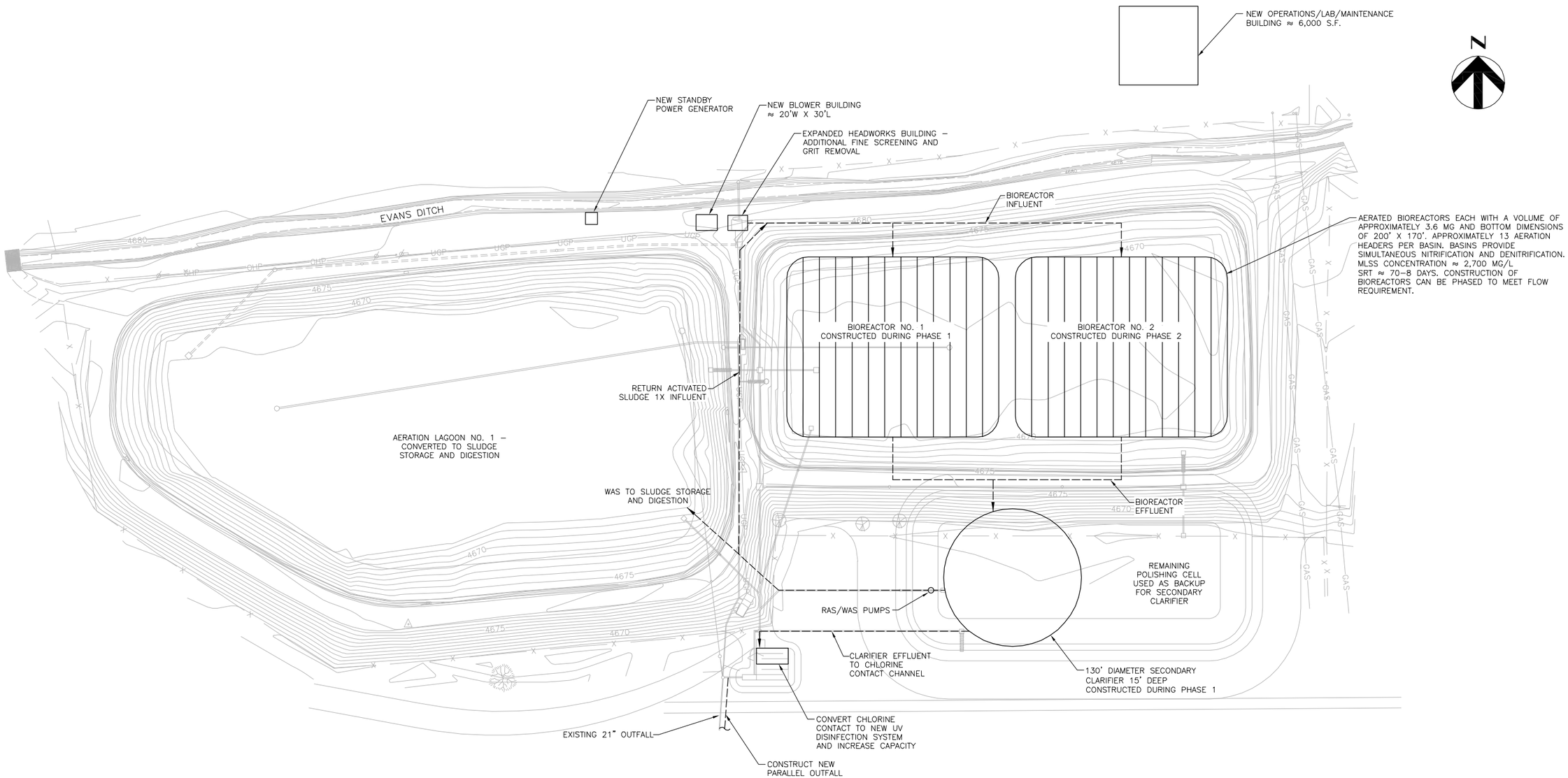
- Minimal operator attention.
- Very good ammonia and nitrate removal.
- Better process control than a standard lagoon treatment system.

- Relatively simple construction and retrofit into the existing process- can be constructed in earthen basins. Requires the shutdown of only Lagoon No. 2 with the remaining treatment process staying in service.
- Good mixing through low energy input.
- Lower energy input than complete mix alternative.
- Compact footprint.
- Ammonia released from anaerobically digested solids does not enter effluent.
- Smaller treatment system surface area decreases heat loss.
- Increased algae control.

Disadvantages include:

- Greater equipment complexity compared to a standard pond treatment system.
- Requires mixed liquor return pumping system.
- Requires return activated sludge pumping system and solids wasting.
- Requires construction of secondary clarifier.

Figure 4-24 provides the general layout for this alternative.



OVERALL SITE PLAN

SCALE: 1" = 50'

HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB5
Convert Aeration Lagoon No. 2 To An Aerated And Complete Mix Reactor, Add Clarification And Return Flow

0 1" 2"

FILENAME	FIG_4–24.DWG
SCALE	AS NOTED

SHEET **FIG. 4-24**

4.7.2.4. Alternative HB6 – Demolish Existing Lagoon Treatment System and Construct Medium Rate Activated Sludge (MRAS) System

Under this alternative, the existing lagoon treatment system would be demolished and a new medium rate activated sludge system, "racetrack style", with anoxic and aerobic zones would be constructed in the footprint of Aeration Lagoon No. 2. The new system would be designed to achieve biological nitrogen removal for plant flows up to the 2030 flow conditions. Aerobic versus anoxic conditions within the basins would be set at approximately 75 percent to 25 percent, respectively. The goal of this treatment system is to achieve effluent total nitrogen concentrations below 10 mg/L. Aeration Lagoon No. 1 would be converted to sludge storage and digestion. A 130 FT diameter secondary clarifier would be installed in the current footprint of the Polishing Cell with the remaining area used for backup as a settling pond in an emergency. Similar to the other long term alternatives, a new blower building would be constructed, the headworks should be expanded to carry a higher flow and grit removal added, and the current chlorine disinfection system would be replaced with a UV system. Waste activated sludge is pumped from the secondary clarifier to Aeration Lagoon No. 1 which would be converted to a sludge storage and digestion basin. Finally, standby power would be added to protect the process from a power failure.

Figure 4-25. MRAS or Racetrack Style

The design of this alternative is similar to a carousel style oxidation ditch, however, unlike a typical oxidation ditch, the operating aerobic solids retention time (SRT) is significantly lower (15 days versus 20 days) and optimized for nutrient removal. The aerobic SRT was designed based on a mixed liquor suspended solids (MLSS) concentration of 4,000 mg/L. The shape of the basin is shown on Figure 4-26.

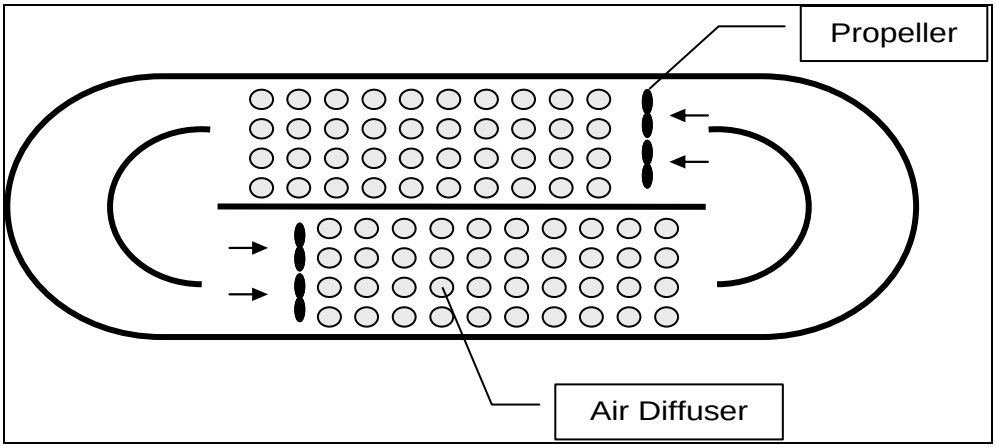


Figure 4-26. Typical Layout of MRAS Alternative

The new racetrack style basin employs a submersible fine bubble aeration grid and submersible mixers in lieu of brush aerators. Diffusers are installed only in certain areas of the basin. This, in combination with the use of propeller mixers, develops a circular flow around the basin passing the flow through distinct zones of aerobic and anoxic conditions. The size of each zone can be customized through the speed control of the mixers and air input to the diffusers. The process can be further optimized for nitrification through the addition of a post aeration basin which would nitrify any residual ammonia. The recirculation of the flow through distinct aerobic and anoxic zones allows for simultaneous nitrification/ denitrification.

Preliminary modeling indicates the volume of the basin is approximately 7 MG resulting in tank dimensions of:

- Length=350 FT
- Width=150 FT
- Depth=20 FT

The overall footprint for this alternative is approximately 52,500 SF. The following is required to convert the existing Hill-n-Park WWTF lagoon system to a conventional activated sludge system:

- Dredge, excavate, and construct new bioreactor in Aeration Lagoon No. 2.
- Dredge and excavate Polishing Cell. Construct new 130-FT Secondary Clarifier (remaining volume of Polishing Cell will be used as a temporary backup unit).
- Construct blower building and air distribution header.
- Construct air delivery piping.
- Install RAS and WAS pumping and piping systems.
- Expand capacity of Headworks and add grit removal.
- Expand capacity and convert to UV Disinfection System.
- Install standby power.
- Convert Aeration Lagoon No. 1 to settling and sludge storage lagoon (either aerobic through the use of the existing mixers or facultative).

Advantages for this option include the following:

- Very good ammonia and nitrate removal through simultaneous nitrification and denitrification.
- Better process control than a standard lagoon treatment system.
- Simple operations.
- Good mixing through low energy input and lower energy input than complete mix alternative.
- Does not require separate mixed liquor pumping.
- An anaerobic zone can be constructed in the future to allow for biological treatment of phosphorus.
- Smaller tank surface area reduced heat loss in the winter.
- Increased algae control.

Disadvantages include:

- Greater equipment complexity compared to a standard lagoon treatment system.
- Requires return activated sludge pumping system and solids wasting.
- Requires construction of secondary clarifier.
- Difficult to phase construction to match flow.
- Difficult to construct while maintaining current treatment capacity. Alternative would require taking Aeration Lagoon No. 2 offline to dredge, and construct new bioreactor resulting in a temporary permit to discharge effluent which may not meet current NPDES permit.

Figure 4-27 provides a process schematic and Figure 4-28 provides the general layout.

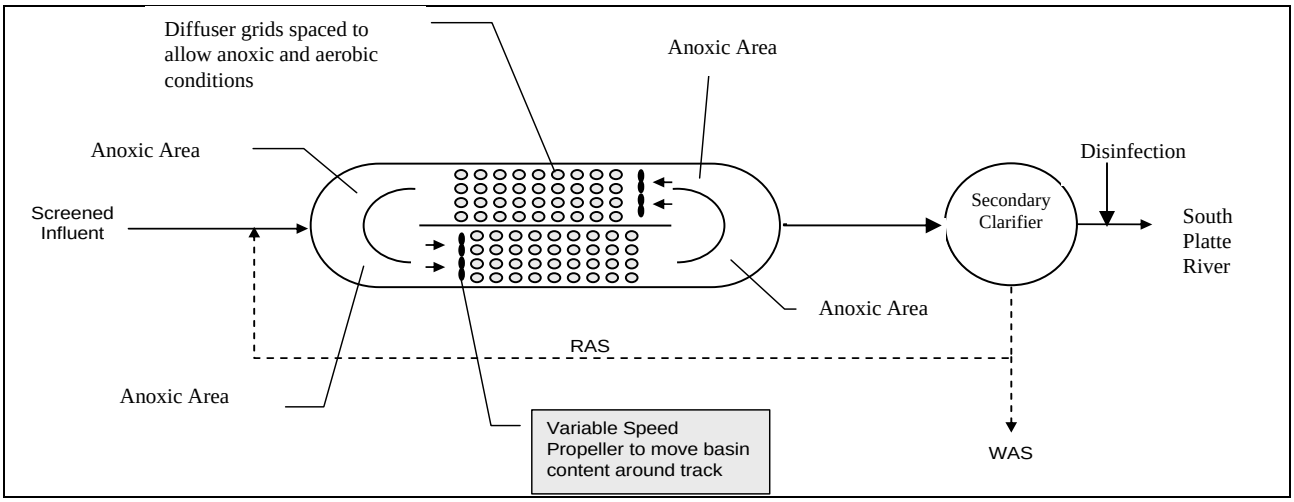
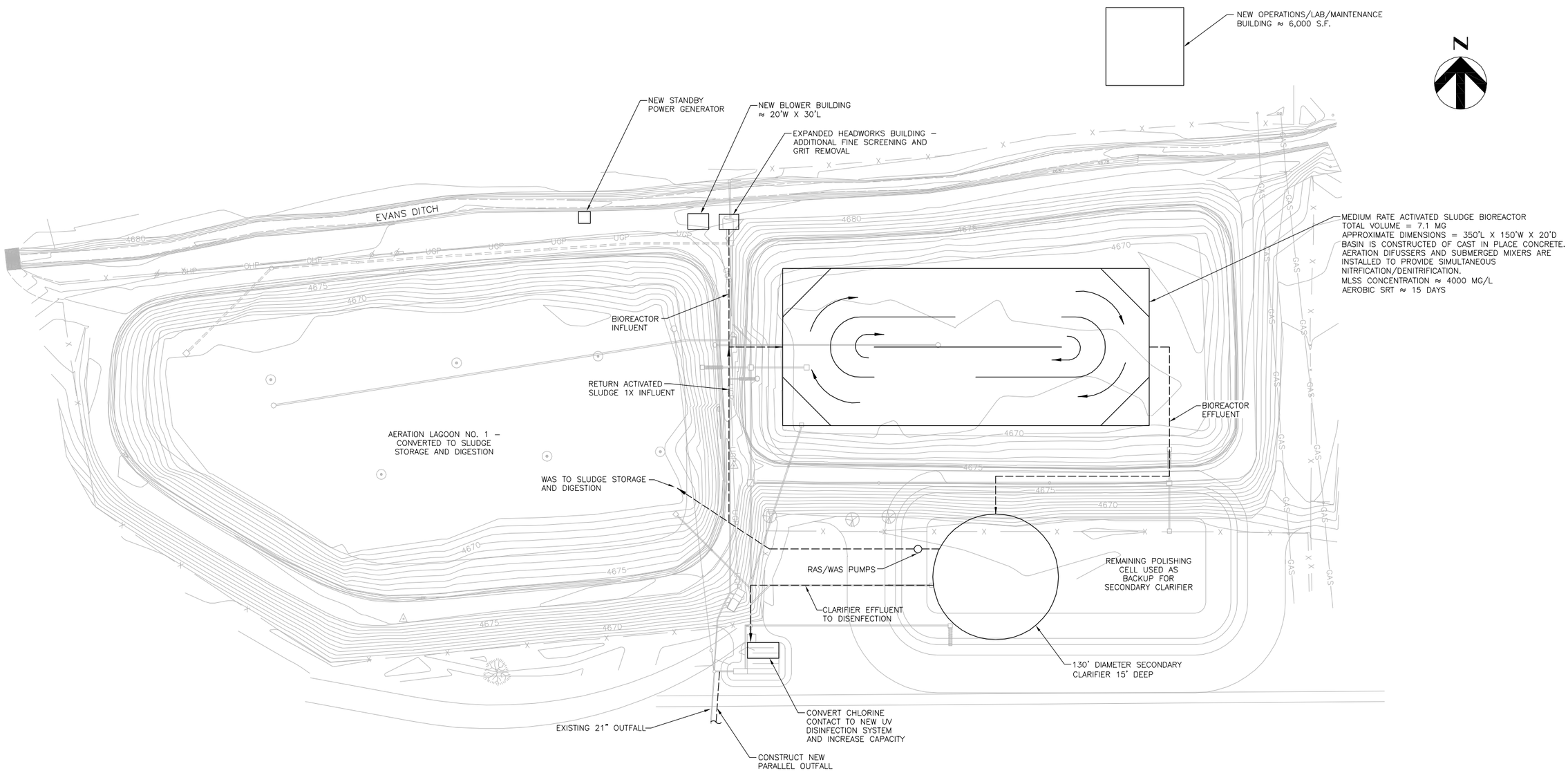


Figure 4-27. MRAS Process Schematic



OVERALL SITE PLAN

SCALE: 1" = 50'

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ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB6
Demolish Existing Lagoon Treatment System And Construct Medium Rate Activated Sludge System

0 1" 2"

FILENAME	FIG_4–28.DWG
SCALE	AS NOTED

SHEET **FIG. 4-28**

4.7.2.5. Alternative HB7 – Demolish Existing Lagoon Treatment System and Construct Conventional Activated Sludge System

For this alternative, a new compartmentalized two stage biological nutrient removal aeration basin with anoxic and aerobic zones would be constructed in the footprint of the existing Aeration Lagoon No. 2. To meet low effluent total nitrogen limits, mixed liquor must be recycled from the end of the aeration basin and combined with aeration basin influent upstream of the anoxic zone. For this alternative, a series of basin compartments or cells are constructed. The benefit of compartmentalizing each zone comes from the kinetic advantages of the plug flow basin configuration and allows for phased construction of the facility.

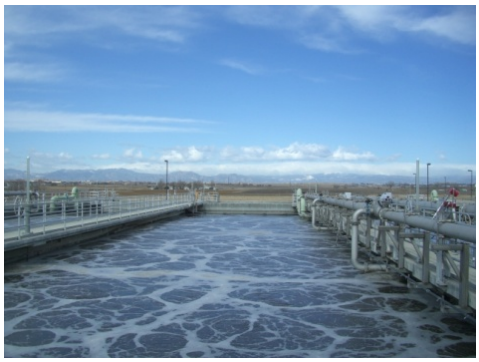


Figure 4-29. Conventional Activated Sludge System

Due to the variation in efficiency of the biological system to remove nutrients during the summer versus winter months (the uptake/nitrification and denitrification are significantly lower during warmer conditions), provision should be made to allow hydraulic retention time maintenance by changing cells from anoxic to aerobic and vice versa.

Preliminary modeling indicates the system would produce effluent nitrogen concentrations below 8 mg/L. To meet the effluent limits discussed previously results in the construction of two tanks with a volume of approximately 2.2 MG resulting in individual tank dimensions of:

- Length=300 FT
- Width=50 FT
- Depth=20 FT

The overall footprint for this alternative is approximately 30,000 SF. The following is required to convert the existing Hill-n-Park WWTF lagoon system to a conventional activated sludge system:

- Dredge, excavate, and construct two new bioreactors in Aeration Lagoon No. 2.
- Dredge and excavate Polishing Cell. Construct 130-FT Secondary Clarifier (remaining volume of Polishing Cell will be used as a temporary backup unit).
- Construct blower building and air distribution header.
- Construct air delivery piping.
- Install RAS and WAS pumping and piping systems.
- Expand capacity of Headworks and add grit removal.
- Expand capacity and convert to UV Disinfection System.
- Install standby power.

- Convert Aeration Lagoon No. 1 to settling and sludge storage lagoon (either aerobic or facultative).

Under this scenario, a new blower building would be constructed, the headworks should be expanded to carry a higher flow and grit removal added, and the current chlorine disinfection system would be expanded or replaced with a UV system. Finally, standby power should be added to protect the process from a power failure.

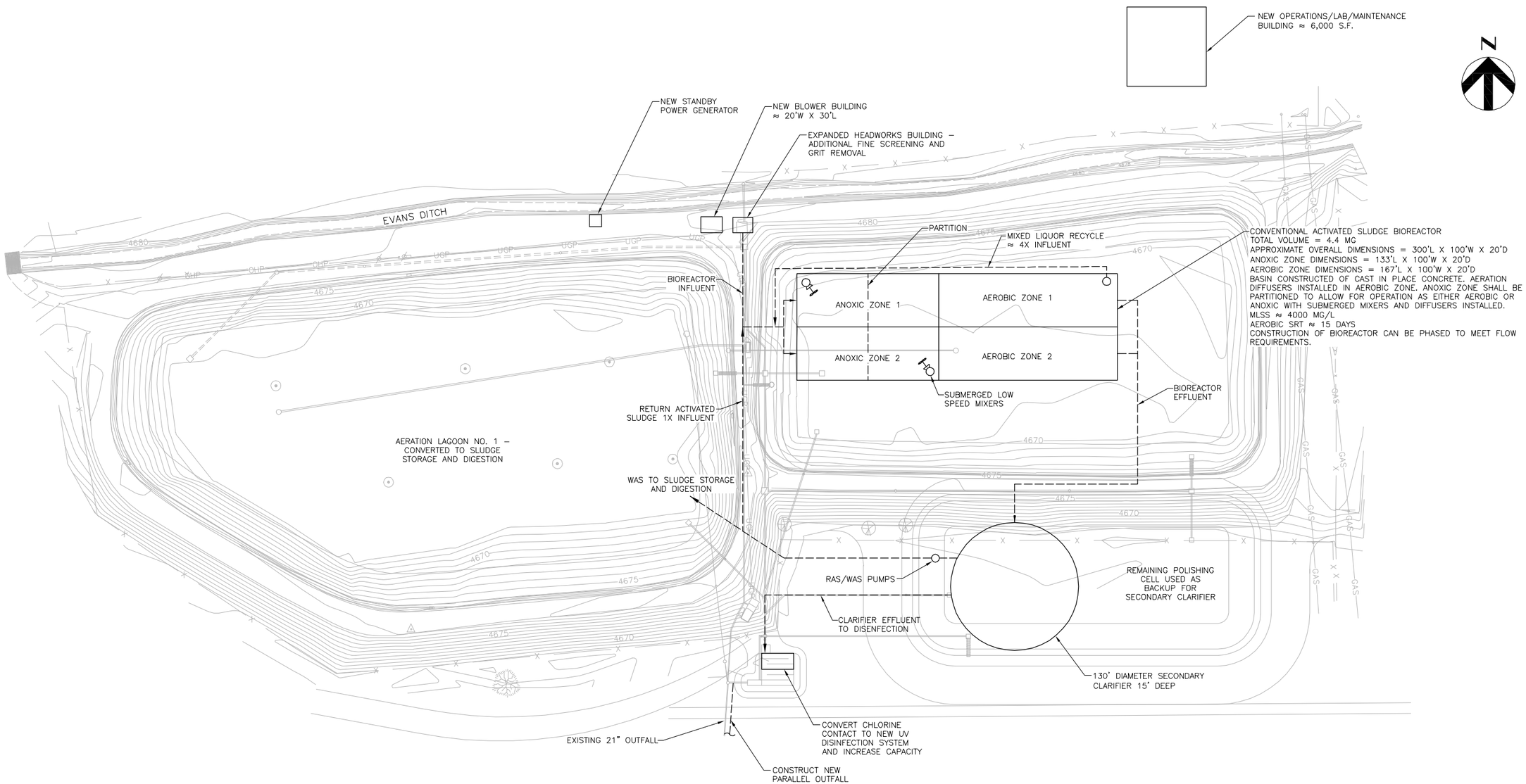
Advantages for this option include the following:

- Very good ammonia and nitrate removal through simultaneous nitrification and denitrification.
- Better process control than a standard lagoon treatment system.
- Good mixing through low energy input and lower energy input than complete mix alternative.
- An anaerobic zone can be constructed in the future to allow for biological treatment of phosphorus.
- Smaller tank surface area reduces heat loss in the winter.
- Standard treatment system for biological nitrogen removal.
- Construction can be phased by constructing one basin at a time to match flow.
- Increased algae control.

Disadvantages include:

- Greater equipment operational complexity compared to a standard lagoon treatment system.
- Requires return activated sludge pumping system and solids wasting.
- Requires construction of secondary clarifiers.
- Difficult to construct while maintaining current treatment capacity. Alternative would require taking Aeration Lagoon No. 2 offline to dredge, and construct new bioreactors resulting in a temporary permit to discharge effluent which may not meet current NPDES permit.

Figure 4-30 provides the general layout for this alternative.



OVERALL SITE PLAN

SCALE: 1" = 50'

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HDR Engineering, Inc.

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER	MATT GOUGH
PROJECT NUMBER	000000000106780

Wastewater Utility Plan

HILL - n - PARK WWTF - ALTERNATIVE HB7
Demolish Existing Lagoon Treatment System And Construct Conventional Activated Sludge System

0 1" 2"

FILENAME	FIG_4–30.DWG
SCALE	AS NOTED

SHEET **FIG. 4-30**

4.7.2.1. Alternative HB8 – Decommission the Evans WWTF, Modify Influent Pump Station and Construct Forcemain to Convey All Flow to the Hill-n-Park WWTF, Demolish Existing Hill-n-Park WWTF Lagoon Treatment System and Construct Conventional Activated Sludge System

For this alternative, treatment facilities are consolidated at the Hill-n-Park WWTF. The Evans WWTF would be decommissioned and the influent Headworks Building/Pump Station modified to convey all current and future wastewater to the Hill-n-Park WWTF. As noted in Table 2-35, average daily and peak hour 2030 flows which must be pumped from the Evans WWTF are approximately 1.432 MGD and 4.087 MGD, respectively. The Headworks Building currently houses three pumps which lift influent flow to Aeration Lagoon No. 1. To allow for pumping to the Hill-n-Park WWTF, the Headworks Building could be retrofitted with new pumps operating at approximately the same flow but at a higher head (due to the higher friction loss). Based on available preliminary alignments, the forcemain between the two facilities is approximately 3.5 miles and could be constructed along the South Platte River to minimize impacts to existing infrastructure.

Similar to the previous alternatives, the existing Hill-n-Park WWTF treatment system must be replaced with an advanced system able to remove nitrogen and treat the additional flow from the Evans WWTF. For the purposes of this discussion, it is assumed a conventional activated sludge system would be constructed in the footprint of the existing Aeration Lagoon No. 2.

4.7.2.1. Alternative HB9 – Demolish Existing Lagoon Treatment System and Construct Membrane Bioreactor System

Due to the cost of construction and operation for this alternative, it has been removed from further evaluation.

4.7.3. Biological Process Improvements – Long Term to Meet Total Phosphorus Removal Requirements

Similar to the Evans WWTF, the Hill-n-Park WWTF is expected to be required to remove total phosphorus. Alternatives are the same as discussed in Section 4.6.3. Pricing for Alternative HB10 is provided in the next section.

4.7.4. Hill-n-Park WWTF Biological Process Improvements Alternative Analysis Summary

4.7.4.1. Cost Evaluation

Estimated capital costs for the Hill-n-Park WWTF biological treatment alternatives are presented in Table 4-19. As stated previously for the Evans WWTF evaluation, costs presented are at the budget estimate level.

Table 4-19. Hill-n-Park WWTF – Biological Process Cost Comparison

Description	Short Term Improvements		Long Term Improvements						
	Alternative HB1	Alternative HB2	Alternative HB3[1]	Alternative HB4[1]	Alternative HB5	Alternative HB6	Alternative HB7	Alternative HB8	Alternative HB10
Base Construction Cost	$375,000	$1,601,000	$3,542,000	$5,015,700	$6,192,000	$6,898,000	$5,899,000	$6,693,000	$986,000
Electrical & Controls (20% of Equipment)	$49,000	$187,000	$263,000	$513,300	$672,000	$334,000	$351,000	$383,000	$69,000
Subtotal A	**$424,000**	**$1,788,000**	**$3,795,000**	**$5,529,000**	**$6,864,000**	**$7,232,000**	**$6,250,000**	**$7,076,000**	**$1,055,000**
Mobilization & Bonds (5%)	$21,000	$89,000	$190,000	$276,000	$343,000	$362,000	$313,000	$354,000	$53,000
Contractor's Overhead & Profit (10%)	$42,000	$179,000	$380,000	$553,000	$686,000	$723,000	$625,000	$708,000	$106,000
Subtotal B	**$487,000**	**$2,056,000**	**$4,365,000**	**$6,358,000**	**$7,893,000**	**$8,317,000**	**$7,188,000**	**$8,138,000**	**$1,214,000**
Miscellaneous Costs Not Itemized (30%)	$146,000	$617,000	$1,310,000	$1,907,000	$2,368,000	$2,495,000	$2,156,000	$2,441,000	$364,000
Subtotal C	**$633,000**	**$2,673,000**	**$5,675,000**	**$8,265,000**	**$10,261,000**	**$10,812,000**	**$9,344,000**	**$10,579,000**	**$1,578,000**
Engineering, Legal, Administration (20%)	$127,000	$535,000	$1,135,000	$1,907,000	$2,052,000	$2,162,000	$1,869,000	$2,116,000	$316,000
Total Project Cost[2]	**$760,000**	**$3,208,000**	**$6,810,000[9]**	**$9,918,000[9]**	**$12,313,000[9]**	**$12,974,000[9]**	**$11,213,000[9]**	**$12,695,000[9]**	**$1,894,000**
Annual Operations Cost[8]	**$76,000**	**$8,000**	**$377,000**	**$90,000**	**$90,000**	**$90,000**	**$90,000**	**$120,000**	**-**
20 Year Capital and Operation Present Worth[3]	**$1,321,000**	**$2,717,000**	**$9,371,000[4]/ $10,692,000[5]**	**$9,128,000[6]/ $11,845,000[7]**	**$9,942,000**	**$10,390,000**	**$9,196,000**	**$10,738,000**	**-**

[1]Cost assumes equipment installed as part of short term improvements is used for long term improvement scenario and is not included a second time.
[2]Cost includes bioreactor improvements and standby generator.
[3]Assumes electrical operations only at $0.08 per kWh. Present worth includes 4% interest rate, and 2% escalation rate. Present worth for Alternatives HB1 and HB2 based on the year 2020. Present worth for Alternatives HB3 through HB8 based on the year 2030.
[4]10 year (2020 through 2030) capital and operations present worth for long term Alternative HB3.
[5]Combined 20 year capital and operations present worth for short/long term Alternatives HB1 and HB3.
[6]10 year (2020 through 2030) capital and operations present worth for long term Alternative HB4.
[7]Combined 20 year capital and operations present worth for short/long term Alternatives HB2 and HB4.
[8]Operation cost assumes electrical cost only.
[9]Cost includes construction of new parallel outfall pipe.

As noted in the previous sections, Alternatives HB1 and HB2 address the short term requirements for ammonia removal. Alternatives HB3 through HB7 increase the capacity of the facility to meet 2030 flow conditions and include nitrogen removal. Alternative HB8 represents the treatment construction costs demolition of the Evans WWTF and transfer of all flow to the Hill-n-Park WWTF for treatment. Alternative HB10 is an adder to all alternatives for the removal of phosphorus.

Alternatives HB1 and HB3 which include the use of surface mixers provide the lowest short term and long term capital expenditure. The cost for meeting both the short and long term solutions is approximately $7,570,000. However, due to the lower efficiency of using surface aerators to provide both mixing energy and process air, a total of 18 mixers are required resulting in approximately 720 horsepower. The 20 year capital and operation present worth for this alternative is the third highest at approximately $10,692,000. This is due to the electrical cost to operate the mixers. Additionally, this alternative will not be able to meet long term nitrogen limits and has been removed from further consideration.

Alternatives HB2 and HB4 make use of MBBR/IFAS technology and results in the highest short and long term capital expenditure with a total cost of approximately $13,126,000. This alternative has a higher combined cost than Alternatives HB1/HB3 but matches the technology proposed for the Evans WWTF and can meet the long term nitrogen and phosphorus limits.

Due to the significant process changes required for construction of Alternatives HB5 through HB7, phasing to meet short term requirements is not possible. Therefore, these alternatives can only be considered for meeting future requirements. Alternative HB5 consists of constructing two new complete mix bioreactors. The cost for constructing this alternative is approximately $12,313,000. Due to the increased oxygen transfer and mixing efficiency of a submerged diffuser system, the yearly blower energy cost is approximately $90,000.

Alternatives HB6 and HB7 consist of replacing the existing earthen lagoon systems with a completely new advanced activated sludge treatment system constructed of concrete tanks. This allows for greater water depth furthering the oxygen transfer efficiency and reducing the blower energy. The resulting power requirement for these alternatives is the lowest at approximately 320 HP. The cost of construction is $12,974,000 and $11,213,000 for Alternative HB6 and HB7, respectively. Yearly energy cost for either alternative is similar to Alternative HB5 at approximately $90,000.

The capital construction cost and 20 year capital and operation present worth for Alternative HB8 is approximately $12,695,000 and $10,738,000, respectively. This alternative takes into account the additional flow from the Evans WWTF which must be treated at the Hill-n-Park WWTF (increasing the size of the treatment facilities). As discussed further in Chapter 5, the capital construction cost and 20 year capital and operation present worth for the pump station and pipeline to convey the flow from Evans WWTF to Hill-n-Park WWTF is approximately $4,000,000 and $3,100,000, respectively. The resulting overall project capital construction cost and 20 year capital and operation present worth for Alternative HB8 is approximately, $16,695,000 and $13,829,000.

4.7.4.2. Criteria Evaluation

In addition to cost, the treatment alternatives were evaluated using the criteria outlined at the beginning of this report. The criteria are weighted from 1 to 3 based on importance to the City and each alternative is then rated on a scale from 1 to 5. Alternatives which meet both short and long term requirements are included together. Table 4-20 summarizes this evaluation.

Table 4-20. Hill-n-Park WWTF – Biological Process Evaluation Summary

Description	Weighting Value	Alternative HB1 and HB3	Alternative HB2 and HB4	Alternative HB5	Alternative HB6	Alternative HB7	Alternative HB8
Regulatory Coordination	3	3/9	5/15	5/15	5/15	5/15	5/15
Operations/ Technology	2	2/4	4/8	4/8	4/8	4/8	5/10
Compatibility with Site	2	3/6	5/10	5/10	4/8	4/8	3/6
Implementation	3	3/9	5/15	4/12	4/12	4/12	3/9
Community/ Environmental	1	2/2	4/4	4/4	4/4	4/4	4/4
Risk	1	3/3	4/4	4/4	4/4	4/4	3/3
Cost (Capital Only)	3	5/15	3/9	4/12	4/12	4/12	2/6
TOTAL		**51**	**65**	**65**	**63**	**63**	**53**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.7.4.3. Hill-n-Park WWTF Biological Process Improvements Recommendation

The combination of Alternatives HB1 and HB3 provides the lowest overall capital expenditure but only meets the needs of the short term requirements. These alternatives would have difficulty meeting future discharge limits which include a nitrate/total nitrogen or phosphorus requirement. Additionally, this alternative requires approximately twice the amount of energy and provides the lowest process control.

Alternatives HB2 and HB4 have a higher combined capital cost but meet both the short and long term requirements. These alternatives also provide the same technology as is recommended for the Evans WWTF.

Alternatives HB5 through HB7 are all similar in treatment process, cost, and operability. The preliminary cost estimate indicates these alternatives are approximately $3,600,000 to $5,500,000 higher than Alternative HB1/HB3, however when factoring in 20 year present worth for capital and operation cost, Alternatives HB5 and HB6 are approximately $2,000,000 lower and Alternative HB7 is approximately $3,000,000 less.

Alternative HB8 proposes to send all flow to the Evans WWTF requiring the construction of a pump station and forcemain. Additionally, immediate improvements to the Evans WWTF would be required to handle the additional flow. Due to the difficultly in phasing this alternative and the high initial cost, it has been removed from further evaluation.

Based on lower operations cost, long term energy use, ability to consistently meet stringent effluent nutrient limits, and to maintain process consistency with the Evans WWTF, Alternatives HB2/HB4 is recommended for implementation to meet both the short and long term requirements. It is possible to phase construction of the improvements and only bring the near term improvements online when additional ammonia removal is required. The system can be expanded in the future to meet lower effluent nitrogen limits by adding additional reactors and secondary clarification. Similar to the Evans WWTF, the Hill-n-Park WWTF can be retrofitted with Alternative HB10 to meet effluent total phosphorus requirements.

4.7.5. Hill-n-Park WWTF Disinfection Improvements

Similar to the Evans WWTF, the Hill-n-Park WWTF disinfection system is designed to utilize liquid chlorine for effluent disinfection. The current permit requirements for disinfection require an *E. Coli* bacteria count of no greater than 986 organisms per 100 mL (30-day average). As noted for the Evans WWTF, future permit limits will likely require *E. Coli* bacteria counts of no greater than 200 organisms per 100 mL.

The disinfection drivers and respective alternatives for the Hill-n-Park WWTF are the same as previously described for Evans WWTF.

4.7.5.1. Disinfection Alternative Analysis Summary

Based upon the projected flow and loadings presented in Chapter 2, the existing chlorine feed system and chlorine contact channels have sufficient capacity for flows up to 1.9 MGD. The disinfection facilities will require expansion to meet the 2030 flow conditions.

Estimated capital and present worth costs for the disinfection alternatives are presented in Table 4-21. The Alternative HD1 will expand the existing chlorine disinfection process and add residual control. The UV light alternative has a lower initial capital cost and provides a more environmentally acceptable solution.

Table 4-21. Hill-n-Park WWTF –Disinfection Alternatives Cost Comparison

Description	Retain Hypochlorite Disinfection and Dechlorination (HD1)	Install UV Light Disinfection (HD2)
Base Construction Cost	$931,000	$710,000
Electrical & Controls (20% of Equipment for Alt D1 and 30% of Equipment for D2)	$30,000	$162,000
Subtotal A	**$961,000**	**$872,000**
Mobilization & Bonds (5%)	$48,000	$44,000
Contractor's Overhead & Profit (10%)	$96,000	$87,000
Subtotal B	**$1,105,000**	**$1,003,000**
Miscellaneous Costs Not Itemized (20%)	$332,000	$301,000
Subtotal C	**$1,437,000**	**$1,304,000**
Engineering, Legal, Administration (20%)	$287,000	$261,000
Total Project Cost	**$1,724,000**	**$1,565,000**
Total O&M Cost/Year	$128,000	$91,000
Total Present Worth[1]	**$2,904,000**	**$2,339,000**

[1]Assumes electrical operations only at $0.08 per kWh. Present worth includes 4% interest rate, and 2% escalation rate.

The disinfection alternatives were also evaluated using the established evaluation criteria. Table 4-22 summarizes the evaluation.

Table 4-22. Hill-n-Park WWTF – Disinfection Alternatives Evaluation Summary

Evaluation Criteria	Weighting Value	Retain Hypochlorite Disinfection and Dechlorination (HD1)	Install UV Light Disinfection (HD2)
Regulatory Coordination	3	2/6	4/12
Operations/Technology	2	3/6	4/8
Compatibility with Site	2	4/8	4/8
Implementation	3	3/9	2/6
Community/Environmental	1	1/1	4/4
Risk	1	2/2	5/5
Cost (Capital Only)	3	4/12	3/9
TOTAL		**44**	**52**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.7.5.2. Disinfection Alternatives Preliminary Recommendations

Although capital costs for installation of UV light disinfection are greater than maintaining chemical disinfection, there are considerable advantages for the City to consider installation of UV light disinfection. UV light is less susceptible to market fluctuations for chemicals, carries less risk for toxic chemical spill and provides greater assurance that future regulatory requirements will be met. Since it is likely the existing chlorination system will need to be modified, it is recommended the City consider installation of UV disinfection to alleviate capacity limitations and to meet future, more stringent, permit requirements.

4.7.6. Biosolids Management

The purpose of this section is to explore options for biosolids disposal from the Hill-n-Park WWTF assuming one of the advanced biological treatment alternatives discussed previously is constructed in the future.

Driving Forces

- **Growth.** The predicted increase in future solids loading demands may require changes to the existing biosolids handling process.
- **Cost.** The current cost of biosolids disposal is relatively inexpensive. In the future, available land required may not be available to meet the solids loading demand resulting in the City paying significantly more for removal and disposal.

Alternatives Considered

Alternatives considered for improvements to the Biosolids Management facilities are:

- Alternative HBM1 - Maintain Liquid Application to Agricultural Land.
- Alternative HBM2 - Dewater and Compost.

4.7.6.1. Alternative HBM1 – No Action - Maintain Liquid Application to Agricultural Land

Biosolids from the Hill-n-Park WWTF are currently dredged and land applied on agricultural fields every 5 or so years. This type of solids stabilization and biosolids management works very well for small lagoon treatment systems. However, in the future, flow to the Hill-n-Park WWTF in expected to increase significantly. To maintain the same treatment system and keep up with future solids loading, solids will require removal on an annual or bi-annual basis.

4.7.6.2. Alternative HBM2 – Dewater and Create Class A Product

Similar to the Evans alternative, sludge dredged from the storage and stabilization lagoon is dewatered and then either pasteurized or composted to produce biosolids that meet Class A requirements. For the purposes of this evaluation we have considered the use of a screw press in which steam could be injected for heating and pasteurizing biosolids. The equipment would be housed in a new dewatering building. Stabilized solids would be pumped from the lagoon to the dewatering building and then stored prior to disposal.

4.7.6.3. Cost Evaluation

This section summarizes the assumptions and possible unknowns related to cost items and present an opinion of probable project cost for each alternative. The comparison is concluded with an analysis of the probable cost of each alternative related to the overall benefit associated with each modification.

Assumptions used in the probable cost estimate for each alternative include:

- Excavated material can be disposed of on site and incurs no additional cost
- Percentage values are included for miscellaneous mechanical and electrical needs

Capital costs were estimated for the biosolids management alternatives evaluated. A summary of the costs are shown in Table 4-23.

Table 4-23. Hill-n-Park WWTF – Biosolids Management Cost Comparison

Description	Alternative HBM2
Base Construction Cost	$994,000
Electrical & Controls	$78,000
Subtotal A	**$1,072,000**
Mobilization & Bonds (5%)	$54,000
Contractor's Overhead & Profit (10%)	$107,000
Subtotal B	**$1,233,000**
Miscellaneous Costs Not Itemized (30%)	$370,000
Subtotal C	**$1,603,000**
Engineering, Legal, Administration (20%)	$321,000
Total Project Cost	**$1,924,000**

4.7.6.4. Criteria Evaluation

In addition to cost, the alternatives were evaluated using the criteria outlined at the beginning of this report. The criteria are weighted from 1 to 3 based on importance to the City and each alternative is then rated on a scale from 1 to 5. Alternatives which meet both short and long term requirements are included together. Table 4-24 summarizes this evaluation.

Table 4-24. Hill-n-Park WWTF – Biosolids Management Alternatives Evaluation

Description	Weighting Value	Alternative HBM1	Alternative HBM2
Regulatory Coordination	3	2/6	4/12
Operations/Technology	2	4/8	3/6
Compatibility with Site	2	4/8	2/4
Implementation	3	4/12	2/12
Community/Environmental	1	2/2	4/4
Risk	1	2/2	4/4
Cost	3	5/15	2/6
TOTAL		**53**	**47**

Example: 4/12 – 4=rating, 12=rating*weighting value

4.7.6.5. Recommendations

At this point in time, it is recommended the current biosolids management (Alternative HBM1) be maintained. However, in the future, regulations or growth may dictate looking at an alternative method for disposal. Biosolids management should be reviewed every 5 years.

4.7.7. Hill-n-Park WWTF Support Facilities

Support Facilities, in this evaluation, comprise electrical power supply and standby power. Additionally, the Hill-n-Park WWTF does not have an operations and maintenance building.

Driving Forces

- **Reliability.** Need to provide backup power and operational facilities at the Hill-n-Park WWTF.
- **Improve Process.** Biological nutrient removal processes require a stable environment. Loss of power and resultant loss of air supply can result in process upsets. Additionally, standby power is required to provide for continuous disinfection during a power outage.

4.7.7.1. Standby Power

Installation of a standby power generator is recommended for critical equipment required to maintain the biological process and to power the proposed administration/ operations building. It is recommended an in-place engine generator with an automatic transfer switch be installed. The standby generator should be capable of operating all primary equipment and be provided with an automatic transfer switch to provide for un-manned restart of equipment in the event of a power outage. It would be installed on a reinforced concrete equipment pad with seismically rated vibration isolators. It would be located outdoors with Non-Walk-In Weather Protective Enclosure with Sound Attenuation. The

diesel storage tank should have a minimum storage capacity to allow full load operation of the generator for 24 hours.

The standby power generator would likely not be used often. Therefore, it would need to be exercised approximately weekly. Exercising should include turning the generator on and running at full capacity for a period of time determined by the manufacturer. Size of the standby generator should be determined during pre-design but based on the size and loads in the facility will likely have a capacity between 1,000 and 1,500 kW. Estimated construction cost for this alternative is approximately $500,000.

4.7.7.1. Operations/Laboratory/Maintenance Building

Currently, the operations and maintenance facility for both WWTFs is located at the Evans WWTF. As discussed in Chapter 2, it is anticipated the largest percentage of increased flow will be directed to the Hill-n-Park WWTF. Within the next 10 to 15 years, the Hill-n-Park WWTF will have almost twice the flow of the Evans WWTF. As such, an operations and maintenance facility will be required at the Hill-n-Park WWTF. To meet the additional operational needs, a dedicated administration/operations and maintenance building will be required. Key support facilities include the following:

- Maintenance facilities
- Operations personnel offices
- Laboratory facilities

To provide the appropriate space for the above facilities, it is estimated a minimum of 6,000 sf is required. Below is a summary of the key design features for the new building. Estimated construction cost for this alternative is approximately $1,800,000.

<u>Maintenance Facilities</u>

The new administration/operations building should have a dedicated area for performing maintenance and storage of equipment for both facilities. Key features include:

- Worktop with tool storage
- Portable equipment hoist
- Storage shelving
- Dedicated chemical and fuel storage area

<u>Operations Facilities</u>

Key features of the operations area include:

- Offices/workspaces
- Mechanical room
- SCADA room for operation of both facilities
- Conference/training room
- Kitchen attached to the conference/training room with capability of separation with a folding divider wall
- A small conference room
- Restroom with dedicated women’s locker room and men’s locker room
- Plan library; copy room

- Electrical/telephone/server room
- Janitorial closet

Additionally, the operations area should incorporate numerous windows and the use of natural light to the greatest extent possible.

Laboratory Facilities

As mentioned previously future effluent permit requirements are expected to continue to become more stringent requiring additional testing. The lab facilities located at the Evans WWTF are not designed to provide adequate room or tools to allow the operations staff to test for these upcoming limits. A new lab facility will be needed.

4.8. Recommendations

Based on capacity limitations, capital and operations and maintenance cost considerations, and the need to meet upcoming effluent permit limits, the Phase 1 (completed within 5 to 10 years- see schedule at the end of this Chapter) recommendations include the following:

- Evans WWTF –
  - Existing Polishing Pond will be dewatered, dredged, excavated and lined with a geomembrane liner.
  - A portion of Aeration Lagoon No. 2 will filled in and an MBBR tank constructed.
  - Use Polishing Pond as secondary clarifier.
  - Construct blower building and air distribution header to new MBBR.
  - Construct air delivery piping in new MBBR.
  - Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
  - Replace standby generator with larger unit.
  - Install UV disinfection.
  - Construct parallel outfall to allow for passage of the peak hour flow.
- Hill-n-Park WWTF –
  - Install new influent screen.
  - Install standby power.

The Phase 2 improvements (completed within 10 to 20 years) include the following:

- Evans WWTF –
  - MBBR will be expanded through the construction of an anaerobic zone and anoxic zone upstream of the MBBR tanks and additional aerobic zones downstream. The MBBR tanks will be converted to IFAS tanks through the addition of suspended growth bacteria through return activated sludge. The anaerobic zone will provide

for biological phosphorus removal while the anoxic tank will provide denitrification to convert nitrate to nitrogen gas.
    - A new secondary clarifier will be constructed in the Polishing Pond to allow for the recycle of suspended growth bacteria to the IFAS tank.
    - New effluent filtration will be constructed to remove additional phosphorus.

- Hill-n-Park WWTF –
    - A new secondary clarifier would be constructed in the Polishing Lagoon.
    - A portion of Aeration Lagoon No. 2 will filled in and an IFAS tank constructed.
    - Construct blower building and air distribution header to new IFAS.
    - Construct air delivery piping in new IFAS.
    - Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
    - Construct new effluent filtration to meet effluent total phosphorus limit.
    - Construct parallel outfall to allow for passage of the peak hour flow.
    - Expand capacity of Headworks and add grit removal.
    - Construct Operations/Laboratory/Maintenance Building.

Following is a summary of the recommended improvements.

4.8.1. Evans WWTF Phase 1 Improvements

4.8.1.1. Alternative EB2 – Construct MBBR Treatment System

The existing Evans WWTF is currently above 90 percent of rated capacity and has had difficulty at times removing ammonia during the winter and spring months. Additionally, the facility will not be able to meet the ammonia limit associated with the rerating nor the future regulations for nitrogen and phosphorus. Based on capital cost, long term energy use, and overall constructability, Alternative EB2 is recommended for implementation to meet both the short term requirements. To meet the long term requirements, the system will be expanded and converted to an IFAS through the addition of a secondary clarifier. This alternative requires a greater initial investment to meet the short term ammonia limits, however, it provides a much lower overall operations cost than the other alternatives and can be constructed without taking the existing treatment facility offline. Additionally, Alternative EB2 provides a higher degree of operational control and can be retrofitted in the future to meet effluent phosphorus limitations, if and when required. The alternative consists of the following:

- Existing Polishing Pond will be dewatered, dredged, excavated and lined with a geomembrane liner.

- A portion of Aeration Lagoon No. 2 will filled in and an MBBR tank constructed.
- Use Polishing Pond as secondary clarifier.
- Construct blower building and air distribution header to new MBBR.
- Construct air delivery piping in new MBBR.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Replace standby generator with larger unit.
- Install UV disinfection.
- Construct parallel outfall to allow for passage of the peak hour flow.

The estimated project cost for constructing Alternatives EB2 is approximately $4,040,000.

When the future regulations require the removal of total nitrogen and phosphorus, Alternative EB5 and EB10 will also be constructed. It will consist of the following:

- MBBR will be expanded through the construction of an anaerobic zone and anoxic zone upstream of the MBBR tanks and additional aerobic zones downstream. The MBBR tanks will be converted to IFAS tanks through the addition of suspended growth bacteria through return activated sludge. The anaerobic zone will provide for biological phosphorus removal while the anoxic tank will provide denitrification to convert nitrate to nitrogen gas.
- A new secondary clarifier will be constructed in the Polishing Pond to allow for the recycle of suspended growth bacteria to the IFAS tank.
- New effluent filtration will be constructed to remove additional phosphorus.

The estimated project cost for constructing Alternatives EB5 and EB10 is approximately $4,224,000.

4.8.1.2. Disinfection Improvements – Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection

The existing sodium hypochlorite disinfection system is not sized to treat flows above 1.2 MGD and will require expansion and/or replacement to treat peak hour flows once the biological improvements have been completed (currently the treatment lagoons act as flow equalization). Although capital costs for installation of UV light disinfection are greater than upgrading the chemical disinfection system, there are considerable advantages for the City of Evans to consider installation of UV light disinfection. UV light is less susceptible to market fluctuations for chemicals, carries less risk for toxic chemical spill and provides greater assurance that future regulatory requirements are met. Since capacity to the existing chlorination system will need to be increased, it is recommended the City construct Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection System to alleviate capacity limitations and to

meet future, more stringent, permit requirements. The alternative consists of the following:

- Two parallel UV channels will be installed in the existing chlorine contact channel.
- Level control facilities, lamp removal hoists and monorails, and lamp cleaning systems will be installed.
- A portion of the existing chlorine storage room will be used for UV lamp maintenance and cleaning purposes, and for installation of the UV electrical service equipment.
- A building will be constructed over the UV channels to provide for weather protection and equipment maintenance.

The estimated project cost for constructing the improvements to the disinfection system is approximately $643,000.

4.8.1.3. Overall Evans WWTF Improvements

Based on the recommended alternatives, the overall project cost for the Evans WWTF Phase 1 improvements is approximately $4,683,000. Table 4-25 provides a summary of the annual cost expenditures.

Table 4-25. Evans WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 15-16
Phase 1 Improvements			
Biological Process Improvements[2]	-Capacity above 90% -Ammonia limits	Design $673,000	Construction $3,367,000
UV Disinfection	-Construction of biological process improvements	Design- $107,000	Construction- $536,000
	Totals	**$780,000**	**$3,903,000**
	Overall Project	**$4,683,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, construction of parallel outfall, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

Future population and flow projections were developed in Chapter 2. Plotting this information along with growth and comparing to the Evans WWTF capacity illustrates the capacities needed at different intervals in the future.

Chart 4-1. Evans WWTF Future Flow and Plant Staging Projection

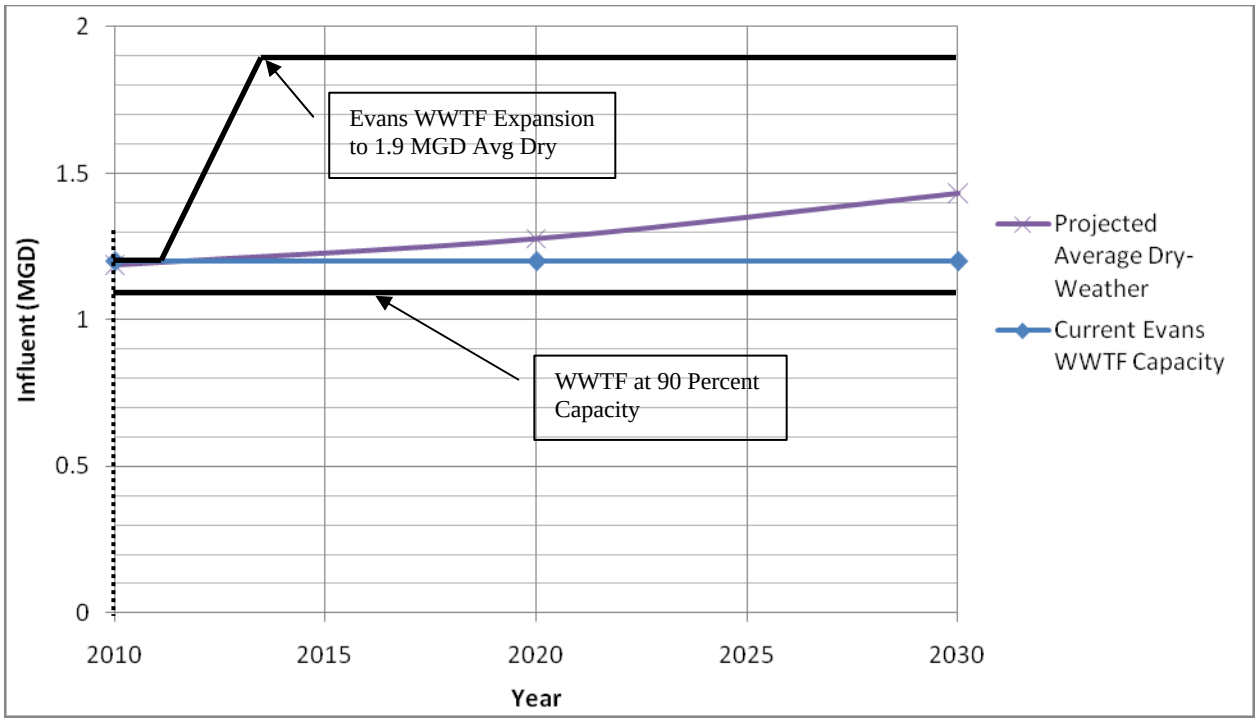


Table 4-26 below provides a preliminary schedule for design, permitting, and construction of the Evans WWTF improvements.

Table 4-26. Evans WWTF Phase 1 Improvements Schedule

Item	Start Date	Completion Date	Notes
Preliminary Engineering Report (PER)	June 20, 2014	July 15, 2014	
Submit Site Application and PER	July 15, 2014		
Site Application Review	July 15, 2014	September 15, 2014	6-8 week review time
Preliminary Design Report (PDR)	September 15, 2014	October 14, 2014	Suggest that Site Application be approved prior to PDR Submittal
Submit for SRF Funding	October 14, 2014	November 18, 2014	30 day review
Project Funding Approved	November 18, 2014		
Final Design	January 9, 2015	April 27, 2015	
Construction	June 4, 2015	June 3, 2016	

4.8.2. Hill-n-Park WWTF Phase 1 Improvements

Due to the lower flow at the Hill-n-Park WWTF and the recent rerating to 0.99 MGD, only a few improvements are required for Phase 1 including replacement of the existing headworks equipment and addition of standby power. The overall project cost for the Hill-n-Park WWTF Phase 1 improvements is approximately $424,000. Table 4-27 provides a summary of the annual cost expenditures.

Table 4-27. Hill-n-Park WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 14-15
Phase 1 Improvements			
Hill-n-Park WWTF Headworks Screening Improvements	-Existing equipment is at end of useful life	Construction $200,000	
Hill-n-Park WWTF Standby Power	-Required to meet regulatory requirements		Construction $224,000
	Totals	**$200,000**	**$224,000**
	Overall Project – Phase I	**$424,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, headworks, construction of parallel outfall, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

5.0 Conveyance Facilities Evaluation & Alternatives Development

5.1. Introduction

This chapter documents the evaluation of existing conveyance facilities and buildout improvement alternatives using a hydraulic model of the sewer system. The first portion of the chapter includes a summary of collection system features, model development, validation, and result review; and establishment of criteria by which the system is analyzed. The summary of system features aids in the general understanding of the City of Evans' wastewater collection system. A solid model build and validation is the key to achieve the desired result confidence level. Evaluation criteria are established for reviewing the model results to identify problem areas.

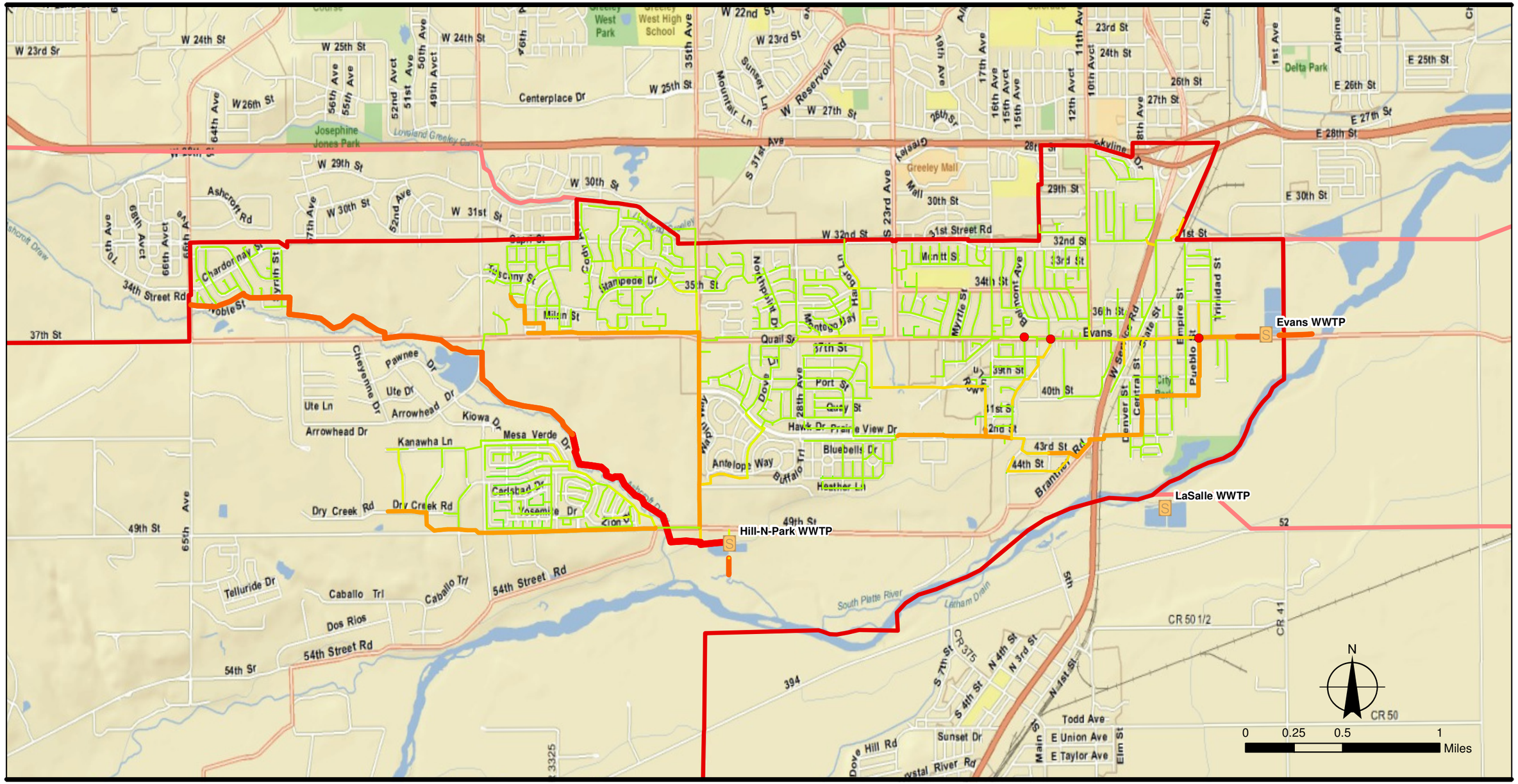
The second portion contains the analysis of the existing collection system with dry-weather and wet-weather flows and identification and characterization of hydraulic capacity issues. By analyzing the existing collection system under existing and buildout conditions against the established system analysis criteria, problem areas can be identified in both conditions. Grouping and characterizing the system problems sets the stage for identifying and prioritizing system improvements.

The third portion of the chapter documents the improvement alternatives and recommendations, outlines the costs for the recommended plan, and establishes an implementation plan based on priority. Three system-wide development alternatives have been developed to make up the Long Term Service Area Plan. A recommended plan with capital costs was established based on the 20-year planning horizon. Improvement projects are scheduled by year to distribute the costs over time and determine sanitary infrastructure priorities with an implementation plan.

5.2. Wastewater Collection and Conveyance System

The proposed Wastewater Utility Service Area (WUSA) contains approximately 8,873 acres which are served by 75 miles of existing sanitary sewer pipes that are divided into two sewer basins. The existing sewer collection system consists entirely of gravity sewer pipes that convey flow to the Evans and Hill-n-Park Wastewater Treatment Facilities (WWTF). There are several diversion manholes which direct flow through various collector systems. Major features and gravity sewer by pipe size that compose the existing system are depicted in Figure 5-1.

The existing City of Evans collection and conveyance system consists of gravity sewers ranging in size from 6-inch to 30-inch. Currently, the system does not have any lift stations or force mains. Gravity piping generally follows the roads within the City limits.



Legend

- Treatment Plants
- Diversion Manhole

Existing Pipes (Diameter)

- 0 - 6
- 6 - 8
- 8 - 10
- 10 - 12
- 12 - 15
- 15 - 24
- 24 - 30

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Existing System
Major Features and Pipe Sizes

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Figure 5.1
July 18th, 2011

5.2.1. Gravity Sewers

The collection system consists of 6-inch to 30-inch gravity sewer mains that follow the topography of the city. Generally, sewers located east of 29th Ave flow to the Evans WWTF with the remaining portion of the western section of the City flowing to the Hill-n-Park WWTF. Table 5-1 summarizes the major features found in the sanitary sewer collection system contained in the WUSA by sewer basin.

Table 5-1. City of Evans Collection System

Gravity Sewer Size	Length (Lineal feet of pipe, number of manholes)
Evans Sewer Basin	
6-inch	1,000
8-inch	180,790
10-inch	8,380
12-inch	17,370
15-inch	11,500
24-inch	1,690
Manholes (Standard)	889
Manholes (Diversion)	3
Hill-n-Park Sewer Basin	
6-inch	6,260
8-inch	112,500
10-inch	14,130
12-inch	1,950
15-inch	19,500
24-inch	13,240
30-inch	6,130
Manholes (Standard)	761
Manholes (Diversion)	0

The gravity mains vary in material throughout the system and generally based on size and age. Most of the pipe materials are known, however a portion of the system, mostly local, is of unknown material. The known pipe materials in the City of Evans' sewer collection system include A2000 PVC, clay, ductile iron (DIP), and various materials that have been repaired by sliplining. The majority of the system, 48.9 miles is PVC pipe.

Gravity sewers are typically classified as local, collector and interceptor sewers. Local sewers have diameters that are typically less than 10-inch and convey wastewater from relatively small service areas (less than about 20 acres). Local sewers have numerous service line connections collecting wastewater from individual customers. Collector sewers have diameters that typically range between 12-inch and 21-inch. Collector sewers convey flow from multiple local sewers and also include individual service line connections; although not as many as local sewers. According to the Northern Front Range Water Quality Planning Association (NFRWQPA) Utility Plan Guidance

document, interceptor sewers are equal to or larger than 24-inch and typically have very few, if any, individual service line connections and convey wastewater from connections with collector sewers. The scope of this study is to analyze the hydraulic capacity of interceptor and collector sewers. Figure 5-2 shows the gravity sewer system by pipe classification.

5.2.2. Diversion / Flow Split Manholes

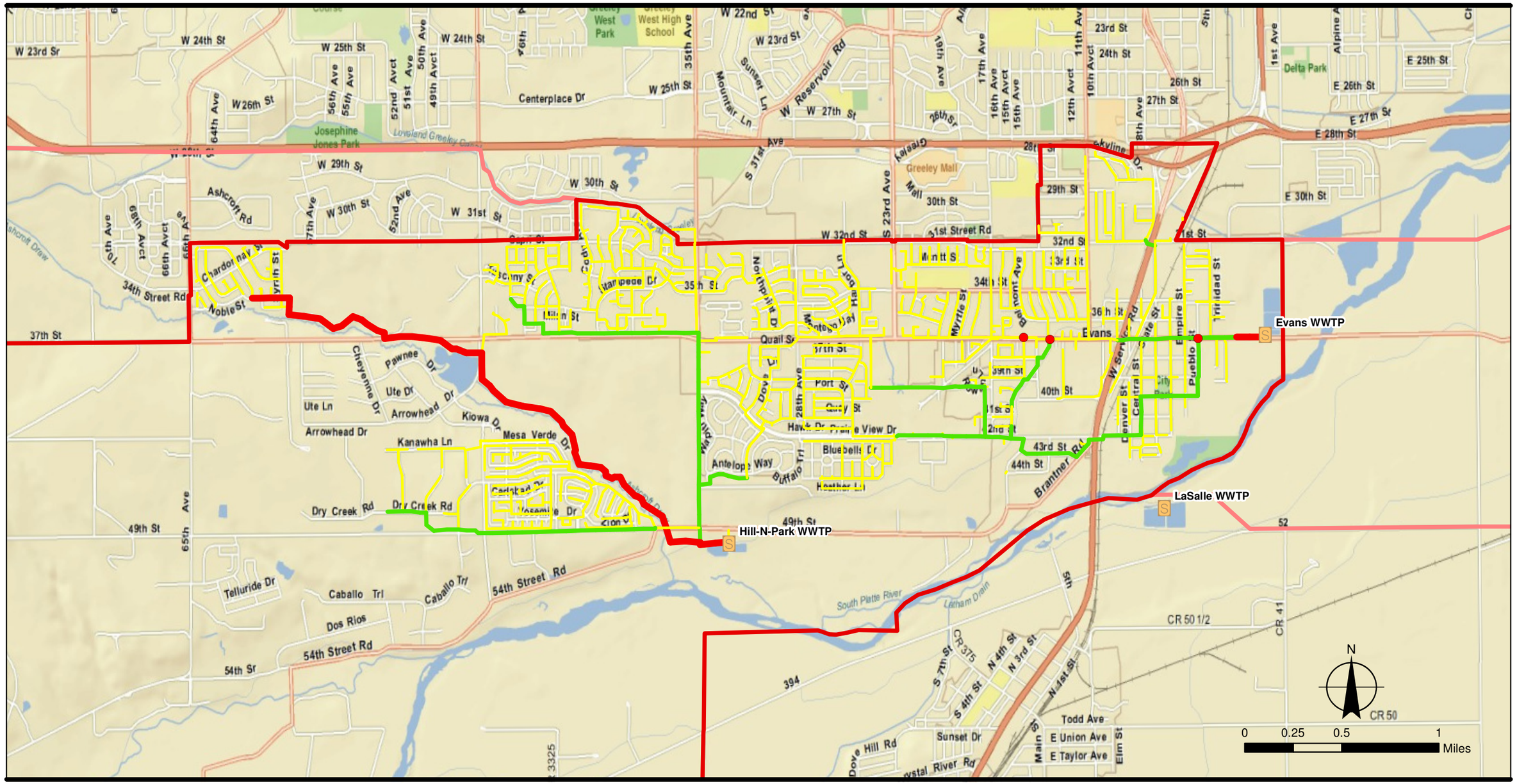
Evan's sewer system contains three active diversion or flow split manholes in the analyzed system which are used to direct flows from the incoming sewers to two outgoing sewers (Figure 5-1). These manholes are used to optimize sewer system capacity by directing flow into either a parallel sewer or into an adjacent sewer system that has additional capacity. Three manholes were identified as active diversion manholes that control and divert flow. Data on the two of the three active diversion manholes was collected by City staff in the field and the model was updated accordingly. Flow split percentages were assigned in the model based on observations in the field including photos and relative pipe capacity.

5.2.2.1. Diversion Manhole Updates and Modeling Assumptions

Manhole MH330 (pictured below) is a diversion manhole at 37th St and Belmont Ave. Flow enters from the west and exits to the east and south. Most of the flow exits to the east and some exits to the south which appears to be plugged but is not. However, the southern line is a local sewer that is not analyzed so in the model 100 percent of the flow has been assigned to the sewer main going east.

Photo 5-1. 37th St. and Belmont Ave. Diversion Manhole

Manhole MH290 (pictured on the next page) is a diversion manhole at 37th St and Valmont Ave. Flow enters from the west and exits to the east and south. The majority of the flow exists to the south into a larger diameter PVC line. The model was set up with 75 percent of the flow entering the manhole exiting to the south with the remaining 25 percent exiting to the east.



Legend

- Treatment Plants
- Diversion Manhole

Existing Pipes (Classification)

- Local
- Collector
- Interceptor

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Existing System Sewer Main Classification

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Figure 5.2
July 19th, 2011

For the buildout scenarios, the flow split at manhole MH290 at the 37th St and Valmont Ave intersection was increased from 25 percent of flow going east to 35 percent to alleviate capacity issues in the southern portion of the system. The percentage increase was determined by increasing the eastern portion of the flow split until the downstream system was reaching capacity. The flow split manhole can be modified in the future by using a short metal plate to raise the effective south invert elevations to divert more flow east.

Photo 5-2. 37th St and Valmont Ave Diversion Manhole

Manhole MH41 (no picture) is a diversion manhole located at 37th St and Pueblo Ave. Flow enters from the west and south and is split into two mains going east. More flow likely travels through the 15-inch main going east than the 12-inch main. The Model was set up using automatic flow allocation for this diversion manhole. Model results show a reasonable ration of increased flow in the 15-inch compared to the 12-inch main.

Manhole MH1476 (no picture) located west of 35th Ave. has been confirmed not be a diversion manhole. The 8-inch pipe P600 likely does not exist and therefore there is no connection between the Ashcroft Draw interceptor and the 15-inch line on 35th Ave.

5.2.3. Survey Data Updates

A comprehensive review and update to the GIS databases was performed which identified a significant amount of data missing from the pipe and manhole layers. A process was used to attempt to resolve as many of those data gaps as possible using as-built and recent project design survey data. A data collection plan was established to obtain key data points in the analyzed system through a field survey. The data collection plan identified 115 key manholes requiring field survey to determine rim elevations. Field survey effort resulted in obtaining pipe invert data for 111 manholes. The remaining manholes were not able to be surveyed due to access issues or not being able to locate the manhole in the vicinity of the GIS feature location. Inverts were not collected to the general availability of flowline (measure down) elevations within the City's GIS and to enable a greater number of manhole rim elevations to be surveyed since more data points were missing than expected.

A number of inverts and rim elevations were available from recent project design survey data and as-built drawings provided by the City of Evans. The next source of data used to assign inverts was using digital elevation model (DEM) data and flowline measurements in the GIS. A manual process was completed to assign inverts to the remainder of the analyzed system using engineering judgment which considered factors such as 1) adjacent and connection system depth, 2) ground slope and connecting sewer slopes, and 3) minimum pipe slopes. All data sources for manhole rim and inverts were identified in the GIS network in the "DataSource" field.

5.2.4. Network Validation

Due to the fact that manhole IDs in the Evans and Hill-n-Park WWTF collection systems were not unique, new unique manhole IDs were assigned to all manholes in the existing system.

Additional steps were taken to validate City of Evans' collection system network including verifying pipe slopes, pipe direction, manhole rim elevations, and interceptor profiles. All the changes were tracked within the GIS network shapefiles in the "ModifiedSource" and "ModSrcComm" fields. Slopes which were negative or excessively high (greater than 15 percent) were identified and pipe inverts were adjusted to correct the slope if required. Pipe direction throughout the entire network was verified and adjusted to correct direction if required. Manhole rim elevations were checked for irregularities, such as elevations causing excessively deep or negative pipe cover at manholes; such elevations were fixed as necessary using detailed elevation data. Finally, within the model environment, all the interceptor profiles were checked for irregularities such as breaks in continuous elevation fall downstream; such irregularities along the interceptor profiles were corrected as necessary.

5.3. Model Development

The hydraulic modeling software selected to analyze the collection system was MWH Soft, Inc. InfoSewer Version 5.0 Update 004. The ArcGIS platform on which the software operates allows for integration of City of Evans' GIS data to perform an efficient model build. The software calculates semi-dynamic hydraulics capturing flow attenuation, locations of surcharging and simplified backwater conditions which are sufficient for the scope of this study.

The steady state model of City of Evans sewer collection system was developed with InfoSewer using the project GIS manhole and pipe network. Wastewater flows were allocated to manholes with GIS spatial distribution of flows using sewershed polygons. Four scenarios were established in the model including an average dry-weather flow (ADWF) and peak hour wet-weather flow (PWWF) scenario for both existing and buildout conditions. These scenarios are based on the flow projections found in Chapter 2, Basis of Planning. The existing conditions model with allocated flows was validated with wastewater plant flow meter data. The collector and interceptor system was extended to serve the undeveloped areas within the WUSA and the entire Long Term Service Area (LTSA) based on the topography and existing roads.

The hydraulic model developed is a steady state and is not capable of extended-period simulation (EPS) or producing continuous results over a specified period of time. ADWF scenarios are based on average flows experienced in the system and can identify problems that would be aggravated in wet-weather flow conditions. PWWF scenarios use the peak hour dry weather flows combined with a wet weather event to determine the worst case flows in the system that also identify problems that may occur during storm events.

5.3.1. Modeling Approach

The modeling approach for the Wastewater Utility Plan (WUP) uses data from the entire pipe and manhole network that was developed but only collector and interceptor mains were modeled per the project scope of work and the requirements of the NFRWQPA. All collector and interceptor systems were analyzed for hydraulic capacity. Figure 5-3 shows the analyzed versus the unanalyzed pipes in the system.

In the future, an all-pipes model could be built, using the local system pipe network, for a more refined evaluation of the collection system. The benefits of an all-pipes model include increased accuracy in allocating wastewater flows to the sewer system, improved capture of actual flow attenuation in upper reaches of system, and simplifies the task of adding to and updating the system with survey data in the future.

Additional survey information gathered in the local system where data is missing can be used to improve the GIS network data in the future. Once that step is performed, importing updates from the pipe and manhole network in the model will be a straightforward process. In addition, new land use scenarios can be added to the model to simulate proposed developments and missing data can be obtained during the development review process. The accuracy of the data for the local and extended systems will increase over time as these data gaps are filled in and the results in these areas can be more examined thoroughly at that time.

5.3.2. Data and Basis of Hydraulic Model Construction

To construct the hydraulic model, pipes and manholes were imported into the model directly from the processed, updated, and validated GIS network described above. A one-way import function from GIS manhole and pipe data to the model database was kept to maintain the integrity of GIS data as the master source for model data input. Connectivity and direction of flow was verified in the model. Diversion manhole functionality was set and verified in the model. The collector and interceptor network for the buildout scenarios was extended out from the existing system using topography, roads, and future loading requirements. Recommended alignments and pipe sizes for the future system extensions in the WUSA and LTSA were established (see Figure 5-4). The pipe sizes presented are adequate based on the analysis criteria for all buildout flow scenarios.

5.3.3. Model Scenarios

A total of four model scenarios were established to represent the existing and buildout flow conditions in the system. Modeling existing and buildout conditions allows for locating current problems as well as predicting where future growth may cause additional

Legend

- Treatment Plants
- Analyzed Manhole
- Existing Interceptor/Collector Sewer (Analyzed)
- Existing Local Sewer (Not Analyzed)

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Existing System
Analyzed System

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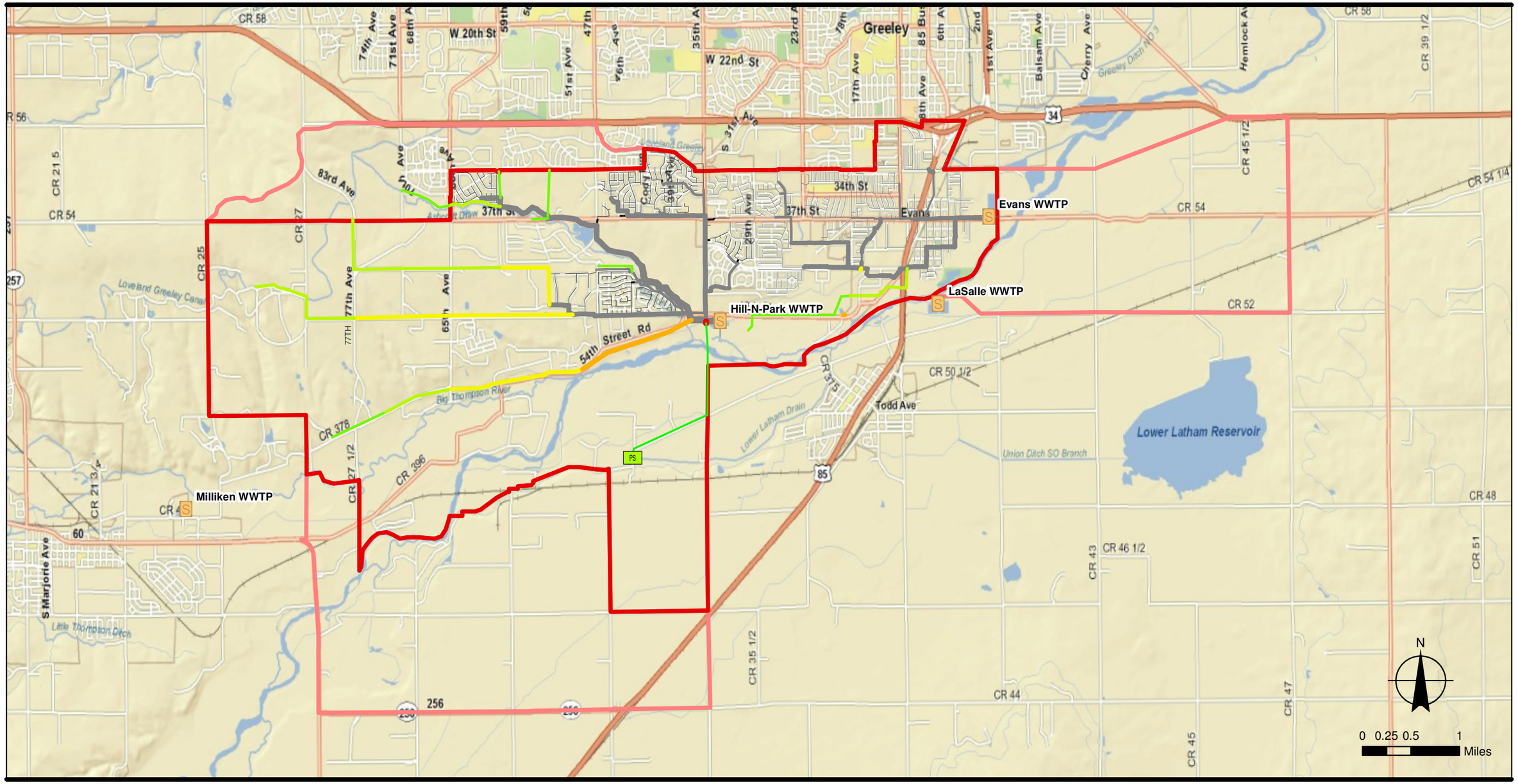
Figure 5.3
July 18th, 2011



Legend

- Treatment Plants
- Future Lift Station

Existing Pipe (Diameter): 8", 10", 12", 15", 24", 30"

Future Pipe (Diameter): 10", 12", 15", 18", 24", 27", 30"

Service Area
- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Planning Boundary
Buildout System Extensions

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Figure 5.4
July 18th, 2011

problems. For each of the two flow conditions, an average dry-weather (ADWF) and a peak hour wet-weather (ADWF + I/I) scenario were created. These scenarios enable separate analysis of all combinations of system loading. The buildout flow scenario is a base scenario that uses existing pipe diameters to evaluate where future problems may occur with existing pipes. Improvement alternative scenarios were developed after problem identification was completed to establish requirements for system improvements that account for buildout flows.

5.3.4. Wastewater Flow Allocation

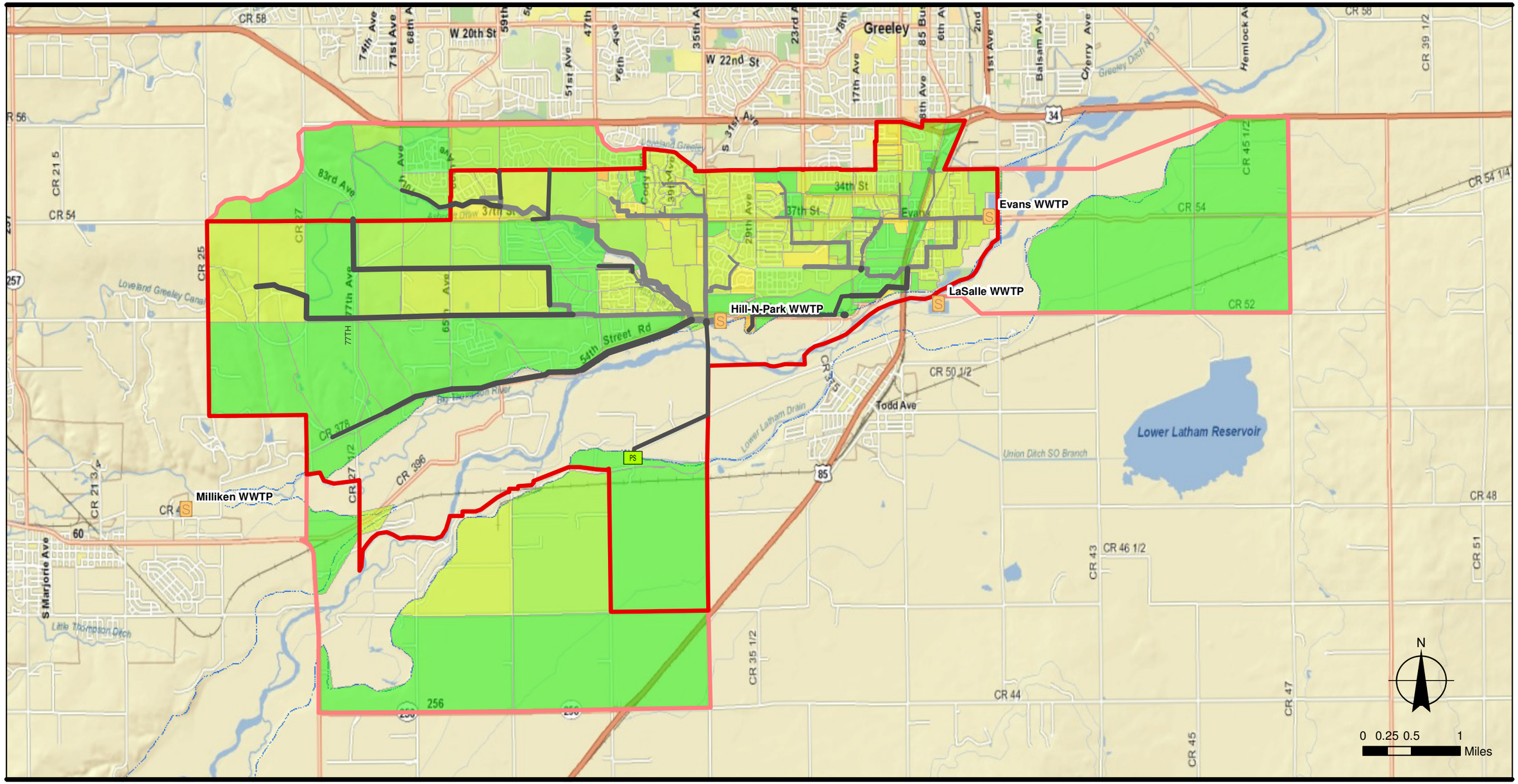
Wastewater flows were distributed to manholes across the system to establish system loading for both the existing and future conditions. The total wastewater flow in the system is a combination of sanitary flow and rainfall-induced infiltration and inflow (I/I). The wastewater flow projections from Chapter 2, Basis of Planning were used as the basis of flow allocation in the model. Both the WUSA and LTSA were included in the base buildout scenarios. The spatial distribution for flow allocation to the model was established by creating sewersheds that divide the service area into sections based on topography, land use, and existing sewer system infrastructure (see Figure 5-5). The ADWF sanitary flow distribution was completed using the sewersheds with unit flow factors and existing and future development conditions that incorporates population projections and land use. The I/I allowance factor, in gpd/acre, determined in Chapter 2 was applied to the ADWF spatial distribution using sewered area to establish PWWF loading.

5.3.5. Sanitary Flow

Sanitary flow or ADWF is the base loading in the system from the unit flow factors found in Chapter 2. ADWF is an averaged value over the year and varies throughout the year due to a number of variables. One such variable, typical in Colorado, is groundwater-induced infiltration especially in the summer months when the irrigation ditches are running. Sanitary flow or base loading includes the portion of groundwater-induced base infiltration experienced during dry-weather.

To distribute ADWF to individual manholes, a process of flow allocation was completed using sewersheds. The sewersheds increase accuracy of flow allocation by capturing general parcel boundaries, topography and collection system connectivity characteristics. Non-sewered area coverage was intersected with the sewersheds to establish the total sewered area in each. Only sewered area in the sewersheds was accounted for so that flow allocation was not overestimated in areas with considerable parks, roads, water bodies, and other non-sewered areas. A downstream manhole that would capture all flow from the sewershed was assigned to each sewershed to collect the load. Sewershed boundaries established for the WUSA and LTSA with ADWF loading are shown in Figure 5-5.

The next step entailed allocating existing and buildout ADWF to the sewersheds. The ADWF for the system was distributed to each sewershed polygon based on population projections and land use. Industrial and commercial flows were assigned to the sewersheds using the established unit flow factor of 1,000 gpd/acre and land use area.



Evans Wastewater Utility Plan

Legend

- Treatment Plants
- Floodplain (Approx.)
- Future Lift Station

Future Pipe (Diameter): 8", 12", 18", 30"

Existing Pipe (Diameter): 8", 10", 12", 15", 24", 30"

Sewersheds (ADWF - gpd/acre): 0 - 500; 500 - 1,000; 1,000 - 2,000; 2,000 - 5,000; 5,000 - 10,000; 10,000 - 20,000

Service Area: Planning Boundary; Long-term Growth/Service Area

Planning Boundary and LTSA Sewersheds and ADWF

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Figure 5.5

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The ADWF flow was transferred to the model based on the unique manhole ID associated with the sewersheds.

5.3.6. Inflow and Infiltration

I/I is a key component to modeling a sanitary collection system. Base infiltration from irrigation ditches is included in the unit flow factors and therefore included in the sanitary flow component. Rainfall-induced component of I/I, however, is accounted for separately and causes a rise in system flows in response to a storm event.

The exact response of the City of Evans' system to rainfall-induced I/I across the system is not well understood. There is little data available to provide a detailed allocation of I/I to the model. To allocate rainfall-induced I/I, a calculated I/I factor was applied to the sewersheds and used to simulate wet-weather conditions. The allowance of 2,011 gpd/acre, as defined in Chapter 2, does not vary over time and is the same system-wide.

5.3.7. T-Bone Lift Station Service Area

Greeley has expressed an interest in the City of Evans serving the T-Bone lift station area in the future. An existing Intergovernmental Agreement (IGA) is in place for this wastewater collection change. Greeley would like to build a gravity sewer south to the City of Evans but is currently lacking funding. The St. Michaels subdivision contributes a majority of the flow to the T-Bone Lift Station. There is undeveloped area east and west of St. Michaels which Greeley expects will be developed in the future. Greeley's sanitary sewer collection system has bottlenecks downstream of T-Bone Lift Station. Therefore Greeley has discussed eliminating the lift station completely and sending all flow to Evan's Ashcroft Draw interceptor via gravity. The T-Bone Lift Station currently experiences ADWF flows of approximately 0.4 mgd and is at 40 percent capacity. Future flows due to development are expected to range between 0.6 and 1.0 mgd total including the existing flows. Further conversations with Greeley have resulted in this potential project being pushed out more than 20 years and therefore it has not been included in the WUSA.

5.3.8. Existing Model Validation

Steady state flows at each WWTF were checked to validate 1) flow allocation to the model in the existing ADWF and PWWF scenarios, and 2) pipe network hydraulics including diversion manhole simulations. However, no flow monitoring or field measurements were conducted to verify the steady state at each WWTF. Since the model is steady state, the flow at the WWTF is basically the sum of the individual loading at manholes within the WWTF service basin.

The model validation results showed flow balancing between loading and the WWTF showing that the flow allocation was correctly assigned to the manholes. Pipe network hydraulics and flow diversions were functioning correctly. The network was validated with all pipes flowing downstream and downhill.

5.4. System Analysis Criteria

A hydraulic analysis using the collection system model enables locating problem areas during dry-weather and wet-weather scenarios both under existing and buildout conditions. Only the pipes identified in the analyzed system as previously discussed were included. To accomplish the analysis, project problem identification criteria is necessary. These criteria also aid in the development of system improvements to verify that problems have been resolved. Problem identification criteria is based on full-flow (or percentage thereof) within pipes and with surcharge conditions at manholes being identified as system problem locations.

The criteria remain the same for existing and future buildout scenarios but differ between peak-hour dry-weather and peak-hour wet-weather. Results for both capacity via depth versus pipe diameter (d/D) and velocities were examined. The problem identification criteria established for this project is comprised of the following:

1. Capacity: Local Collector / Collector System (8” - 18”)
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of one-half of the full pipe (d/D = 50%)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100%)
2. Capacity: Interceptor (> 18”) System
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of 70 percent of the full pipe (d/D = 70%)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100%)
3. Velocities: Entire Analyzed System
    a. Peak-hour dry-weather – Minimum velocity equal to 2 fps.
    b. Peak-hour wet-weather – Maximum velocity equal to 10 fps.

The interceptor system has a greater peak-hour dry-weather criteria value since the interceptor system is larger than the collector system and equal flow increases do not affect the flow depth as drastically in the interceptors compared to the local collector and collector system pipes. Additionally, the interceptors typically have greater d/D values during normal dry-weather conditions. For the collector system, the design criteria set forth in City of Evans’ Water and Sanitary Sewer Specifications, Paragraph 7.03 is 60 percent which is higher than presented above for peak-hour, although the specifications do not differentiate between dry and wet-weather conditions. The analysis criteria set are conservative compared to the design specifications and the problem pipe is overridden if the d/D is not much over the 50 percent full criteria.

5.5. Existing and Buildout Condition Model Results

The model results were compared against the analysis criteria to locate potential hydraulic problems within the system. Results from the validated model were obtained

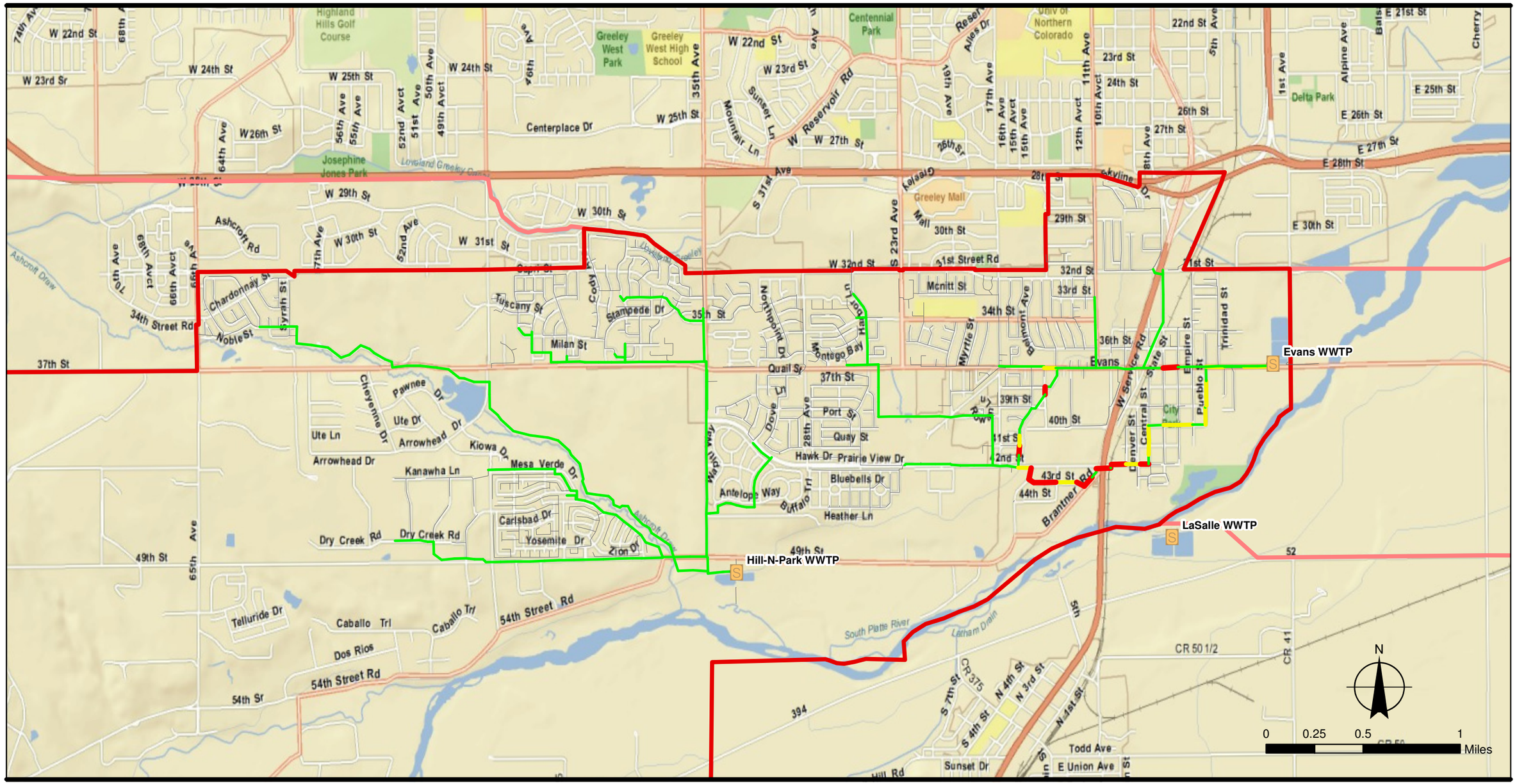
from the four scenarios including existing and unimproved buildout dry-weather and wet-weather conditions. For pipes, depth of flow versus pipe diameter (d/D) was utilized as the indicator of capacity problems. Freeboard depth in manholes was taken from the model results to locate possible SSO risk. A summary of resultant flows at the WWTFs are shown in Table 5-2 for each of the four base scenarios assuming flow from the WUSA and the entire LTSA for Buildout going to their respective treatment facilities.

Table 5-2. WWTF Flows from Model Results

Evans WWTF	
Flow Scenario	**WWTF Influent**
Existing ADWF	1.230 mgd
Existing PWWF	3.284 mgd
Buildout ADWF	2.207 mgd
Buildout PWWF	8.815 mgd
Hill-n-Park WWTF	
Flow Scenario	**WWTF Influent**
Existing ADWF	0.491 mgd
Existing PWWF	1.660 mgd
Buildout ADWF	4.350 mgd
Buildout PWWF	23.322 mgd

The hydraulic model was executed for existing conditions to identify existing condition hydraulic problems. The hydraulic model was also executed for the base buildout condition sanitary flow scenarios using the future collector and interceptor sewer alignment extensions. The buildout scenario includes the entire WUSA and LTSA which is the worst-case flow and infrastructure scenario. The model results are depicted in Figures 5-6, 5-7, 5-8, and 5-9 for existing dry-weather, existing wet-weather, buildout dry-weather, and buildout wet-weather scenarios. Pipes with potential problems based the established criteria and manholes with freeboard of less than 2 feet were highlighted. Manholes with freeboard of less than 2 feet are considered at-risk for SSOs.

Velocities should remain below 15 fps in all scenarios to minimize the potential for pipe and manhole erosion and excess odor generation. A number of pipes with high velocities (up to 20 fps in wet-weather scenarios) were noted in the existing system. The high velocities are due to pipes having steeper slopes and likely cannot be corrected without significant investments. There were also a number of existing pipes with low velocities below 2 fps in the peak hour dry-weather flow conditions that could affect the self-cleaning of the pipes. Yearly pipe flushing should be conducted in areas where there are low velocities to prevent clogging due to solids.



Legend

- Treatment Plants
- Exist. Local Sewer (Unanalyzed)

Manhole (Unfilled Depth)
- 0 - 1 ft
- 1 - 2 ft

Pipe Capacity (d/D)
- 0 - 40%
- 40 - 50%
- 50 - 100%

Service Area
- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Planning Boundary
Existing ADWF - Model Results

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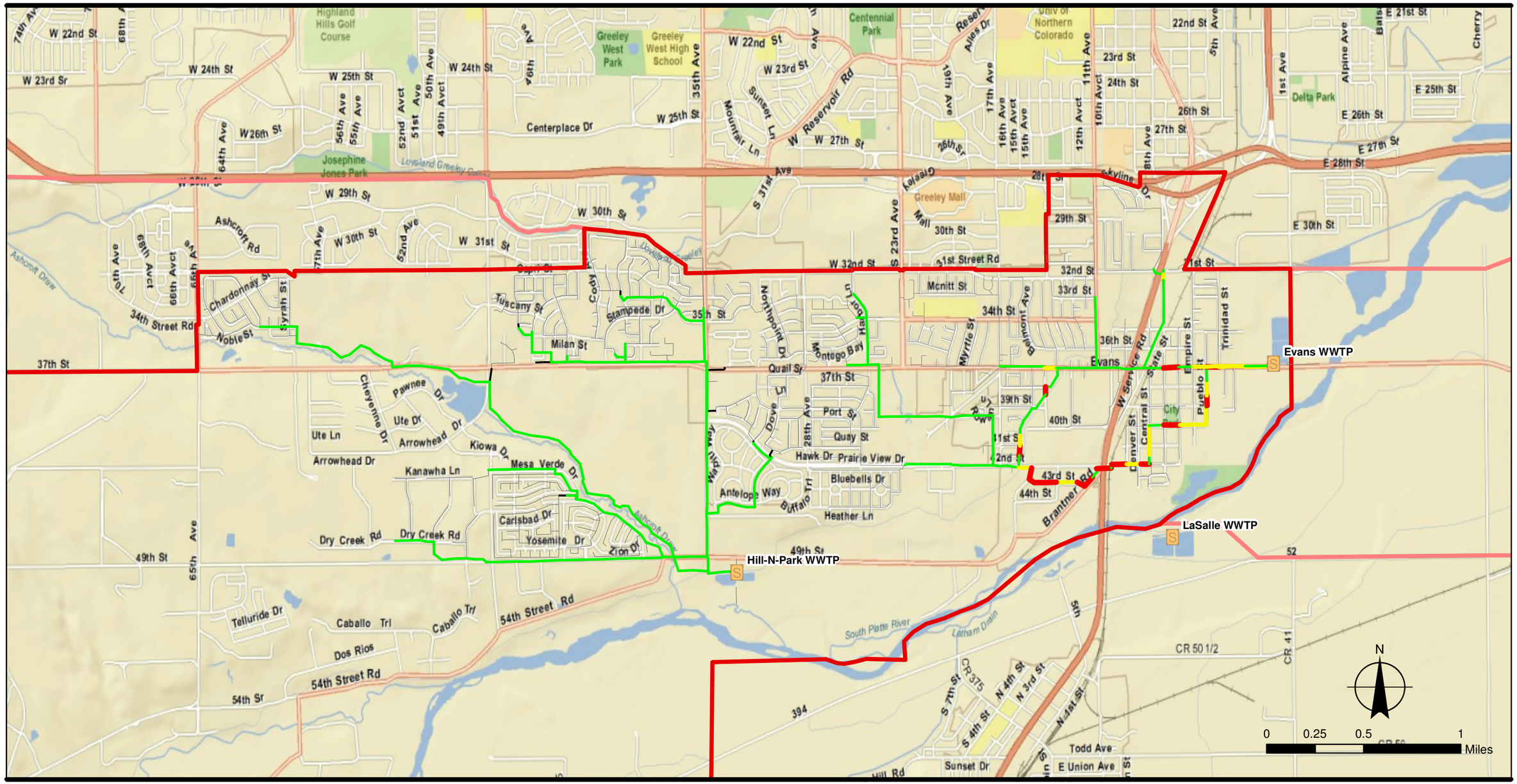
Figure 5.6
July 18th, 2011



Legend

- TreatmentPlants
- Exist. Local Sewer (Unanalyzed)

Manhole (Unfilled Depth)
- 0 - 1 ft
- 1 - 2 ft

Pipe Capacity (d/D)
- 0 - 70%
- 70 - 90%
- 90 - 100%

Service Area
- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Planning Boundary
Existing PWWF - Model Results

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Figure 5.7
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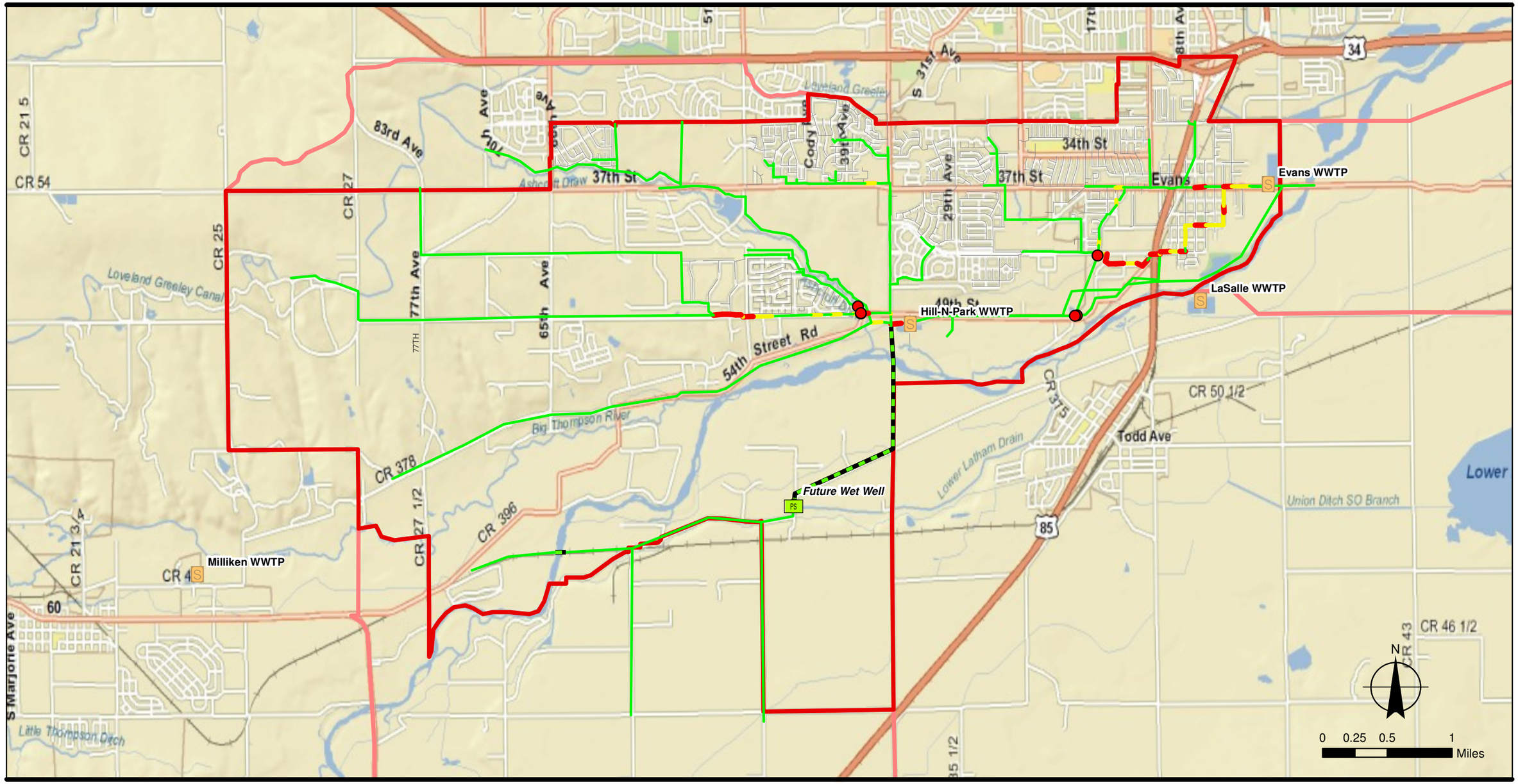

Legend

- Treatment Plants
- Exist. Local Sewer (Unanalyzed)
- Future Lift Station
- Future Forcemain

Manhole (Unfilled Depth)

- 0 - 1 ft
- 1 - 2 ft

Pipe Capacity (d/D)

- 0 - 40%
- 50 - 50%
- 50 - 100%

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Planning Boundary and LTSA Buildout ADWF - Model Results

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Figure 5.8
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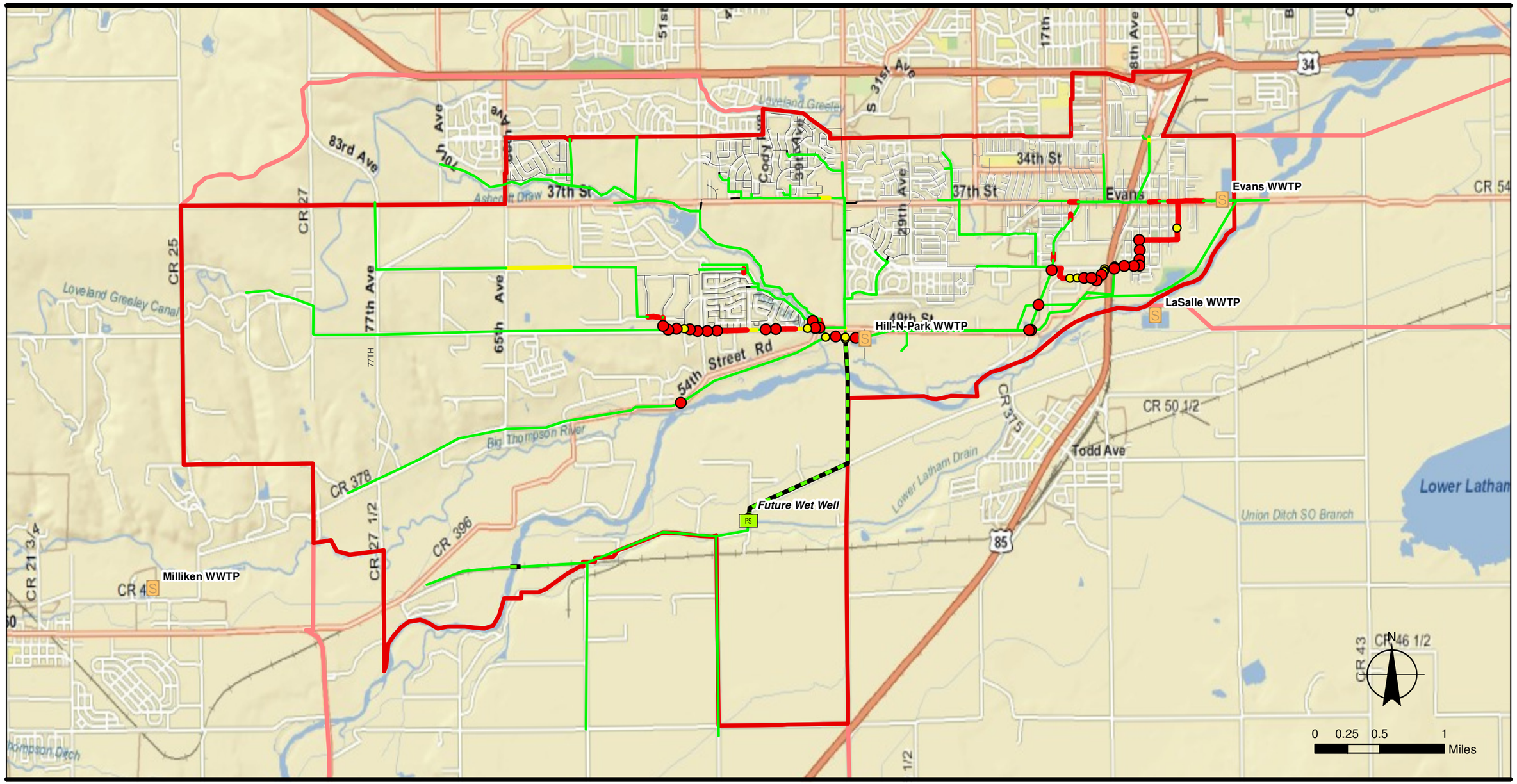

Legend

- Treatment Plants
- Exist. Local Sewer (Unanalyzed)
- Future Lift Station
- Future Forcemain

Manhole (Unfilled Depth)

- 0 - 1 ft
- 1 - 2 ft

Pipe Capacity (d/D)

- 0 - 70%
- 70 - 90%
- 90 - 100%

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Planning Boundary and LTSA Buildout PWWF - Model Results

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Figure 5.9
July 18th, 2011

5.6. Problem Identification and Characterization

Hydraulic problems within the existing system for the existing and buildout condition scenarios were identified using the established problem criteria. The potential problem pipes established from the model results were examined to identify likely hydraulic issues under the various flow scenarios. There are a total of 60 pipes in the existing system with hydraulic deficiencies identified including 18 problem pipes under existing flow conditions. A problem identification and characterization process was completed to better understand the nature and extent of these problems.

5.6.1. Problem Identification

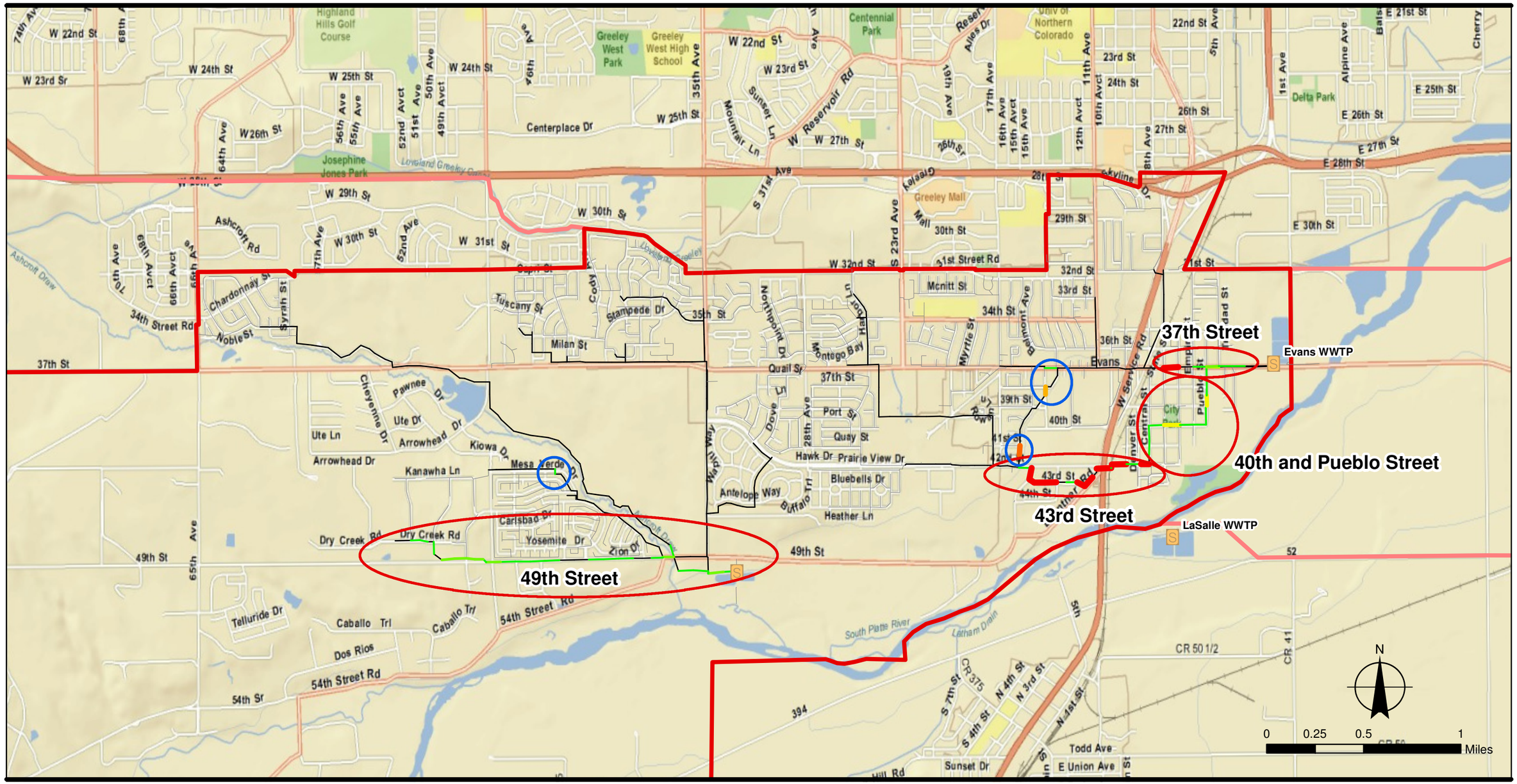
The hydraulic problems were separated into two categories; Type A and Type B. Type A problems consisted of a series of problem pipes and that were hydraulically connected to one another. Type B problems are isolated hydraulic restrictions that are not hydraulically connected to other problem locations or series of problem pipes. Type A problems accounted for 93 percent of the problem pipes or a total of 55 pipes with a cumulative length of approximately 3.3 miles of pipe. Type B problems accounted for the remaining 7 percent of problem pipes or a total of 5 pipes with a cumulative length of approximately 0.2 miles of pipe. Both Type A and Type B problems have system improvement recommendations developed and estimates of capital cost prepared. It is envisioned that Type A problems are identified as recommendations for capital improvements while the Type B problems are addressed through coordination with development activities, system rehabilitation and replacement R&R work, or replacement with other department CIP projects.

A total of four Type A problems areas were identified using engineering judgment based on their relative proximity of the problem pipes and hydraulic connectivity (see Figure 5-10). Each problem group was given an identification location named by the street or intersection where the problems occur.

To assess the relative magnitude of the hydraulic problems, the Type A and Type B problem pipes were scored based on a weighted scale depending on the extent of the problem which is signified by the scenarios in which the problem is present. The heaviest-weighted scenario was existing wet-weather conditions with weight of 8 when worst-case criteria (d/D = 1) identifies the problem under existing flow conditions. The remaining weighted scores were assigned in the following order: existing dry-weather conditions with a weight of 6, buildout wet-weather conditions with a weight of 4, and buildout dry-weather conditions with a weight of 2. This scoring methodology allowed individual pipes to be assigned a problem score from 2 to 20 that captured the magnitude of the problem over all flow scenarios. Figure 5-10 shows the location of the problems in the system and the relative magnitude of the problem based on the flow scenarios in which they occur.

5.6.2. Problem Characterization – Type A Problem Locations

Type A problem areas were characterized using a set of descriptive categories to better understand the nature, extent, and criticality of the problems for the development of improvements. Table 5-3 identifies the descriptive categories and what they entail in



Evans Wastewater Utility Plan

Legend

- Treatment Plants
- Exist. Local Sewer (Unanalyzed)
- Exist. Interceptor/Collector Sewer (Unanalyzed)

Problem Pipes (Score)

- 4
- 6
- 8
- 14
- 16
- 18
- 20

Service Area

- Planning Boundary
- Long-term Growth/Service Area

- Type A Problem Areas
- Type B Problem Areas

Existing Problem Pipes and Areas

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Figure 5.10
July 18th, 2011

helping to define the problem groups. Tables 5-4 through 5-7 characterize each of the problem groups utilizing the descriptive categories.

Table 5-3. Problem Characterization Categories and Definitions

Problem Summary	
Location:	Identifies the sewer basin and major street intersection or adjacent feature.
Problem Summary:	Summarizes the system problems as developed using the problem identification criteria.
Pipe Classification:	Identifies if the pipes are categorized as collector or interceptors.
Diameter Range:	Summarizes the pipe diameters.
Material Types:	Summarizes the pipe material types.
Problem Extent (Existing):	Summarizes the total length of problem pipe under existing dry- and wet-weather flow scenarios.
Problem Extent (Buildout):	Summarizes the total length of problem pipe under buildout dry- and wet-weather flow scenarios.
Manhole SSO Risk (Existing):	Summarizes the number of manholes that have less than 1 foot of freeboard under the existing wet-weather flow scenario.
Manhole SSO Risk (Buildout):	Summarizes the number of manholes that have less than 1 foot of freeboard under the buildout wet-weather flow scenario.
Data Confidence:	Summarizes the number of manhole inverts that did not have survey or as-built data and were then interpolated for modeling purposes. This category is presented as a percentage of the total number of manholes within the problem area. Data confidence of 100 percent indicates that all manhole invert data was obtained from as-built or survey data and was not adjusted as part of the network validation process.
Accessibility:	Identifies if the problem pipes are generally located in roadways, creek/stream corridors or other alignment conditions.
Comments:	Provides a brief problem assessment considering the characterization findings for each category.

Table 5-4. Problem Characterization – 37th Street

Problem Summary	
Location:	Problem area located in Evans WWTF basin along 37th Street from Boulder Street to Riverside Parkway
Problem Summary:	Due to small pipe slopes limited by shallow pipe depths there are capacity problems in existing and buildout dry- and wet-weather flow scenarios.
Pipe Classification:	Collector
Diameter Range:	12 to 15-inch
Material Types:	PVC and Clay Pipe
Problem Extent (Existing):	735 ft
Problem Extent (Buildout):	2,513 ft
Manhole SSO Risk (Existing):	4
Manhole SSO Risk (Buildout):	4
Data Confidence:	86%
Accessibility:	Roadways
Comments:	Problem area is located in shallow sloped 37th Street where problems generally occur under wet-weather conditions but one pipe has existing dry-weather capacity issues. All pipes are located within the roadway and some are in parallel collector system leading to Evans WWTF.

Table 5-5. Problem Characterization – 40th and Pueblo Street

Problem Summary	
Location:	Problem area located in Evans WWTF basin along Pueblo Street from 37th Street to 40th Street, and 40th Street from Pueblo Street to Central Street, and Central Street from 40th Street to 42nd Street.
Problem Summary:	Due to small pipe slopes limited by shallow pipe depths there are capacity problems in existing wet-weather and buildout dry- and wet-weather flow scenarios.
Pipe Classification:	Collector
Diameter Range:	15-inch
Material Types:	PVC
Problem Extent (Existing):	702 ft
Problem Extent (Buildout):	3,488 ft
Manhole SSO Risk (Existing):	2
Manhole SSO Risk (Buildout):	7
Data Confidence:	75%
Accessibility:	Roadway Corridors
Comments:	Problem area is located in shallow sloped Pueblo Street, 40th Street, and Central Street where problems generally occur under buildout conditions but one pipe has existing wet-weather capacity issues. There is significant SSO risk under wet-weather conditions. All pipes are located within the roadway right-of-ways.

Table 5-6. Problem Characterization – 43rd Street

Problem Summary	
Location:	Problem area located in Evans WWTF basin along 42nd Street from Central Street to US Hwy 85, and 43rd Street from US Hwy 85 to Industrial Parkway, and 42nd Street from Industrial Parkway to the Evans Town Ditch.
Problem Summary:	Due to shallow pipe slopes limited by shallow pipe depths there are capacity problems in existing and buildout dry- and wet-weather flow scenarios.
Pipe Classification:	Collector
Diameter Range:	12 to 15-inch
Material Types:	PVC and Clay Pipe
Problem Extent (Existing):	2,623 ft
Problem Extent (Buildout):	3,771 ft
Manhole SSO Risk (Existing):	0
Manhole SSO Risk (Buildout):	14
Data Confidence:	72%
Accessibility:	Roadway corridors, local unpaved streets, bore under US Hwy 85, crossing Evans Town Ditch.
Comments:	Problem area is located in shallow sloped 42nd and 43rd Streets where problems occur under both existing and buildout conditions with significant SSO risk under wet-weather conditions. Pipes are located within the roadway right-of-ways, unpaved streets, and under highways and ditches.

Table 5-7. Problem Characterization – 49th Street

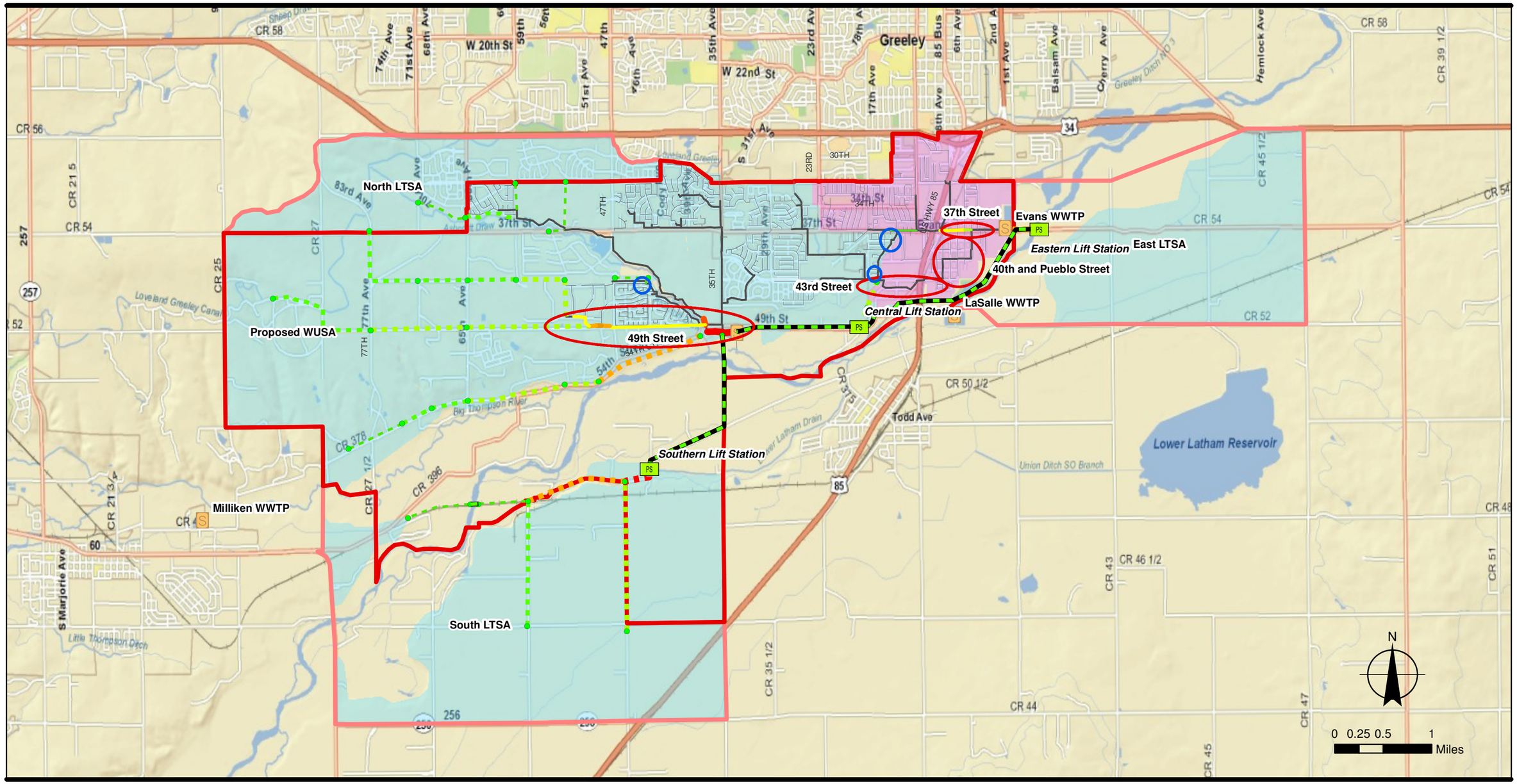
Problem Summary	
Location:	Problem area located in Hill-n-Park WWTF basin along Dry Creek Rd from Pendleton Ave to Charleston Ave, and Charleston Ave from Dry Creek Rd to 49th St, and 49th Street from Charleston Ave to 54th Street, and across local unpaved roads to the Hill-n-Park WWTF.
Problem Summary:	Due to growth and increased buildout flow there are capacity problems in buildout dry- and wet-weather flow scenarios.
Pipe Classification:	Collector and Interceptor
Diameter Range:	8 to 30-inch
Material Types:	A2000, DIP, PVC and Clay Pipe
Problem Extent (Existing):	0 ft
Problem Extent (Buildout):	8,110 ft
Manhole SSO Risk (Existing):	7
Manhole SSO Risk (Buildout):	21
Data Confidence:	76%
Accessibility:	Roadway corridors and local unpaved streets
Comments:	Problem area is located in Dry Creek Rd, Charleston Ave, and 49th Street to WWTF where problems occur only under buildout conditions with significant SSO risk under wet-weather conditions. Problems are mostly due to increased flow from growth in west. Pipes are located within the roadway right-of-ways and unpaved streets.

5.7. Improvement Alternatives and Recommendations

Four improvement alternative plans were constructed to alleviate the problems within the existing and buildout systems. The base buildout scenario in the InfoSewer model was used to properly size improvements in the three alternative plans to pass the analysis criteria in Section 5.4.

5.7.1. Improvement Alternatives

Improvement Alternative 1 consists of the proposed WUSA and entire LTSA. The WWTF basin boundaries were modified based on limiting the buildout ADWF at the Evans WWTF to 1 mgd. To limit the flow to the Evans WWTF, a portion of the western Evans WWTF basin has been transferred to the Hill-n-Park WWTF basin. Flow is collected from this transferred area to a new Central Lift Station (located near 17th St and South Platte River) by gravity and then pumped to the Hill-n-Park WWTF. The flows from the East and South LTSA go to Hill-n-Park WWTF by the Eastern and Southern Lift Station and forcemains, respectively. The North LTSA flow is conveyed by gravity to the Hill-n-Park WWTF via the Ashcroft Draw interceptor. Figure 5-11 shows Improvement Alternative 1 and the modified WWTF basin boundaries. This alternative



Legend

- Treatment Plants
- Future Manhole
- Future Lift Station
- Future Forcemain
- Exist. Local Sewer (Unanalyzed)
- Exist. Interceptor/Collector Sewer (Analyzed)

Existing Pipe Improvements

- 12"
- 18"
- 21"
- 24"
- 27"
- 36"
- 42"

Future Pipe Extensions

- 12"
- 15"
- 18"
- 24"
- 30"

Service Area

- Planning Boundary
- Long-term Growth/Service Area

WWTF Basin

- Evans
- Hill-n-Park

- Type A Problem Areas
- Type B Problem Areas

Evans Wastewater Utility Plan

Planning Boundary and LTSA Improvement Alternative 1

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Figure 5.11

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slightly decreases the problems in the Evans WWTF but increases the problems in the existing pipes in the Hill-n-Park WWTF basin.

Improvement Alternative 2 also consists of the proposed WUSA and entire LTSA. The WWTF basin boundaries were not modified from the original locations. This alternative does not reduce flow to the Evans WWTF and therefore a new Central Lift Station is not required. The flows from the East LTSA go to the Evans WWTF by gravity and a siphon across the South Platte River. South LTSA flows go to Hill-n-Park WWTF by the Southern Lift Station and forcemain. The North LTSA flow is conveyed by gravity to the Hill-n-Park WWTF via the Ashcroft Draw interceptor. Figure 5-12 shows Improvement Alternative 2 and the original WWTF basin boundaries. This alternative causes problems in the Evans WWTF basin but not as many in the Hill-n-Park WWTF basin as Alternative 1.

Improvement Alternative 3 consists only of the proposed WUSA and the Northern portion of the LTSA. Alternative 3 does not include the South and East LTSAs as these are outside the 20-year planning horizon. The North LTSA includes an IGA with Greeley and it is anticipated this transfer of service area from City of Greeley to City of Evans will occur in the near future. The WWTF basin boundaries were not modified from the original locations. This alternative does not reduce flow to the Evans WWTF so no Central Lift Station is required. The Southern Lift Station and forcemain are still required because of the buildout area in the South WUSA that can not be delivered to the Hill-n-Park WWTF by gravity. The North LTSA flow is conveyed by gravity to the Hill-n-Park WWTF via the Ashcroft Draw interceptor. Figure 5-13 shows Improvement Alternative 3 and the original WWTF basin boundaries.

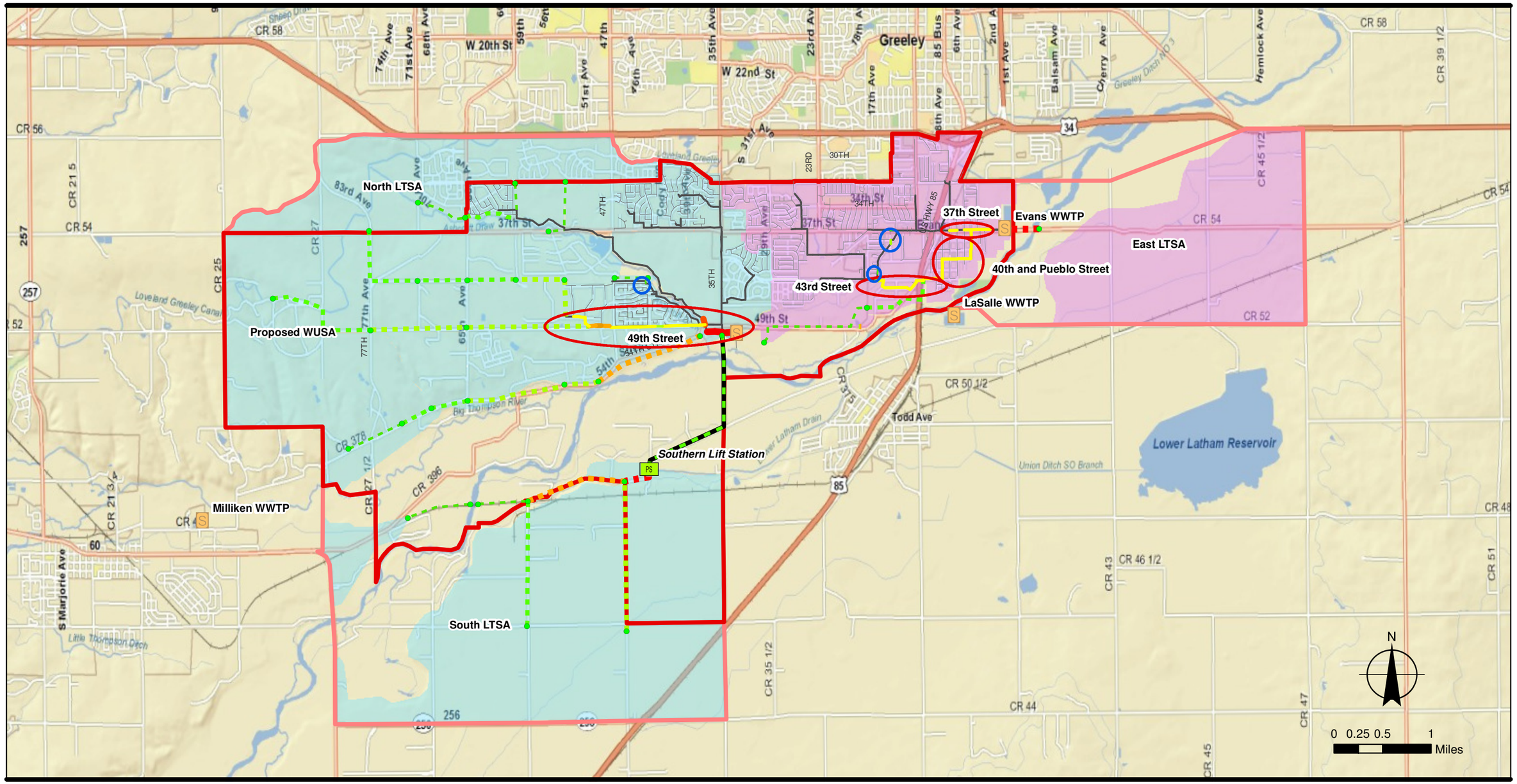
Improvement Alternative 4 consists of the proposed WUSA and entire LTSA. Alternative 4 includes taking the Evans WWTF offline, so it can operate as a lift station that will pump influent from the Evans WWTF basin to the Hill-n-Park WWTF to be treated. Repurposing the Evans WWTF from a treatment plant to a lift station will include the addition of three submersible pumps in the headworks facility and the installation of 3.5 mile 14 inch forcemain between the two facilities. Figure 5-14, Improvement Alternative 4, displays the potential alignment of the new forcemain. To accommodate the additional influent, the Hill-n-Park WWTF will require upgrades in order to meet the entire demand of the system, which would be a costly project.

5.7.2. Recommended Improvement Alternative

Improvement Alternative 3 is the Recommended Plan as it represents the expected development within the planning horizon for this study and requires the fewest lift stations and forcemains when compared to Alternatives 1, 2 and 4. The Evans WWTF would need to be improved to treat more than the initial limit of 1.2 mgd. However, these improvements are acceptable given the treatment process improvements and future permitting recommendations within the WWTF evaluation section of this study.

Improvement fact sheets for the Recommended Plan have been developed for each Type A problem area. The fact sheets characterize the improvements that have been developed that alleviate the hydraulic problems and the associated benefits and issues. An improvement priority tier is assigned to each project as a recommendation of projects that

should happen first. A basis of costs has been established capturing source of unit costs and assumptions made during the cost estimating task. Finally, an implementation plan has been established that outlines the projects ordered by priority and the overall improvement project costs. Type B problems and associated improvements are captured in the implementation plan. Future infrastructure expansion costs have been estimated for the Recommend Plan.



Evans Wastewater Utility Plan

Planning Boundary and LTSA Improvement Alternative 2

Legend

- TreatmentPlants
- Exist. Local Sewer (Unanalyzed)
- Exist. Interceptor/Collector Sewer (Analyzed)
- Future Lift Station
- Future Manhole
- Future Forcemain

Existing Pipe Improvements

- 12"
- 18"
- 21"
- 24"
- 27"
- 36"
- 42"

Future Pipe Extensions

- 12"
- 15"
- 18"
- 24"
- 30"

Service Area

- Planning Boundary
- Long-term Growth/Service Area
- Type A Problem Areas
- Type B Problem Areas

WWTF Basin

- Evans
- Hill-n-Park

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Figure 5.12
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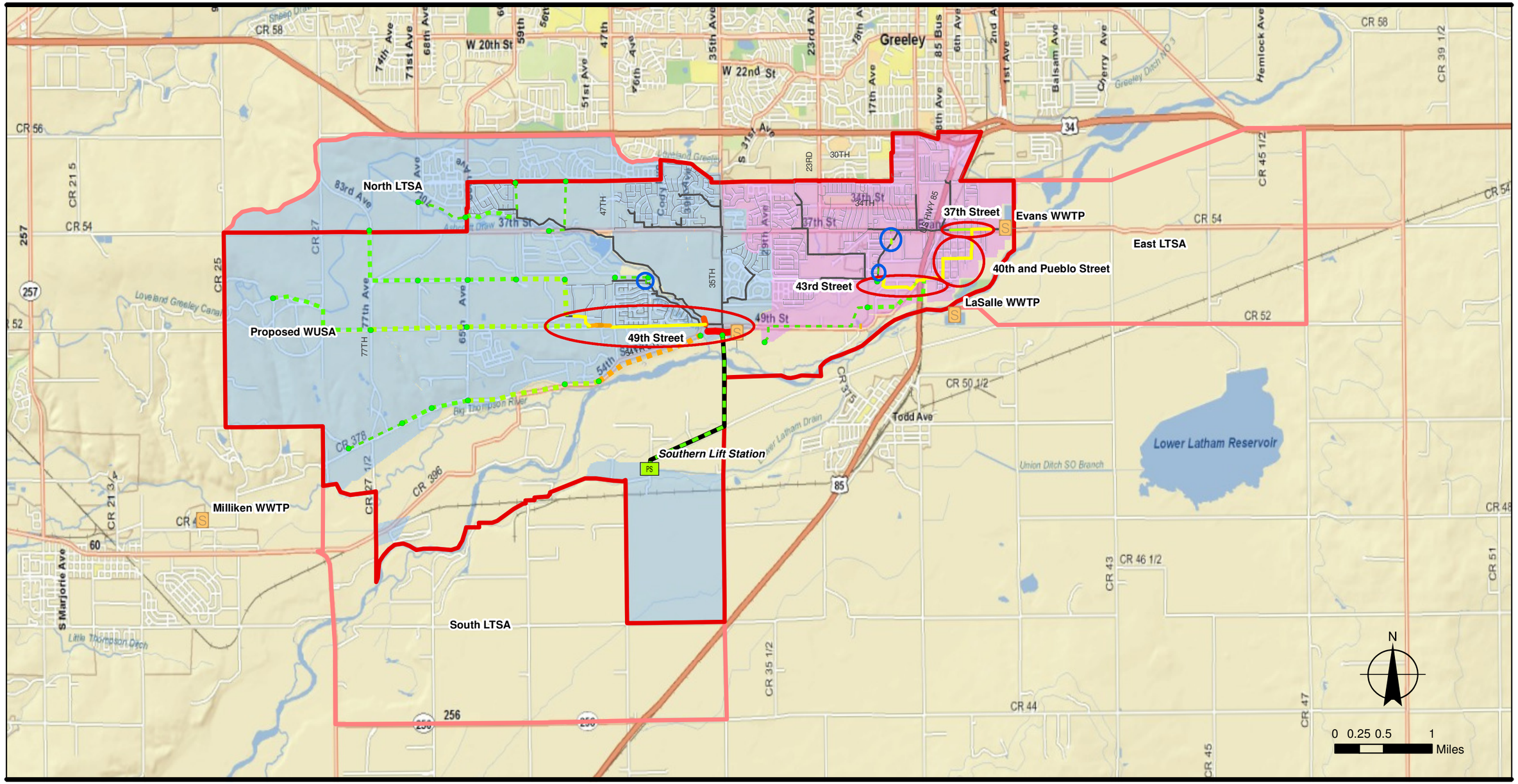

Legend

- Treatment Plants
- Exist. Local Sewer (Unanalyzed)
- Exist. Interceptor/Collector Sewer (Analyzed)
- Future Manhole
- Future Lift Station
- Future Forcemain

Existing Pipe Improvements

- 12"
- 18"
- 21"
- 24"
- 27"
- 36"
- 42"

Future Pipe Extensions

- 12"
- 15"
- 18"
- 24"
- 30"

Service Area

- Planning Boundary
- Long-term Growth/Service Area

WWTF Basin

- Evans
- Hill-n-Park

- Type A Problem Areas
- Type B Problem Areas

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Evans Wastewater Utility Plan

Planning Boundary and North LTSA Improvement Alternative 3

Figure 5.13

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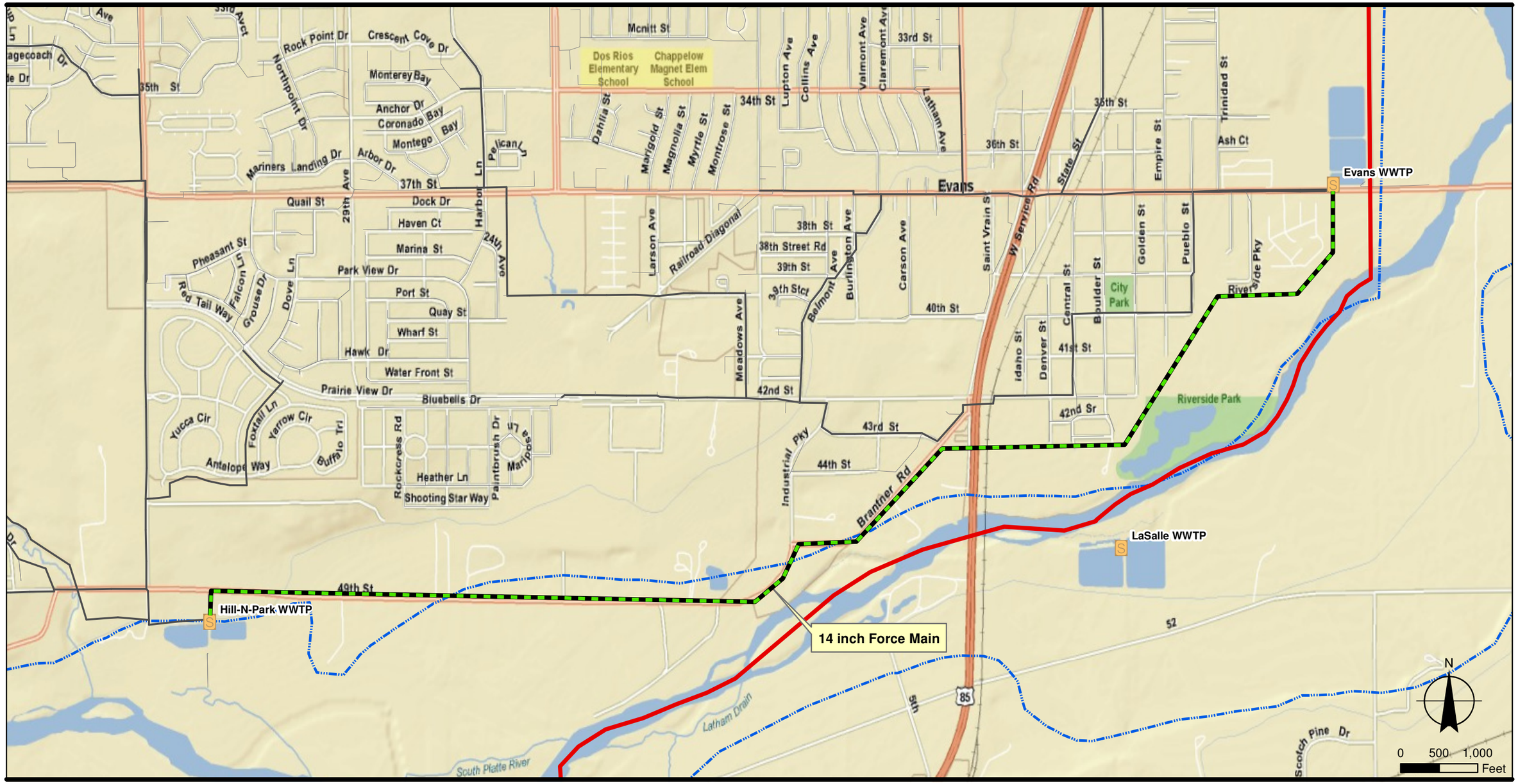

Legend

- Treatment Plants
- Existing Pipe Analyzed
- Existing Pipe Not Analyzed
- City Limits (Neighboring Communities)
- Floodplain (Approx.)

Service Area

- Planning Boundary
- Long-term Growth/Service Area

Evans Wastewater Utility Plan

Improvement Alternative 4

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Figure 5.14
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5.7.3. Improvement Priority

Problem areas are grouped in three tiers to establish implementation priority: Tier 1 problem areas are Type A improvements and have the highest priority; Tier 2 projects are also Type A improvements but have lower priority compared to Tier 1; and Tier 3 projects are Type B improvements which have the lowest priority. The priorities were assigned based on a number of qualitative factors including the extent of the problem and the flow conditions in which they occur, potential for SSOs, ease of constructability, and data confidence. These factors are summarized in the problem characterization tables within Section 5.6.

Table 5-8 lists each Type A problem area and an assigned Tier (Tier 1 or 2) depending on the overall priority of improvements based on the previously listed factors.

Table 5-8. Improvement Priority by Problem Area

Problem Area	
40th and Pueblo Street	Tier 1
43rd Street	Tier 1
37th Street	Tier 2
49th Street	Tier 2

5.7.4. Anticipated Improvement Timeline

It is anticipated that Tier 1 projects would be scheduled for construction within the next 10 years. Tier 2 projects would be completed in the following 10-years within the 20-year planning horizon or before as opportunities arise associated with other utility or roadway projects in the project vicinity. It is anticipated that Tier 3 projects could be addressed as minor capital projects or with system rehabilitation work in the project vicinity. Future infrastructure expansion due to growth is expected to be constructed as development occurs.

5.8. Improvement Alternative Cost Estimating

Costs for improvements were developed to provide the City of Evans with a budget cost estimate for the Recommended Plan. The improvements to the existing system were included in the estimate as Type A and Type B improvements. The future infrastructure expansion projects were included as a separate cost estimate.

5.8.1. Cost Estimating Rationale

An opinion of probable costs has been prepared for each alternative. The American Association of Cost Engineers has defined three basic categories of estimates in an effort to establish an expected accuracy range for various types of cost estimates. They include:

- *Order of Magnitude Estimate* – This is an approximate estimate made without detailed engineering data. Some examples would be: an estimate from cost-estimating curves, an estimate using scale-up or scale-down factors, or an approximate ratio estimate. It is normally expected that an estimate of this type would be accurate within +50% or -30%.

- *Budget Estimate* – The term budget in this case applies to the owner's budget and not the budget as a project-control document. A budget estimate is prepared with the use of spreadsheets, layouts and equipment details. It is normally expected that an estimate of this type would be accurate within +30% or -15%.
- *Definitive Estimate* – As the name implies, this is an estimate prepared from well defined engineering data. At a minimum, the data must include: fairly complete plans and elevations, piping diagrams, equipment data sheets and quotations, structural sketches, soil data and a complete set of specifications. The "maximum" definitive estimate would be made from Approved for Construction" drawings and specifications. It is expected that a definitive estimate would be accurate within +15% or -5%.

Based on the current level of this project's development and engineering efforts, and considering that no subsurface (geotechnical) investigations have been performed, an order of magnitude estimate is the most accurate description of the probable project costs that can be expected. As site specific information is obtained and designs are refined, a more accurate cost opinion can be developed. The primary objective of the cost opinions presented here is to provide a basis for comparison of relative costs between alternatives.

This section summarizes the assumptions and possible unknowns related to cost items, and presents an opinion of probable project cost for each alternative. The comparison is concluded with an analysis of the probable cost of each alternative related to the overall benefit associated with each modification.

Capital costs are expressed in 2009 dollars. The accuracy of all costs is order of magnitude. These estimates are approximations made without detailed engineering or site-specific data. Estimates of this type can be expected to vary from 50 percent less than to 30 percent more than actual final project costs.

The sources of construction cost data are:

- Construction cost data for the recent Colorado Front Range area projects and recent HDR designed projects, adjusted to 2009 dollars.
- Recent construction costs for other, similar facilities, adjusted to regional market conditions and 2009 dollars.
- Equipment pricing from manufacturers, including installation, structure, and housing costs.

A Order of Magnitude Cost Estimate was prepared for planning and decision purposes based on the Recommended Plan. This is an approximate estimate made without detailed engineering data. Some examples would be: an estimate from cost-estimating curves, an estimate using scale-up or scale-down factors, or an approximate ratio estimate. It is normally expected that an estimate of this type would be accurate within +50% or -30%.

5.8.2. Cost Estimating Assumptions

The Recommended Plan identified systems (Type A) and pipes (Type B) that would have to be removed and replaced, augmented, or regraded. In addition, future expansion pipes have been identified to collect flow from the North LTSA and western part of the WUSA. This information was broken down per problem or future infrastructure area. Pipe diameter, manhole diameter and improvement bury depths were taken into account. The capital project costs were individually analyzed based on the factors with stated assumptions below. The Budget Cost estimate is presented in 2009 dollars. Land acquisition, if needed, costs are not included.

Each heading is discussed in the order presented in the Budget Cost estimate sheets;

- Insurance and Bonding – 10 percent of Pipe Improvements Subtotal costs for contractor insurance and bonding.
- Mobilization – 6 percent of Pipe Improvement Subtotal costs for equipment mobilization and staging setup.
- Traffic Control – 5 percent of Pipe Improvement Subtotal costs for required traffic control during improvement construction.
- Utility Relocation – 5 percent of Pipe Improvement Subtotal costs for temporary relocation of utilities encountered during improvement construction.
- Dewatering – 5 percent of Pipe Improvement Subtotal costs for dewatering of open trenches due to pipe alignments elevations determined to be at or below the mapped 8 foot groundwater table.
- Bypass Pumping – 15 percent of Pipe Improvement Subtotal costs for temporary routing of wastewater flows for pipes removed in active system.
- Removal and Disposal – Cost based on pipe diameter of existing pipe and manhole to be removed and disposal. This does not include excavation of material above the pipe as this is reflected in the pipe diameter and depth cost component.
- Connect to Existing – Counts the number of and reflects the cost to connect existing main lines that remain to the proposed new/replacement manholes.
- Pipe – Shows the cost per diameter and depth of each pipe run within system. Budget costs reflect differences in excavation, bedding, and backfill quantities per pipe diameter. This cost includes controlled fill placement over pipe but does not include final surface treatment. All future expansion pipes are assumed to be 8-ft bury depth as exact design is unknown at this time.
- Stream Crossing – Stream and ditch crossings are difficult to estimate based on unknown costs for permitting, stream flow rates, surrounding improvements and channel confinement among other factors. For this budget level cost estimate, sewers crossing streams and ditches were assumed to be cased with a casing pipe diameter 18” greater than the carrier pipe diameter. Based on a review of the stream crossings, it was

assumed that the typical stream and ditch crossing would be 100' in length. Unit costs are based on inch-diameter/lineal foot for the casing pipe.

- Surface Restoration – Cost to restore surface to final condition, landscaped or hard paved. Assumes subgrade has been placed to appropriate elevation and density. The three surface restoration types assume a 12 foot surface restoration width regardless of the pipe diameter or depth. Asphalt depth was assumed to be 6 inches.
- Manholes – Cost to replace manholes based on manhole type and diameter. Manhole depth was assumed to be a standard 10 feet. Manholes for the future expansion projects in the figures are represented. Manhole placement every 400 ft was assumed for these future expansion projects. Actual number of manholes will likely vary when actual growth occurs and infrastructure design is completed.
- Lift Station and Forcemain – The only lift station and forcemain in the recommended plan is the one to serve the South Hill-n-Park WUSA area. For budget cost estimate purposes, the Southern Lift Station was assumed to have 3 pumps with an approximate total pumping capacity of 1,800 gpm. The Forcemain is 10-inch in diameter and is assumed to be constructed of High Density Polyethylene (HDPE). Additional planning and design is required to establish final lift station pumping capacity and forcemain sizes taking into account local wetwell storage, hourly and seasonal peaking factors, and more detailed loading information from updated landuse.
- Design Contingency – Type A and Growth Improvement Design Contingency is 30 percent, typical for budget planning efforts based on the detail of design data. Type B Improvement Design Contingency is 35 percent due to the averaging and lumping of improvements and therefore lost accuracy in Budget Costs.
- Engineering and Administration – This cost is a standard 20 percent cost, typical for this level of budget planning.

5.9. Recommended Improvement Plan

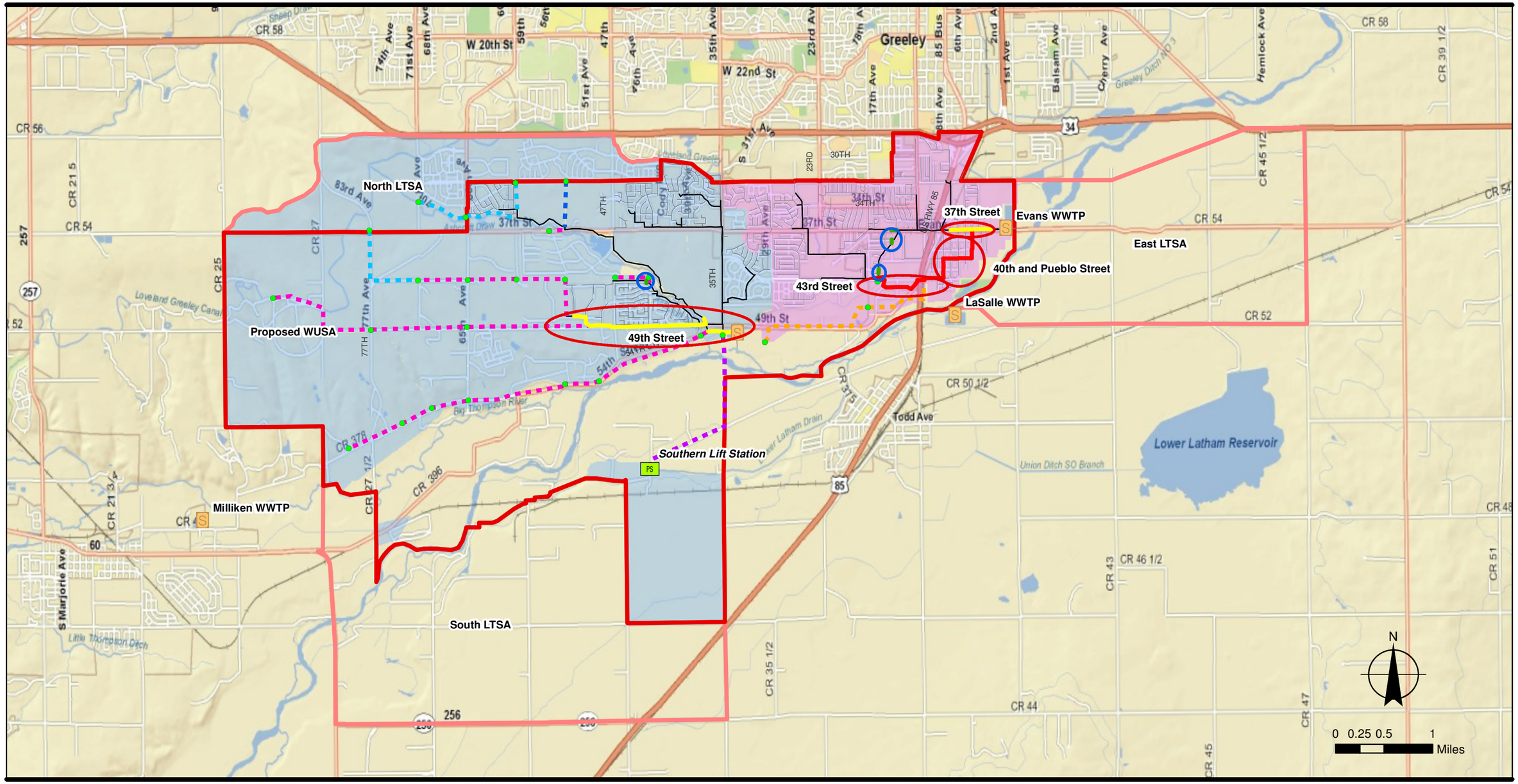
The Recommended Plan forms the basis of future improvements for the Evans and Hill-n-Park sanitary sewer collection systems. An ArcReader document containing system improvement elements, system design data (diameter, slope, etc) and hydraulic results has been established for future use in coordinating CIP projects. Recommended Plan fact sheets have been developed for the four Type A problem areas, Type B problems, and future growth-based expansion projects. The future expansion projects were divided into five distinct areas including Evans WUSA, West Hill-n-Park WUSA, South Hill-n-Park WUSA, North LTSA, and the T-Bone Lift Station Area. Figure 5-15 depicts the Recommended Plan with the project priority tier or future extension area assigned to each project.

The Budget level estimates of capital cost listed below are a summary of the individual detailed costs contained in Appendix B. Table 5-9 shows the Recommended Plan with

the improvement projects and their estimated costs sorted by priority tier and improvement type.

Table 5-9. Implementation Plan and Estimated Costs

Location	Improvement Type	Estimated Cost
Tier 1 Improvements		
40th and Pueblo Street	Problem	$1,502,000
43rd Street	Problem	$2,080,000
	Tier 1 Subtotal	**$3,582,000**
Tier 2 Improvements		
37th Street	Problem	$617,000
49th Street	Problem	$3,684,000
	Tier 2 Subtotal	**$4,301,000**
Tier 3 Improvements		
All Type B Improvements	Problem	$308,000
	Total Estimate of Problem Improvement Costs	**$8,191,000**
Future Expansion Projects		
Location	**Improvement Type**	**Estimated Cost**
Evans WUSA	Growth	$2,743,000
West Hill-n-Park WUSA	Growth	$12,973,000
South Hill-n-Park WUSA	Growth	$3,354,000
North LTSA	Growth	$2,931,000
T-Bone Lift Station Area	Service Transfer	$785,000
	Total Estimate of Future Expansion Costs	**$22,786,000**
	Total Estimate of Recommended Plan	**$30,977,000**



Legend

- Treatment Plants
- Future Manhole
- Future Lift Station
- Exist. Local Sewer (Unanalyzed)
- Exist. Interceptor/Collector Sewer (Analyzed)

Existing Pipe Improvements

- Tier 1
- Tier 2
- Tier 3 (Type B Problems)

Future Extension Areas

- Evans WWTF Area
- South Hill-n-Park WWTF Area
- West Hill-n-Park WWTF Area
- North LTSA
- T-Bone Lift Station Area

Service Area

- Planning Boundary
- Long-term Growth/Service Area

WWTF Basin

- Evans
- Hill-n-Park

- Type A Problem Areas
- Type B Problem Areas

Evans Wastewater Utility Plan

Recommended Plan Project Priority and Future Extension Areas

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Figure 5.15
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The fact sheets below capture various categories of information for the problem improvement projects such as technical data, data confidence, flow triggers, land ownership, implementation issues, and estimate of capital cost.

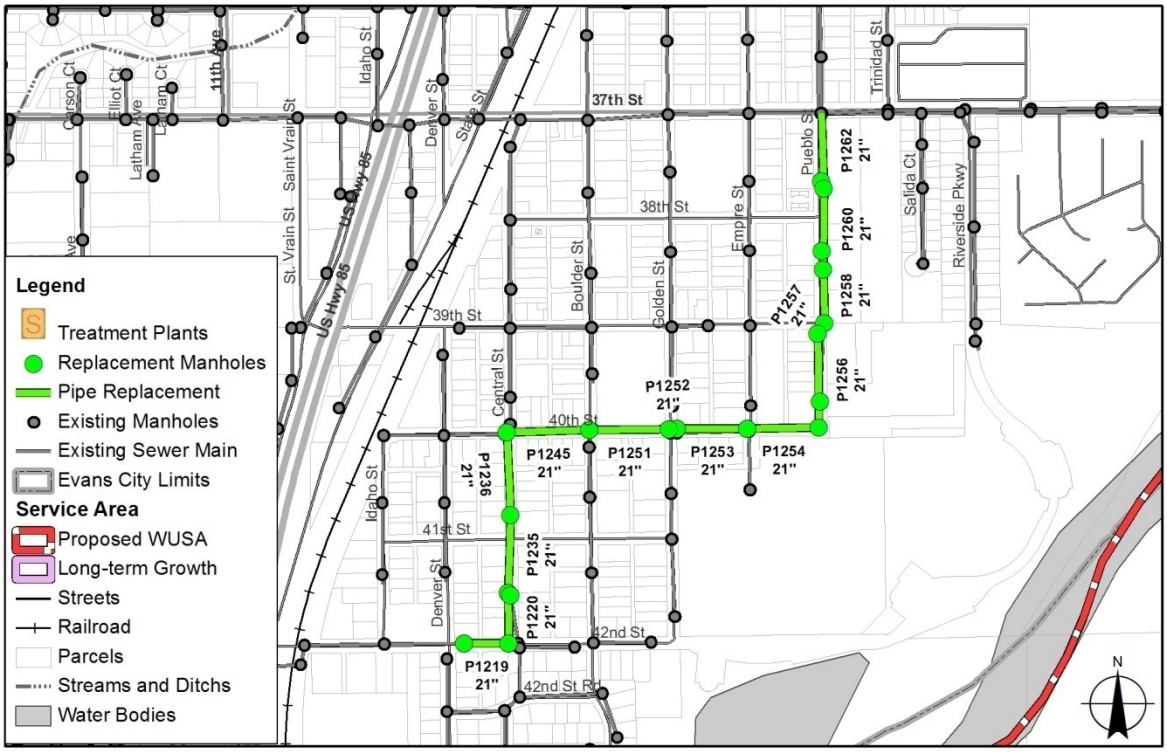
5.10. Hydraulic Model and Data Recommendations

It is recommended that when enough network data is obtained that the model be converted to an all pipes EPS model. The advantage of EPS over steady-state with peaking factors is that EPS allows the system hydraulics to establish flow attenuation and resulting peak hour flows and to provide additional information about the cumulative volumes of sewage. The base flow is multiplied by a factor based on the diurnal pattern at each one-hour time-step and infiltration and inflow (I/I) factors are also applied.

It is also recommended that an inflow and infiltration study be performed to better assess the system's response to rainfall and non-rainfall induced infiltration. The items below are general recommendations that may assist in improving flow allocation accuracy and work towards an EPS model in the future.

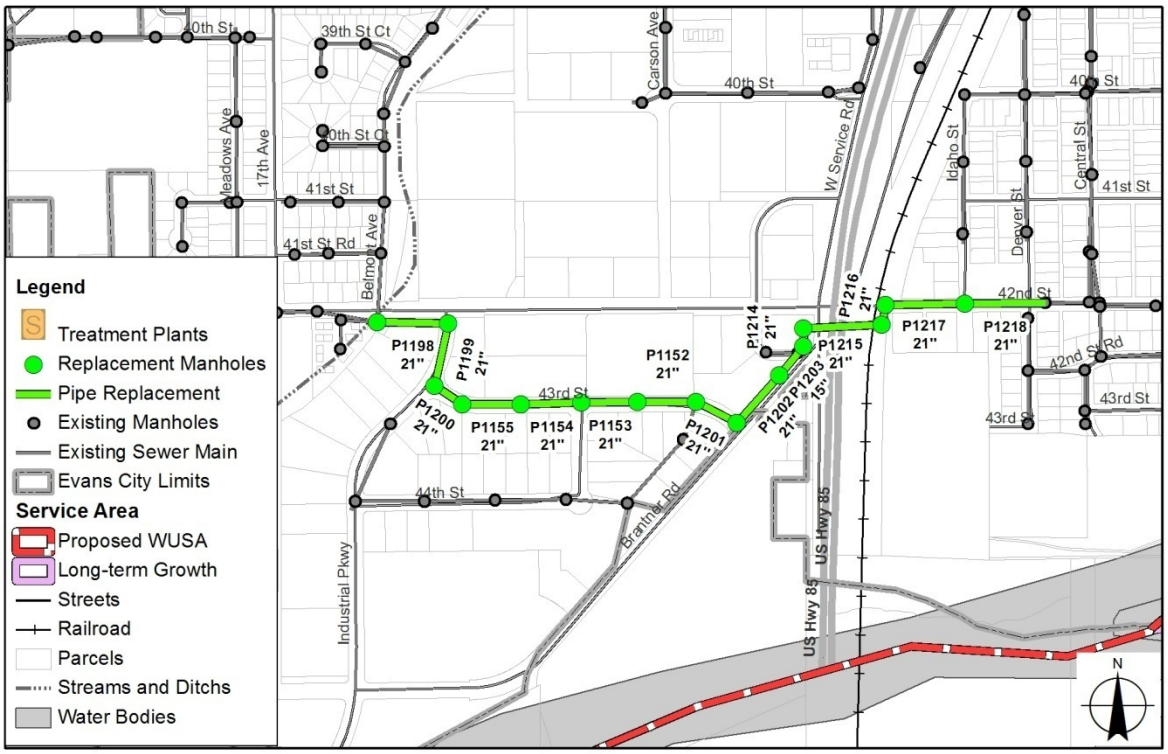
1. Evaluate water meter data and dry weather temporary flow meter data to better define the sanitary unit flow factors for population and employment.
2. Perform in-depth I/I study to characterize system-response to various frequencies of storm events. Consider include strength of sewage (BOD) testing and analysis in study to estimate percentage of groundwater (inflow and pumped) and rainfall-induced I/I within flow streams during dry-weather compared to wet-weather.
3. Install semi-permanent flow meters in number of locations strategically placed across system to capture wet-weather events to better understand system response to I/I during wet-weather events and to help define diurnal curves for the system on a sewer basin level during dry-weather weathers.
4. Evaluate flow meter data during dry-weather periods when the irrigation ditches are active to define the influence that these ditches have on groundwater-induced base infiltration.

PROBLEM IMPROVEMENTS – 40TH AND PUEBLO STREET	
	Discussion
Improvement Location:	Problem area located in Evans WWTF basin along Pueblo Street from 37th Street to 40th Street, and 40th Street from Pueblo Street to Central Street, and Central Street from 40th Street to 42nd Street.
Area ID:	**40th and Pueblo Street (Tier 1 Priority Level)**



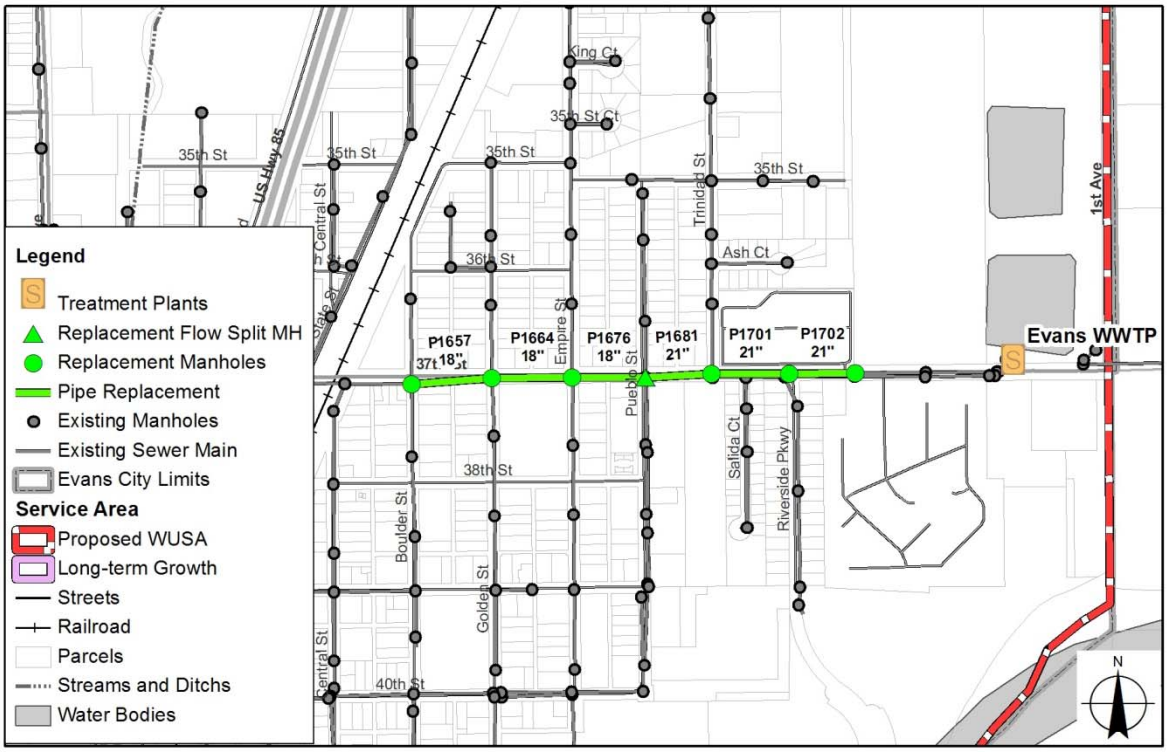
Improvement Description	Replacement - Replace 4,397 ft of 15" collector with 21" along Pueblo Street, 40th Street, and Central Street.
Technical Data:	The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow is approximately 1,700 gpm.
Data Confidence:	Medium
Flow Trigger:	Ranges from 300 gpm in P1219 to 1,500 gpm in P1262.
Land Ownership:	All construction appears to be within the established ROW.
Implementation Issues:	Constructability issues due to major roads. Requires construction along residential and commercial streets. Requires bypass pumping for all of project length. Surface treatment of concrete and asphalt pavement.
Capital Cost:	$1,502,000

PROBLEM IMPROVEMENTS – 43RD STREET	
	Discussion
Improvement Location:	Problem area located in Evans WWTF basin along 42nd Street from Central Street to US Hwy 85, and 43rd Street from US Hwy 85 to Industrial Parkway, and 42nd Street from Industrial Parkway to the Evans Town Ditch.
Area ID:	**43rd Street (Tier 1 Priority Level)**



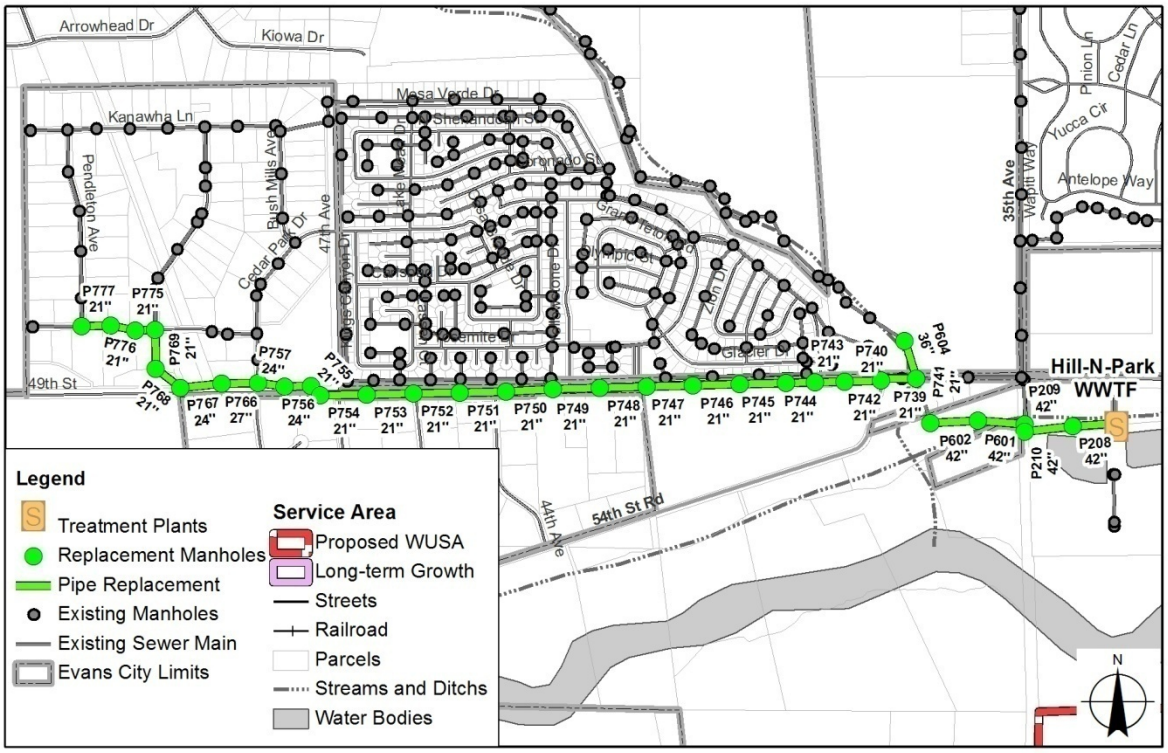
Improvement Description	Replacement – Replace 1,427 ft of 12" collector with 21", 2,496 ft of 15" collector with 21" and re-grade 144 ft of 15" collector along 42nd Street and 43rd Street.
Technical Data:	The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 1,200 to 1,600 gpm.
Data Confidence:	Medium
Flow Trigger:	Ranges from 430 gpm in P1202 to 1,450 gpm in P1216.
Land Ownership:	All construction appears to be within the established ROW.
Implementation Issues:	Constructability issues due to highways and major roads. Requires construction through commercial area. Requires bypass pumping for all of project length. Requires boring under US Highway 85. Surface treatment of concrete and asphalt pavement.
Capital Cost:	$2,080,000

PROBLEM IMPROVEMENTS - 37TH STREET AREA	
	Discussion
Improvement Location:	Problem area located in Evans WWTF basin along 37th Street from Boulder Street to Riverside Parkway
Area ID:	**37th Street (Tier 2 Priority Level)**



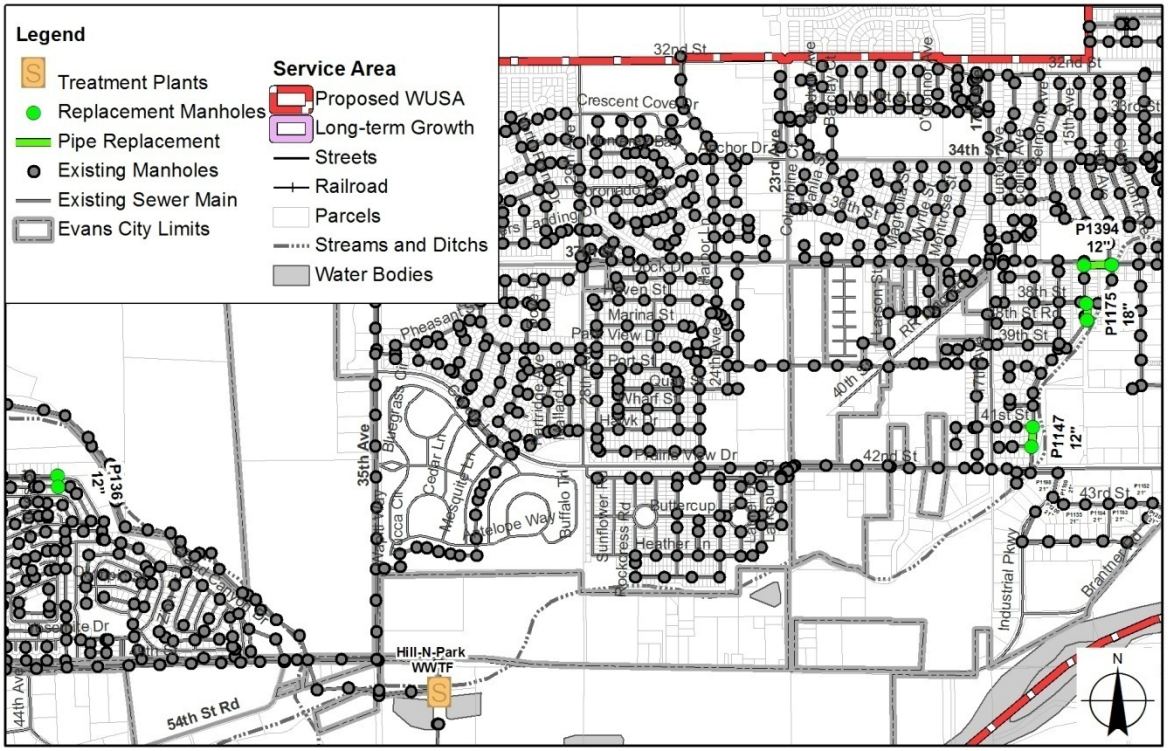
Improvement Description	Replacement - Replace 802 ft of 12" collector with 18", 360 ft of 12" collector with 21", and 1,044 ft of parallel 15" collector with 21" along 37th Street.
Technical Data:	The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 700 gpm in 18" to 2,000 gpm in 21".
Data Confidence:	High
Flow Trigger:	300 gpm (in 12" collector) and 1,400 gpm (in 15" collector).
Land Ownership:	All construction appears to be within the established ROW.
Implementation Issues:	Constructability issues due to highway and major roads. Requires construction along busy 37th Street. Requires bypass pumping of existing 12-inch portion. Possible use of existing 15-inch parallel mains instead of bypass pumping for 21-inch replacement portion. Surface treatment of concrete and asphalt pavement.
Capital Cost:	$617,000

PROBLEM IMPROVEMENTS – 49TH STREET	
	Discussion
Improvement Location:	Problem area located in Hill-n-Park WWTF basin along Dry Creek Rd from Pendleton Ave to Charleston Ave, and Charleston Ave from Dry Creek Rd to 49th St, and 49th Street from Charleston Ave to 54th Street, and across local unpaved roads to the Hill-n-Park WWTF.
Area ID:	**49th Street (Tier 2 Priority Level)**



Improvement Description	Replacement – Replace 300 ft of 8" collector with 21", 6,168 ft of 15" collector with 21", 813 ft of 15" collector with 24", 312 ft of 15" collector with 27", 334 ft of 30" interceptor with 36", and 1,639 ft of 30"interceptor with 42" along 49th Street.
Technical Data:	The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 1,950 to 4,440 gpm in the collector system and 10,000 to 12,600 gpm in the interceptor system.
Data Confidence:	Medium
Flow Trigger:	Ranges from 400 gpm in P740 to 11,300 gpm in P208 when growth occurs in West Hill-n-Park WUSA area.
Land Ownership:	All construction appears to be within the established ROW.
Implementation Issues:	Constructability issues due to major roads. Requires construction through residential area. Requires bypass pumping for all of project length. Surface treatment of concrete and asphalt pavement.
Capital Cost:	$3,684,000

PROBLEM IMPROVEMENTS – TYPE B PROJECTS	
	Discussion
Improvement Location:	Type B problems are located in Evans and Hill-n-Park WWTF basin on Mesa Verde Drive, at the intersection of Belmont Avenue and 42nd Street, at the intersection of Burlington Avenue and 38th Street, and at the intersection of 37th Street and Valmont Avenue.
Area ID:	**Type B (Tier 3 Priority Level)**



Improvement Description	Replacement – Replace 146 ft of 8" with 12" on Mesa Verde Drive. Replace 254 ft of 8" with 12" at the intersection of Belmont Avenue and 42nd Street. Replace 213 ft of 12" with 18" at the intersection of Burlington Avenue and 38th Street. Replace 323 ft of 8" with 12" at the intersection of 37th Street and Valmont Avenue.
Technical Data:	The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 60 to 370 gpm.
Land Ownership:	All construction appears to be within the established ROW.
Implementation Issues:	Constructability issues due to major roads. Requires construction through residential and commercial areas. Requires bypass pumping for all project locations. Surface treatment of concrete and asphalt pavement.
Capital Cost:	$308,000

6.0 Recommendations and Capital Improvements Plan

6.1. Introduction

The purpose of this chapter is to develop recommendations for improving the City of Evans wastewater system based on analysis completed in previous sections, prioritize recommendations, estimate the cost of recommended improvements, and provide an implementation plan in the form of a capital improvements schedule.

6.2. Public Participation

During the development of this Plan, opportunity for public participation was provided during three separate work sessions. The first was at the Evans Water and Sewer Board meeting held on November 18, 2010. The second and third opportunities were during City Council meetings held on February 1, 2011 and April 5, 2011. All meetings were advertised to the public. During each, presentations were given on the status of the Plan, recommendations, and the resulting capital improvements schedule.

6.3. Capital Improvement Program Prioritization

The City of Evans has limited resources to invest in wastewater infrastructure, making prioritization of capital improvement projects a necessity. Multiple criteria govern the prioritization of capital improvement projects. The following list highlights the criteria that dictate capital improvements priorities for wastewater treatment and conveyance:

- Permit Changes
- Surface Water Protection
- Protection of Public Health
- Collection and Treatment System Reliability
- Growth
- Coordination and Compatibility With Other Capital Programs
- Renewal and Replacement
- Regulatory Compliance

6.4. Capital Improvement Recommendations

Following is a summary of the recommended improvements the timing for design and construction. Costs for improvements were developed to provide the City of Evans with a budget cost estimate for the Recommended Plan.

6.4.1. Cost Estimating Rationale

An opinion of probable costs has been prepared for each alternative. The American Association of Cost Engineers has defined three basic categories of estimates in an effort to establish an expected accuracy range for various types of cost estimates. They include:

- *Order of Magnitude Estimate* – This is an approximate estimate made without detailed engineering data. Some examples would be: an estimate from cost-estimating curves, an estimate using scale-up or scale-down

factors, or an approximate ratio estimate. It is normally expected that an estimate of this type would be accurate within +50 percent or -30 percent.

- *Budget Estimate* – The term budget in this case applies to the owner's budget and not the budget as a project-control document. A budget estimate is prepared with the use of spreadsheets, layouts and equipment details. It is normally expected that an estimate of this type would be accurate within +30 percent or -15 percent.
- *Definitive Estimate* – As the name implies, this is an estimate prepared from well defined engineering data. At a minimum, the data must include: fairly complete plans and elevations, piping diagrams, equipment data sheets and quotations, structural sketches, soil data and a complete set of specifications. The "maximum" definitive estimate would be made from Approved for Construction" drawings and specifications. It is expected that a definitive estimate would be accurate within +15 percent or -5 percent.

Based on the current level of this project's development and engineering efforts, and considering that no subsurface (geotechnical) investigations have been performed, an order of magnitude estimate is the most accurate description of the probable project costs that can be expected. As site specific information is obtained and designs are refined, a more accurate cost opinion can be developed. The primary objective of the cost opinions presented here is to provide a basis for comparison of relative costs between alternatives.

This section summarizes the assumptions and possible unknowns related to cost items, and presents an opinion of probable project cost for each alternative. The comparison is concluded with an analysis of the probable cost of each alternative related to the overall benefit associated with each modification.

Capital costs are expressed in 2013 dollars. The accuracy of all costs is order of magnitude. These estimates are approximations made without detailed engineering or site-specific data. Estimates of this type can be expected to vary from 50 percent less than to 30 percent more than actual final project costs.

The sources of construction cost data are:

- Construction cost data for the recent Colorado Front Range area projects and recent HDR designed projects, adjusted to 2013 dollars.
- Recent construction costs for other, similar facilities, adjusted to regional market conditions and 2013 dollars.
- Equipment pricing from manufacturers, including installation, structure, and housing costs.

An Order of Magnitude Cost Estimate was prepared for planning and decision purposes based on the Recommended Plan. This is an approximate estimate made without detailed engineering data. Some examples would be: an estimate from cost-estimating curves, an estimate using scale-up or scale-down factors, or an approximate ratio estimate. It is normally expected that an estimate of this type would be accurate within +50 percent or -30 percent. All capital costs include allowances for sitework and yard piping; contractor

mark-up; contingencies; and engineering, legal and administration costs. The cost estimating procedure is presented in Table 6-1.

Table 6-1. Illustration of Cost Estimating Procedure

Cost Item	Cost
Base Construction Cost (Includes General Conditions)	$1,000
Electrical and Controls (20%-30% of Equipment Costs)	$300
Subtotal A	**$1,300**
Mobilization and Bonds (5% of A)	$65
Contractor's Overhead and Profit (10% of A)	$130
Subtotal B	**$1,495**
Miscellaneous Costs not Itemized (30% of B)	$448
Subtotal C (Construction Cost)	**$1,943**
Engineering, Legal, Administration (20% of C)	$389
Total Project Cost	**$2,332**

For most treatment processes, the economic comparison of alternatives is strongly driven by capital costs. Consequently, O&M costs are discussed only when there is a substantial difference in O&M requirements between the alternatives.

6.4.2. Recommended Treatment Alternatives

Chapter 3 analyzed future regulatory requirements. Chapter 4 analyzed treatment plant capacity and system deficiencies in detail and evaluated alternatives for expanding the Evans WWTF and Hill-n-Park WWTF capacity and eliminating deficiencies. It is recommended that expansion and improvement be accomplished utilizing a phased approach. For the purposes of this study two phases will be considered. Phase 1 includes improvements recommended to improve operations or are required to meet regulatory or permit requirements in the near future. Phase 1 improvements will be designed, for the most part, for a capacity and service life to 2025. This approach will allow the City to spread the cost of this work over a longer period of time, not build too much capacity too soon, and pass the cost of new capacity to new customers (growth). Phase 2 includes improvements recommended to expand capacity of processes and equipment beyond 2025.

Both treatment facilities face similar short and long term challenged including capacity limitations and future permit requirements. Following is a summary of the recommended improvements.

6.4.2.1. Evans WWTF Phase 1 Recommendations

Biological Process Improvements

The existing Evans WWTF is currently above 95 percent of rated capacity and has had difficulty at times removing ammonia during the winter and spring months. Additionally, the facility will not be able to meet future permit requirements for ammonia and nitrate.

As discussed further in Chapters 2 and 4, two flow scenarios were examined for the Evans WWTF:

1. Allow both the Evans and Hill-n-Park WWTFs to continue operating under the current flow split.
2. Construct a pump station at the Evans WWTF and pump all flow to the Hill-n-Park WWTF for treatment.

Based on capital cost, long term energy use, and overall constructability, flow scenario 1 (maintain current flow split) with treatment Alternative EB2/EB5 –MBBR/IFAS is recommended for implementation to meet the short and long term requirements. To meet future phosphorus limits, Alternative EB10 will be implemented. For this alternative, the expected wastewater loadings to the Evans WWTF are summarized in Table 6-2.

Table 6-2. Evans WWTF Existing and Projected Wastewater Loading

Criteria	Year 2010	Year 2020	Year 2030
Sewered Population	12,380	12,941	13,502
Average Dry-Weather Flow, MGD[1]	1.230	1.277	1.432
Peak Hour Dry-Weather Flow, MGD[2]	1.968	2.043	2.291
Peak Hour Wet-Weather Flow, MGD[3]	3.284	3.516	4.087
Influent BOD_5, lb/d[4]	2,874	2,984	3,346
Influent TSS, lb/d[4]	2,638	2,739	3,071
Influent Total Ammonia, lb/d[4]	328	341	382

[1]Yearly average from 2008 flow data.
[2]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[3]Calculated by adding the average dry-weather flow with peak hour storm I&I flow (I&I factor multiplied by the sewered area).
[4]Calculated by average dry-weather flow multiplied by average influent concentration.

This alternative requires a greater initial investment to meet the short term ammonia limits, however, it provides a much lower overall operations cost than the other alternatives and can be constructed without taking the existing treatment facility offline. Additionally, Alternative EB2/EB5 provides a higher degree of operational control and can be retrofitted in the future to meet lower effluent total nitrogen and phosphorus limitations, if and when required. The alternative consists of the following (Phase 1):

- Existing Polishing Pond will be dewatered, dredged, excavated and lined with a geomembrane liner.
- A portion of Aeration Lagoon No. 2 will filled in and an MBBR tank constructed.
- Polishing Pond will be used as the secondary clarifier.
- Construct blower building and air distribution header to new MBBR.
- Construct air delivery piping in new MBBR.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Replace standby generator with larger unit.
- Construct parallel outfall to allow for passage of the peak hour flow.

The estimated project cost for constructing the Phase 1 improvements to the biological process is approximately $4,040,000.

If the future regulations require the removal of total nitrogen and phosphorus, the MBBR system will be upgraded to and IFAS through the following (Phase 2):

- MBBR will be expanded through the construction of an anaerobic zone and anoxic zone upstream of the MBBR tanks and additional aerobic zones downstream. The MBBR tanks will be converted to IFAS tanks through the addition of suspended growth bacteria through return activated sludge. The anaerobic zone will provide for biological phosphorus removal while the anoxic tank will provide denitrification to convert nitrate to nitrogen gas.
- A new secondary clarifier will be constructed in the Polishing Pond to allow for the recycle of suspended growth bacteria to the IFAS tank.
- New effluent filtration will be constructed to remove additional phosphorus.

The estimated project cost for constructing Phase 2 is approximately $4,224,000 which results in a total biological improvements cost of $8,264,000.

Disinfection Improvements

The existing sodium hypochlorite disinfection system is not sized to treat flows above 1.2 MGD and will require expansion and/or replacement to treat peak hour flows once the biological improvements have been completed (currently the treatment lagoons act as flow equalization). Although capital costs for installation of UV light disinfection are greater than upgrading the chemical disinfection system, there are considerable advantages for the City to consider installation of UV light disinfection. UV light is less susceptible to market fluctuations for chemicals, carries less risk for toxic chemical spill and provides greater assurance that future regulatory requirements are met. Since capacity to the existing chlorination system will need to be increased, it is recommended the City construct Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection System to alleviate capacity limitations and to meet future, more stringent, permit requirements. The alternative consists of the following:

- Two parallel UV channels will be installed in the existing chlorine contact channel.
- Level control facilities, lamp removal hoists and monorails, and lamp cleaning systems will be installed.
- A portion of the existing chlorine storage room will be used for UV lamp maintenance and cleaning purposes, and for installation of the UV electrical service equipment.
- A building will be constructed over the UV channels to provide for weather protection and equipment maintenance.

The estimated project cost for constructing the improvements to the disinfection system is approximately $643,000.

Evans WWTF Phase 1 Improvements Summary

Based on the recommended alternatives, the overall project cost for the Evans WWTF Phase 1 improvements is approximately $4,683,000. Table 6-3 provides a summary of the annual cost expenditures.

Table 6-3. Evans WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 15-16
Phase 1 Improvements			
Biological Process Improvements[2]	-Capacity above 90% -Ammonia limits	Design $673,000	Construction $3,367,000
UV Disinfection	-Capacity above 90%	Design $107,000	Construction $536,000
	Totals	**$780,000**	**$3,903,000**
	Overall Total	**$4,683,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, construction of parallel outfall, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

Evans WWTF Phase 2 Improvements Summary

As mentioned previously, it is anticipated the Evans WWTF will need to be upgraded in the future to meet the Regulation 31 total nitrogen and total phosphorus limits. Based on the recommended alternatives, the overall project cost for the Evans WWTF Phase 2 improvements is approximately $4,224,000. Improvements include expanding the capacity of the MBBR treatment system and converting it to an IFAS through the construction of a secondary clarifier. Effluent filtration would also be added at this time to remove phosphorus. Table 6-4 provides a summary of the annual cost expenditures.

Table 6-4. Evans WWTF Phase 2 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]
		FY 23-28
Phase 2 Improvements		
Biological Process Improvements[2]	-Reg 31 TN and TP effluent limits	Design $704,000 Construction $3,520,000
	Overall Total	**$4,224,000**

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, construction of parallel outfall, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

6.4.2.2. Hill-n-Park WWTF Recommendations

Biological Process Improvements

The Hill-n-Park WWTF is currently rated for 0.99 MGD and it is expected the future flows to the Hill-n-Park WWTF will increase significantly. Additionally, it is expected future effluent limits will require the removal of ammonia, total nitrogen and total phosphorus. The City recently installed improvements to the Hill-n-Park WWTF to increase the capacity from 0.5 MGD to 0.99 MGD. The short term improvements allow for additional ammonia removal and long term to increase capacity and meet nitrogen/phosphorus limits. Table 6-5 provides a summary of the expected wastewater loading projections to the Hill-n-Park WWTF.

Table 6-5. Hill-n-Park WWTF Existing and Projected Wastewater Flow and Loading (Maintain Current Flow Split Between WWTFs)

Criteria	2010	Year 2020	Year 2030
Population	9,366	20,305	28,483
Average Dry-Weather Flow, MGD[1]	0.491	1.949	2.902
Peak Hour Dry-Weather Flow, MGD[2]	0.687	2.729	4.063
Peak Hour Wet-Weather Flow, MGD[3]	1.660	7.968	13.982
Influent BOD_5, lb/d[4]	1,134	4,554	6,780
Influent TSS, lb/d[4]	1,012	4,180	6,223
Influent Total Ammonia, lb/d[4]	122	520	775

[1]Yearly average from 2008 flow data.
[2]Calculated from the average dry-weather flow multiplied by the established peak hour peaking factor.
[3]Calculated by adding the average dry-weather flow with peak hour storm I&I flow (I&I factor multiplied by the sewered area).
[4]Calculated by average dry-weather flow multiplied by average influent concentration.

Due to the lower operations cost and ability to consistently meet stringent effluent nutrient limits, it is recommended Alternatives HB2/HB4 be carried forward as a place holder for further evaluation in the future when the improvements are required. Similar to Alternative EB2/EB5 for the Evans WWTF, this alternative constructs an MBBR/IFAS system. Additionally, this alternative will be phased to meet future flow and regulatory requirements. The improvements consist of the following:

- A new secondary clarifier would be constructed in the Polishing Lagoon.
- A portion of Aeration Lagoon No. 2 will filled in and an IFAS tank constructed.
- Construct blower building and air distribution header to new IFAS.
- Construct air delivery piping in new IFAS.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Construct new effluent filtration to meet effluent total phosphorus limit.
- Construct parallel outfall to allow for passage of the peak hour flow.
- Expand capacity of Headworks and add grit removal.

The estimated total project cost this alternative is $15,020,000 (this includes constructing an IFAS system, secondary clarifier, effluent filtration, improvements to the headworks, and blower building).

Disinfection Improvements

The disinfection improvements for the Hill-n-Park WWTF are similar to those previously mentioned for the Evans WWTF. To meet future capacity requirements, the existing sodium hypochlorite disinfection system will be replaced with Alternative HD2-Convert Sodium Hypochlorite Disinfection System to UV Disinfection.

The estimated project cost for constructing the improvements to the disinfection system is approximately $1,565,000.

Standby Power

The Hill-n-Park WWTF does not currently have backup power. Installation of a standby power generator is recommended for critical equipment required for maintaining the biological process. It is recommended that an in-place engine generator with an automatic transfer switch be installed at the Hill-n-Park WWTF.

The estimated project cost for installation of standby power is approximately $224,000. This cost is already included in the cost developed for Alternative HB5.

Hill-n-Park WWTF Phase 1 Improvements Summary

Due to the lower flow at the Hill-n-Park WWTF and the recent rerating to 0.99 MGD, only a few improvements are required for Phase 1 including replacement of the existing headworks equipment and addition of standby power. The overall project cost for the Hill-n-Park WWTF Phase 1 improvements is approximately $424,000. Table 6-6 provides a summary of the annual cost expenditures.

Table 6-6. Hill-n-Park WWTF Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 13-14	FY 14-15
Phase 1 Improvements			
Hill-n-Park WWTF Headworks Screening Improvements	-Existing equipment is at end of useful life	Construction $200,000	
Hill-n-Park WWTF Standby Power	-Required to meet regulatory requirements		Construction $224,000
	Totals	**$200,000**	**$224,000**
	Overall Total	**$424,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.

6.4.2.3. Hill-n-Park WWTF Phase 2 Recommendations

Biological Process Improvements

The Phase 1 improvements are expected to address issues at the Hill-n-Park WWTF for the next 10 to 15 years. To meet capacity and regulatory requirements beyond this time frame, the Hill-n-Park WWTF will require expansion and conversion to a more robust treatment system to bring the average dry-weather capacity to 3 MGD. Phase 2 improvements will include the following:

- Improvements to headworks including grit removal.
- New IFAS reactor, secondary clarifier, and blower building.
- Convert either Aeration Lagoon No. 1 or the remaining area of Aeration Lagoon No. 2 to sludge storage and stabilization lagoon. Line lagoon with geomembrane liner and relocate unused existing surface aerators to lagoon or convert to a facultative lagoon.
- Construct new effluent filtration to meet effluent total phosphorus limit.
- Construct parallel outfall to allow for passage of the peak hour flow.
- UV disinfection.

The project cost of constructing the Phase 2 biological process improvements is approximately $15,020,000.

Support Facilities

Additionally, it is expected the construction of the new operations/laboratory/maintenance building will be required to meet the additional operational needs. Key support facilities include the following:

- Maintenance facilities
- Operations personnel offices
- Laboratory facilities

To provide the appropriate space for the above facilities, it is estimated a minimum of 6,000 sf is required. Estimated project cost for this alternative is approximately $1,800,000.

Hill-n-Park WWTF Phase 2 Improvements Summary

Based on the recommended alternatives, the overall project cost for the Hill-n-Park WWTF Phase 2 improvements is approximately $17,717,000. Table 6-7 provides a summary of the Phase 2 annual cost expenditures.

Table 6-7. Hill-n-Park WWTF Phase 2 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]	
		FY 18-23	FY 23-28
Phase 2 Improvements			
Biological Process Improvements[2]	-Capacity above 80% (i.e. increase of flows above 0.8 MGD) -Reg 31 TN and TP effluent limits	Design $2,758,000 Construction $12,516,000	
UV Disinfection	-Revised Federal Ammonia Criteria- ammonia limit lowered below 1 mg/L -No longer use chloramine disinfection	Design- $107,000 Construction- $536,000	
Operations Building	-Required for process control		Design- $360,000 Construction- $1,440,000
	Totals	**$15,917,000**	**$1,800,000**
	Overall Total	**$17,717,000**	

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes.
[2] Cost estimate includes the following: contractor general conditions, site work with onsite disposal of excavation, use of prefabricated metal buildings when needed, and piping. Project cost does not include equipment replacement in Headworks Building.

6.4.3. Recommended Conveyance System Improvements

The proposed planning boundary contains approximately 8,873 acres which are served by 75 miles of existing sanitary sewer pipes that are divided into two sewer basins. The existing sewer collection system consists entirely of gravity sewer pipes that convey flow to the Evans and Hill-n-Park WWTFs. Generally, sewers located east of 29th Ave flow to the Evans WWTF with the remaining portion of the western section of the City flowing to the Hill-n-Park WWTF.

A hydraulic analysis using the calibrated collection system model was performed to locate problem areas during dry-weather and wet-weather scenarios both under existing and buildout conditions. Problem areas were identified by the following criteria:

1. Capacity: Local Collector / Collector System (8 IN – 18 IN)
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of one-half of the full pipe (d/D = 50 percent)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100 percent)
2. Capacity: Interceptor (> 18 IN) System
    a. Sanitary sewer overflows (SSO)
    b. Average dry-weather - flow equal to depth of 70 percent of the full pipe (d/D = 70 percent)
    c. Peak-hour wet-weather - flow depth equal to the full pipe (d/D = 100 percent)

3. Velocities: Entire Analyzed System

   a. Peak-hour dry-weather – Minimum velocity equal to 2 fps.
   b. Peak-hour wet-weather – Maximum velocity equal to 10 fps.

The base buildout scenario in the InfoSewer model was used to properly size improvements based on only the proposed planning boundary and the Northern portion of the LTSA. The modeling did not include the South and East LTSAs as these are outside the 20-year planning horizon. The North LTSA includes an IGA with Greeley and it is anticipated this transfer of service area from City of Greeley to City of Evans will occur in the near future. The WWTF basin boundaries were not modified from the original locations. Flow was not reduced to the Evans WWTF so no Central Lift Station is required. The Southern Lift Station and forcemain are still required because of the buildout area in the South planning boundary that can not be delivered to the Hill-n-Park WWTF by gravity. The North LTSA flow is conveyed by gravity to the Hill-n-Park WWTF via the Ashcroft Draw interceptor.

The potential problem pipes established from the model results were examined to identify likely hydraulic issues under the various flow scenarios. There are a total of 60 pipes in the existing system with hydraulic deficiencies identified including 18 problem pipes under existing flow conditions. The hydraulic problems were separated into two categories; Type A and Type B. Type A problems consisted of a series of problem pipes and that were hydraulically connected to one another. Type B problems are isolated hydraulic restrictions that are not hydraulically connected to other problem locations or series of problem pipes. Type A problems accounted for 93 percent of the problem pipes or a total of 55 pipes with a cumulative length of approximately 3.3 miles of pipe. Type B problems accounted for the remaining 7 percent of problem pipes or a total of 5 pipes with a cumulative length of approximately 0.2 miles of pipe. Both Type A and Type B problems have system improvement recommendations developed and estimates of capital cost prepared. It is envisioned that Type A problems are identified as recommendations for capital improvements while the Type B problems are addressed through coordination with development activities, system rehabilitation and replacement R&R work, or replacement with other department CIP projects.

Problem areas are grouped in three tiers to establish implementation priority: Tier 1 problem areas are Type A improvements and have the highest priority; Tier 2 projects are also Type A improvements but have lower priority compared to Tier 1; and Tier 3 projects are Type B improvements which have the lowest priority. The priorities were assigned based on a number of qualitative factors including the extent of the problem and the flow conditions in which they occur, potential for SSOs, ease of constructability, and data confidence. It is anticipated that Tier 1 projects would be scheduled for construction within the next 10 years. Tier 2 projects would be completed in the following 10-years within the 20-year planning horizon or before as opportunities arise associated with other utility or roadway projects in the project vicinity. It is anticipated that Tier 3 projects could be addressed as minor capital projects or with system rehabilitation work in the project vicinity. Future infrastructure expansion due to growth is expected to be constructed as development occurs.

6.4.3.1. Conveyance Phase 1 Improvements

40th and Pueblo Street

Problem area located in Evans WWTF basin along Pueblo Street from 37th Street to 40th Street, and 40th Street from Pueblo Street to Central Street, and Central Street from 40th Street to 42nd Street. Due to small pipe slopes limited by shallow pipe depths there are capacity problems in existing wet-weather and buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow is approximately 1,700 gpm. Improvements include replacement of 4,397 ft of 15" collector with 21" along Pueblo Street, 40th Street, and Central Street. Estimated project cost for this alternative is approximately $1,502,000.

43rd Street

Problem area located in Evans WWTF basin along 42nd Street from Central Street to US Hwy 85, and 43rd Street from US Hwy 85 to Industrial Parkway, and 42nd Street from Industrial Parkway to the Evans Town Ditch. Due to shallow pipe slopes limited by shallow pipe depths there are capacity problems in existing and buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 1,200 to 1,600 gpm. Improvements include replacement of 1,427 ft of 12" collector with 21", 2,496 ft of 15" collector with 21" and re-grade 144 ft of 15" collector along 42nd Street and 43rd Street. Estimated project cost for this alternative is approximately $2,080,000.

Conveyance Phase 1 Improvements Summary

Based on the recommended alternatives, the overall project cost for the conveyance Phase 1 improvements is approximately $3,582,000. Table 6-8 provides a summary of the Phase 1 annual cost expenditures.

Table 6-8. Conveyance Phase 1 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]
		FY 23-28
Phase 1 Improvements		
40th and Pueblo Street	-Flow ranges from 300 gpm in P1219 to 1,500 gpm in P1262	Design- $300,000 Construction-$1,202,000
43rd Street	-Flow ranges from 430 gpm in P1202 to 1,450 gpm in P1216	Design- $416,000 Construction- $1,664,000
	Overall Total	**$3,582,000**

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting

6.4.3.1. Conveyance Phase 2 Improvements

37th Street Area

Problem area located in Evans WWTF basin along 37th Street from Boulder Street to Riverside Parkway. Due to small pipe slopes limited by shallow pipe depths there are capacity problems in existing and buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 700 gpm in 18" to 2,000 gpm in 21". Improvements include replacement of 802 ft of 12" collector with 18", 360 ft of 12" collector with 21", and 1,044 ft of parallel 15" collector with 21" along 37th Street. Estimated project cost for this alternative is approximately $617,000.

49th Street

Problem area located in Hill-n-Park WWTF basin along Dry Creek Rd from Pendleton Ave to Charleston Ave, and Charleston Ave from Dry Creek Rd to 49th St, and 49th Street from Charleston Ave to 54th Street, and across local unpaved roads to the Hill-n-Park WWTF. Due to growth and increased buildout flow there are capacity problems in buildout dry- and wet-weather flow scenarios. The entire system is required to convey buildout dry-weather and wet-weather flows to the established analysis criteria. Expected peak flow ranges from 1,950 to 4,440 gpm in the collector system and 10,000 to 12,600 gpm in the interceptor system. Improvements include replacement of 300 ft of 8" collector with 21", 6,168 ft of 15" collector with 21", 813 ft of 15" collector with 24", 312 ft of 15" collector with 27", 334 ft of 30" interceptor with 36", and 1,639 ft of 30"interceptor with 42" along 49th Street. Estimated project cost for this alternative is approximately $3,684,000.

Conveyance Phase 2 Improvements Summary

Based on the recommended alternatives, the overall project cost for the conveyance Phase 2 improvements is approximately $4,301,000. Table 6-9 provides a summary of the Phase 2 annual cost expenditures.

Table 6-9. Conveyance Phase 2 Capital Improvements Summary

Project Description	Triggers	Estimated Cost[1]
		FY 28-32
Phase 2 Improvements		
37th Street Area	-Flow ranges from 300 gpm (in 12" collector) and 1,400 gpm (in 15" collector)	Design- $123,000 Construction- $494,000
49th Street	-Flow ranges from 400 gpm in P740 to 11,300 gpm in P208 when growth occurs in West Hill-n-Park Planning Boundary area	Design- $737,000 Construction- $2,947,000
	Overall Project	**$4,301,000**

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting.

6.5. Capital Improvements Schedule

Table 6-10 summarizes the recommended the overall capital improvements by year.

Table 6-10. Capital Improvements Summary

Project Description	Estimated Cost[1]							
	Trigger	FY 13-14	FY 14-15	FY 15-16	FY 16-17	FY 18-23	FY 23-28	FY 28-32
Hill-n-Park WWTF Headworks Screening Improvements	-Existing equipment is at end of useful life	Construction $200,000						
Evans WWTF Biological Process Improvements – Phase 1	-Capacity above 90% -Ammonia limits	Design $673,000	Construction $3,367,000					
Evans WWTF UV Disinfection	-Capacity above 90%	Design $107,000	Construction $536,000					
Hill-n-Park WWTF Standby Power	-Required to meet regulatory requirements		Construction $224,000					
Hill-n-Park WWTF Biological Process Improvements	-Capacity above 80% (i.e. increase of flows above 0.8 MGD) -Reg 31 TN and TP effluent limits					Design $2,758,000 Construction $12,516,000		
Hill-n-Park WWTF UV Disinfection	-Revised Federal Ammonia Criteria-ammonia limit lowered below 1 mg/L -No longer use chloramine disinfection					Design $107,000 Construction $536,000		
40th and Pueblo Street	-Flow ranges from 300 gpm in P1219 to 1,500 gpm in P1262						Design $300,000 Construction $1,202,000	
43rd Street	-Flow ranges from 430 gpm in P1202 to 1,450 gpm in P1216						Design $416,000 Construction $1,664,000	

Project Description	Estimated Cost[1]							
	Trigger	FY 13-14	FY 14-15	FY 15-16	FY 16-17	FY 18-23	FY 23-28	FY 28-32
Evans WWTF Biological Process Improvements – Phase 2	-Reg 31 TN and TP effluent limits						Design $704,000 Construction $3,520,000	
Operations Building	-Required for process control						Design $360,000 Construction $1,440,000	
37th Street Area	-Flow ranges from 300 gpm (in 12" collector) and 1,400 gpm in (15" collector)							Design $123,000 Construction $494,000
49th Street	-Flow ranges from 400 gpm in P740 to 11,300 gpm in P208 when growth occurs in West Hill-n-Park Planning Boundary area							Design $737,000 Construction $2,947,000
	Yearly Totals	**$980,000**	**$4,127,000**	**-**	**-**	**$15,917,000**	**$9,606,000**	**$4,301,000**
Total Project Costs		**$34,931,000**						

[1] All costs are in 2013 dollars. Estimates should be escalated to the mid-point of construction for budgeting purposes
Costs include allowances for construction contingency, engineering, legal, and fiscal costs.

7.0 Financial Evaluation

7.1. Introduction

The purpose of this Chapter is to address financial planning to implement the recommendations of the facility plan. This chapter begins with a general discussion of current City of Evans wastewater utility financing and includes an assessment of the potential impact of capital improvements on the current user charge system. Potential outside funding sources are discussed, including federal and state loans and grants. Finally, it concludes with a summary of the implementation plan and recommendations for pursuing the modifications and improvements identified within this Utility Plan.

7.2. Program Costs

Chapter 4 presented a summary of estimated capital and operating costs for a variety of alternative wastewater management programs. These are preliminary, planning level costs and are subject to change as the implementation process proceeds and facility requirements become further refined through more detailed engineering. Alternatives were compared in Chapter 4 in terms of evaluation criteria, including capital and O&M costs, in order to assess the relative strengths and weaknesses of each option and recommendations were made with regard to a phased approach for improving and expanding the City's wastewater facilities. Chapter 5 outlined the recommended plan and presented a schedule for implementing the plan. The impact on user charges (rates and system development fees) resulting from the recommended capital improvement plan must be evaluated. Therefore a financial evaluation was developed that projects the needs to adequately fund program implementation.

7.3. Capital Improvement Program

Chapter 6 presented a scheduled plan for implementing capital improvements to the City of Evans wastewater system. Table 7-1 summarizes the total recommended capital improvement costs by year.

Table 7-1. Capital Improvement Plan Summary

Estimated Cost							
Year	FY 13-14	FY 14-15	FY 15-16	FY 16-17	FY 18-23	FY 23-28	FY 28-32
Total	$980,000	$4,127,000	-	-	$15,917,000	$9,606,000	$4,301,000

7.4. City of Evans Wastewater Utility Financing

The existing City of Evans wastewater system is funded through user charges which pay for on-going operations and maintenance expenses and modest facility improvements. Both sanitary sewer and municipal water utilities are funded in this manner. The following sections discuss the current wastewater budgets and user charges.

7.4.1. Existing Budgets

The annual operations and maintenance budget for the City of Evans sewer utility totals approximately $1,129,325, based on the 2012 budget. Of the total budget, personnel expenses for operations staff salaries and benefits amount to approximately $293,711. Operations, maintenance, and supplies total approximately $310,765. An annual debt service payment of $111,708 principle and interest is scheduled on a debt balance of $115,454 as of 2012. A transfer to the City's general fund in the amount of $413,141 is made for administrative services provided by general City services. The City has also instituted an Asset Management Plan beginning with the 2010 budget which is intended to provide funds for a portion of the City's renewal and replacements necessary to maintain reasonable operating condition of its infrastructure. For the sewer utility, this amount is budgeted at $10,763 for 2010. Net revenues for the 2012 fiscal year amount to approximately $1,127,280. The net result of income to expenditures results in a net operating loss of $2,044.

Cash reserves for capital projects are currently $297,181. In addition, $90,984 in sewer system development fee revenue is projected for connection to the sanitary sewer system in 2012. These fees are designed to pay for the investment the City has in excess capacity in the existing system and to fund expansion of the system to serve growth. These fees, combined with capital project cash reserves, total $388,165.

7.4.2. Existing User Charges

User charges to customers provide the needed revenue to operate the City of Evans wastewater utility. User charges are comprised of monthly rates and sewer development fees (connection fees). The current customer base consists of approximately 8,280 sewer connections.

Residential customers are charged a sewer rate based on the following:

- Service Tap Fee per month = $11.15

For commercial sewer service, the rates are based on the following:

- Service Tap Fee per month = $4.31
- Volume Charge = $1.08 per 1,000 gallons of potable water metered
- Total Sewer Rate=Base Service Charge + Volume Charge

The sewer rate results in an average charge of approximately $220.85 per month per commercial connection.

7.4.2.1. Sewer System Development Fees

The City Council has adopted by Resolution 45-2009, sewer system development fees. The resolution calls for a sewer system development fees that vary based upon the size of the water tap size. Table 7-2 presents the existing sewer system development fees for various tap sizes.

Table 7-2. Existing Sewer System Development Fees

Water Tap Size	Meter Equivalent	Inside City Sewer System Development Fee	Outside City Sewer System Development Fee
¾”	1	$3,636.50	$5,454.75
1”	1.67	$6,072.96	$9,109.43
1 ½”	3.33	$12,109.55	$18,164.32
2”	5.33	$19,382.55	$29,073.82
3”	10.67	$38,801.46	$58,202.18
4”	16.67	$60,620.46	$90,930.68
6”	33.33	$121,204.55	$181,806.82
8”	53.33	$193,934.55	$290,901.82
10”	126.67	$460,635.46	$690,953.18
12”	166.67	$606,095.46	$909,143.18

7.4.3. Financial Evaluation

A brief financial analysis has been undertaken in order to estimate the user charges necessary to support the capital improvement program (CIP) outlined in Chapter 6. Capital improvements of any significant magnitude will require additional funding as described below. System development fees are set at a level to provide the City with growth based funding for expansion of the wastewater collection and treatment system.

7.4.3.1. Historical Financial Analysis – Fiscal Years 2005 through 2012

The City of Evans provided historical financial documents detailing revenues and expenses for the five year period from 2005 through 2012. The results of the City’s historical financial analysis are provided in Table 7-3. From this analysis, it appears as though rate revenues were not sufficient to cover current operations and maintenance expenses, debt service and administrative transfers in the period from 2005 through 2008. However, in 2009, the City showed a balance of funds after O&M, debt service and transfers. This is due to the adjustment of rates accomplished in Resolution 45-2009.

Table 7-3. Historical Financial Analysis

Account Name	FY 2005	FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011	FY 2012
REVENUE								
Rate Revenues	$844,478	$876,369	$898,149	$918,467	$1,094,250	$1,022,094	$1,027,800	$1,125,063
Non-Operating Revenues	$65,852	$113,993	$162,961	$85,723	$17,442	$11,029	$2,994	$2,218
Total Source of Funds	$910,330	$990,362	$1,061,110	$1,004,190	$1,111,692	$1,033,123	$1,030,794	$1,127,281
EXPENSES								
Personnel	$0	$0	$0	$0	$0	$289,261	$284,713	$293,711
Operations	$1,113,429	$1,001,070	$964,399	$557,334	$672,257	$300,002	$310,895	$310,765
Total Operations & Maintenance	$1,113,429	$1,001,070	$964,399	$557,334	$672,257	$589,263	$595,608	$604,476
TOTAL DEBT SERVICE	$108,486	$184,643	$184,602	$184,167	$109,687	$115,005	$113,361	$111,708
CAPITAL IMPROVEMENTS FUNDED THROUGH RATES	$0	$0	$0	$0	$0	$0	$0	$0
TRANSFER TO OTHER FUNDS	$0	$245,206	$264,288	$334,002	$289,528	$363,517	$412,824	$413,141
TOTAL CHANGE IN RESERVE FUNDS	$0	$0	$0	$0	$0	$0	$0	$0
TOTAL REVENUE REQUIREMENTS	$1,221,915	$1,430,919	$1,413,289	$1,075,503	$1,071,472	$1,067,785	$1,121,793	$1,129,325
Balance/(Deficiency) of Funds	$(311,585)	($440,557)	($352,179)	($71,313)	$40,220	($34,662)	($90,999)	($2,044)

7.4.3.2 Projected Future Financial Analysis– Fiscal Years 2013 through 2030

The impact of financing the recommended capital improvements summarized in Table 6-10 was evaluated against current user rates and fees. The largest impact to rates is directly related to the projected cost and timing of the Evans and Hill-n-Park WWTF improvement projects. In the evaluation, it was assumed that growth would continue at a rate of 5 percent per year as predicted in Chapter 2, and the cost of operating and maintaining the system would also increase at an average rate of 3 percent per year.

Table 7-4. Summary Revenue Requirements Analysis (2013 Through 2030)

CITY OF EVANS
SEWER- EXHIBIT 1
REVENUE REQUIREMENT SUMMARY

SEWER REVENUE REQUIREMENT SUMMARY
PROJECTED 2013 - 2030

Account Name	FY 2013	Projected FY 2014	FY 2015	FY 2016	FY 2017	FY 2018	FY 2019	FY 2020	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030
REVENUE																		
Rate Revenues	$1,216,525	$1,222,608	$1,228,721	$1,234,864	$1,241,039	$1,250,346	$1,259,724	$1,272,321	$1,285,044	$1,297,895	$1,310,874	$1,323,983	$1,337,222	$1,350,595	$1,364,101	$1,377,742	$1,391,519	$1,405,434
Non-Operating Revenues	4,955	6,465	8,287	9,910	12,170	12,305	12,524	13,481	14,805	16,066	17,094	16,300	16,625	17,384	18,016	18,712	19,315	19,699
Total Sources of Funds	**$1,221,480**	**$1,229,073**	**$1,237,008**	**$1,244,774**	**$1,253,208**	**$1,262,652**	**$1,272,248**	**$1,285,802**	**$1,299,849**	**$1,313,961**	**$1,327,968**	**$1,340,282**	**$1,353,847**	**$1,367,978**	**$1,382,117**	**$1,396,454**	**$1,410,834**	**$1,425,134**
EXPENSES																		
Personnel	328,280	339,770	351,662	363,970	376,709	389,894	403,540	417,664	432,282	447,412	463,071	479,279	496,054	513,415	531,385	549,984	569,233	589,156
Operations	336,256	346,344	356,734	367,436	378,459	389,813	401,507	413,552	425,959	438,738	451,900	465,457	479,421	493,803	508,617	523,876	539,592	555,780
Total Operations & Maintenance	**$664,536**	**$686,113**	**$708,396**	**$731,406**	**$755,168**	**$779,707**	**$805,047**	**$831,216**	**$858,241**	**$886,150**	**$914,971**	**$944,736**	**$975,474**	**$1,007,219**	**$1,040,002**	**$1,073,859**	**$1,108,825**	**$1,144,936**
TOTAL DEBT SERVICE	**$115,454**	**$164,915**	**$430,709**	**$434,423**	**$430,279**	**$434,196**	**$338,860**	**$323,661**	**$605,105**	**$886,550**	**$1,231,319**	**$1,301,680**	**$1,372,041**	**$1,463,510**	**$1,509,245**	**$1,649,967**	**$1,678,112**	**$1,706,256**
RATE FUNDED CAPITAL	**$0**	**$50,000**	**$100,000**	**$150,000**	**$200,000**	**$250,000**	**$300,000**	**$350,000**	**$400,000**	**$450,000**	**$500,000**	**$550,000**	**$600,000**	**$650,000**	**$700,000**	**$750,000**	**$800,000**	**$850,000**
TRANSFER TO OTHER FUNDS	**$430,156**	**$440,910**	**$451,933**	**$463,231**	**$474,812**	**$486,682**	**$498,849**	**$511,320**	**$524,103**	**$537,206**	**$550,636**	**$564,402**	**$578,512**	**$592,975**	**$607,799**	**$622,994**	**$638,569**	**$654,533**
TOTAL CHANGE IN RESERVE FUNDS	$11,335	$105,981	($13,892)	$129,443	$281,335	$430,237	$539,567	$579,088	$326,245	$172,851	($27,765)	($1,751)	$35,602	$63,157	$147,895	$69,490	$108,557	$152,605
TOTAL REVENUE REQUIREMENTS	**$1,221,480**	**$1,447,919**	**$1,677,146**	**$1,908,503**	**$2,141,594**	**$2,380,821**	**$2,482,323**	**$2,595,286**	**$2,713,695**	**$2,932,757**	**$3,169,161**	**$3,359,067**	**$3,561,629**	**$3,776,861**	**$4,004,942**	**$4,166,311**	**$4,334,062**	**$4,508,331**
Balance/(Deficiency) of Funds	$0	($218,847)	($440,138)	($663,729)	($888,386)	($1,118,170)	($1,210,076)	($1,309,484)	($1,413,845)	($1,618,795)	($1,841,193)	($2,018,785)	($2,207,782)	($2,408,883)	($2,622,825)	($2,769,857)	($2,923,228)	($3,083,197)
Balance/(Deficiency) as a % of Rate Rev.	**0.0%**	**17.9%**	**35.8%**	**53.7%**	**71.6%**	**89.4%**	**96.1%**	**102.9%**	**110.0%**	**124.7%**	**140.5%**	**152.5%**	**165.1%**	**178.4%**	**192.3%**	**201.0%**	**210.1%**	**219.4%**
Proposed Rate Adjustment	**0.0%**	**17.9%**	**15.2%**	**13.2%**	**11.6%**	**10.4%**	**3.5%**	**3.5%**	**3.5%**	**7.0%**	**7.0%**	**5.0%**	**5.0%**	**5.0%**	**5.0%**	**3.0%**	**3.0%**	**3.0%**
Cumulative Proposed Rate Adjustment	**0.0%**	**17.9%**	**35.8%**	**53.7%**	**71.6%**	**89.4%**	**96.1%**	**102.9%**	**110.0%**	**124.7%**	**140.5%**	**152.5%**	**165.1%**	**178.4%**	**192.3%**	**201.0%**	**210.1%**	**219.4%**
Debt Service Coverage Ratio																		
Before Rate Adjustment	1.10	0.62	0.18	0.12	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
After Proposed Rate Adjustment	1.10	1.95	1.20	1.64	2.12	2.57	3.48	3.87	2.20	1.70	1.38	1.42	1.46	1.49	1.56	1.50	1.54	1.59
Residential Monthly Average Bill																		
After Proposed Rate Adjustment	**$11.15**	**$13.15**	**$15.14**	**$17.14**	**$19.13**	**$21.12**	**$21.86**	**$22.63**	**$23.42**	**$25.06**	**$26.81**	**$28.15**	**$29.56**	**$31.04**	**$32.59**	**$33.57**	**$34.57**	**$35.61**
Monthly $ Change	**$0.00**	**$2.00**	**$2.00**	**$2.00**	**$1.99**	**$1.99**	**$0.74**	**$0.77**	**$0.79**	**$1.64**	**$1.75**	**$1.34**	**$1.41**	**$1.48**	**$1.55**	**$0.98**	**$1.01**	**$1.04**
Commercial Monthly Average Bill																		
After Proposed Rate Adjustment	**$220.85**	**$260.38**	**$299.96**	**$339.55**	**$378.94**	**$418.35**	**$433.00**	**$448.15**	**$463.84**	**$496.30**	**$531.05**	**$557.60**	**$585.48**	**$614.75**	**$645.49**	**$664.85**	**$684.80**	**$705.34**
Monthly $ Change	**$0.00**	**$39.53**	**$39.58**	**$39.59**	**$39.39**	**$39.41**	**$14.64**	**$15.15**	**$15.69**	**$32.47**	**$34.74**	**$26.55**	**$27.88**	**$29.27**	**$30.74**	**$19.36**	**$19.95**	**$20.54**
Ending Fund Balance																		
Operating Reserve	$308,516	$414,497	$400,605	$505,049	$511,383	$516,620	$531,187	$610,276	$661,520	$734,372	$706,606	$704,856	$740,457	$778,615	$801,510	$846,001	$859,557	$882,163
Capital Reserve	724,274	621,774	425,024	633,299	1,144,902	1,856,780	2,718,934	3,606,460	3,013,695	2,296,309	1,235,305	1,030,354	875,794	882,628	989,861	963,829	866,204	853,989

As can be seen in Table 7-4, the timing of the capital improvement plan has a significant impact on the rates of the utility. This is due primarily to the lack of any outside funding sources other than revenue bonds to fund these projects. With near-term revenue adjustments and bond funding to accomplish the capital improvement schedule set forth in Chapter 6, the City's primary impact is in the first three years of this plan. After this initial period, the need for rate adjustments based upon the planning assumptions used in this analysis is reduced to slightly more than an inflationary adjustment. This is due to the building and use of reserve funding in conjunction with the collection of system development fees based upon calculated growth.

Another important item to note in Table 7-4 is the line item "Capital Improvements From Rates". A general rule of thumb for funding utility renewal and replacement projects is to fund a minimum level of annual depreciation expense. The 2009 depreciation expense is $472,000. As shown in the summary table above, the utility gradually increases the funding of capital from rates increasing this level above current annual depreciation expense over the 6-year review period shown above. While the capital improvements funded through rates amount begins to exceed current depreciation, the current depreciation figures should be adjusted as the plant improvements are brought on line.

This type of capital funding also improves debt service coverage requirements that are required by bonding agencies if revenue bond funding is used to pay for the project costs. Most bond covenants require a minimum of 1.25 coverage. This means the utility has the capacity to pay the debt service obligations, plus an additional 25 percent of those obligations, each year. As seen in the Table 7-4, the utility is not currently meeting debt service coverage requirements. However, with the proposed rate adjustments, that situation is quickly remedied.

It is recommended that the City of Evans pursue available grants and low-interest loans to offset the cost of the wastewater treatment plant improvement projects. Following is a detailed discussion of outside sources of funding available for projects of this type.

7.5. Funding Sources

A key implementation consideration is the method of funding and financing the recommended improvements. This section reviews potential outside funding sources that may be considered.

7.5.1. Federal and State Funding Sources

The City has the ability to apply for grant and loan funds available to public entities for wastewater utility system projects. The State of Colorado has revolving funds available for grants and low-interest loans for water and wastewater facility projects which are funded through the U.S. Department of Environmental Protection (EPA) and the Federal Clean Water Act. This is the primary funding source available for the City's capital improvement plan. Table 7-5 provides a summary of the agencies and their contacts for the Colorado Water Pollution Control Revolving Funds. This source rarely provides full funding of a construction project. The City must supplement these funds with matching

funds to meet eligibility criteria and to ensure that implementation of the recommended capital improvement projects can occur.

Table 7-5. Colorado Water Pollution Control Revolving Fund Agencies and Contacts

Agency	Address	Phone	Internet
Colorado Department of Public Health and Environment	Water Quality Control Division Erick Worker 4300 Cherry Creek Drive South Denver, CO 80246-1530	303.692.3594	www.cdphe.state.co.us erick.worker@state.co.us
Colorado Water Resources & Power Development Authority	Colorado Water Resources and Power Development Authority Mike Brod 1580 Logan St, Suite 620 Denver, CO 80203	303.830.1550 Ext. 15	www.cwrpda.com mbrod@cwrpda.com
Department of Local Government Affairs (DOLA)	Division of Local Government Affairs Barry Cress 1313 Sherman St, Room 521 Denver, CO 80203	303.866.2352	www.cwrpda.com barry.cress@state.co.us

A brief description of each available funding source is provided below.

7.5.1.1. Colorado Water Pollution Control Revolving Fund and State Domestic Wastewater Treatment Plant Grant Program

Colorado Senate Bill 50 in 1988 amended Title 37 of Colorado Revised Statutes (CRS) Title 37 of Article 95 established the Water Pollution Control Revolving Fund (WPCRF) as an on-going funding mechanism for water quality projects. This was authorized and completed under the guidelines of the federal Clean Water Act. The purpose of these funds is to improve or benefit water quality in the state. The program is administered through a partnership between the Colorado Department of Public Health and Environment through the Water Quality Control Division (WQCD), the Colorado Department of Local Affairs (DOLA), and the Colorado Water Resources and Power Development Authority (Authority).

The Authority was created by the General Assembly to aide other state agencies in managing the funds for various funding programs for water and wastewater capital improvements. The Authority assists governmental entities such as cities and special districts by issuing revenue bonds and loaning the proceeds to the governmental entity with substantial savings in costs of issuance and interest rates. Eligible projects include wastewater treatment plants, storage reservoirs, water distribution systems, water wells and pumping stations. Eligible costs include design, engineering, costs of issuance, financing reserves, interest during construction, site acquisition, planning, environmental documentations, construction and mitigation costs.

There are two loans types applicable to the City. One is a direct loan, if under $2 million, and the other is a leveraged loan, which is any loan in an amount over $2 million.

Evaluation Criteria:

This funding source is geared to fund projects that meet the following prioritized categories:

1) Projects that improve or benefit public health, or remediate a public health hazard.

2) Projects that enable an entity to achieve permit compliance.

3) Projects that contribute to the prevention of a public health hazard; enable an entity to maintain permit compliance, or enable an entity to address a future possible effluent limit of emerging issue.

4) Nonpoint source (NPS) projects from a NPS management plan.

5) Projects that implement a source water protection plan.

6) Project that seek funding under the American Recovery and Reinvestment Act of 2009 that were not already on the Project Eligibility list of January 2009.

Application time frames:

The two main deadlines for applications during the year are December for the spring bond issue, and June for the fall bond issue. Other interim dates include:

- November 15th for consideration at the January Authority's Board meeting
- December 15th for consideration at the Authority's March Board meeting
- February 15th for consideration at the Authority's April Board meeting
- August 15th for consideration at the Authority's October Board meeting
- October for consideration at the Authority's December Board meeting

It is recommended to submit applications at least two weeks prior to any of the deadlines listed above to allow for WQCD staff and DOLA staff to work with the City if there are any outstanding information needs to process the application. The December 15th deadline is also the deadline for direct loan consideration (loans under $2 million). Additionally, an Engineering Report must be submitted one month prior to the deadline to allow State staff time to review the document and verify any assumptions or questions.

Financing Terms and criteria:

Terms of these loans are 20 years. The Authority's Board sets the interest rates each year. The rates listed here are for FY 2010. Direct loans of $2 million or less will be at an interest rate of 2 percent. (For disadvantaged communities the interest rate is 1 percent. Disadvantaged communities are those whose median household income is 61 percent to

80 percent of the state average household income.) For leveraged loans (>$2 million) the interest rate is currently 2.5 percent.

7.5.1.2. Colorado Water Resources and Power Development Authority Interim Loan Policy

Interim loans can be made to entities for their water and wastewater infrastructure projects for up to two years, or until closing of the next bond issue for leveraged loans. The types of projects funded are the same as those funded by the Authority (as noted above, and other programs related to water, NPS, and reuse projects). The project does not necessarily need to be on the eligibility list, but must be eligible to be on the list. The interim loans are generally only available for projects that are assumed to receive a loan under the WPCRF program but have financial obligations prior to the receipt of bond proceeds.

The interest rate is higher than the previously mentioned loans, at 3.5 percent. If not paid within the two year time frame, the interest rate defaults to the prime rate plus two percent. This program is intended to fund projects until they can receive other loan or financing mechanisms. The time frame for applying is any time outside the funding cycle for the programs listed above.

7.5.1.3. State and Tribal Assistance Grants (STAG Grants)

These funds are generally available through Congressionally earmarked funds through the State's Congressional delegation. Funds come through the Department of Interior Appropriations Bill, within the EPA's budget categorized as "Congressionally Requested Project" STAG grants. There are approximately $400 million in STAG grants appropriated each year. These are grants of up to 55 percent of the project cost, with the rest matched by the entity or other non-federal funding source (revolving funds qualify as match). Funds can also be requested retroactively, for projects already constructed, especially if the financing of that project resulted in utility rates that create a financial hardship.

The best way to begin this process is to call the local Congressional representative's/Senator's office and talk to their chief of staff about the project, its merits, why it is important to your community. This helps you to determine if the Congressional member will be supportive of the project and take the request forward.

Evaluation Criteria:

There are no specific evaluation criteria for this funding source. As with numerous other funding sources, an entity often must request funding at least twice before funding may be received. In general, the first application makes the Congressional delegation aware of the project and issues it is designed to address. Unless the project is addressing a well-known emergency health or safety issue, it is unlikely the project will receive funding on the first attempt. Often, the second or third time the funds are requested, they understand that this is a serious project, and funding may be provided.

It is important to identify the community problems the project will. If there are any energy efficiency improvements that will improve (reduce) future operating costs, use

less energy, or if the project(s) contains renewable energy components or “green solutions” these components can enhance probability of funding success.

Depending upon the timing of the funding request, a NEPA review may be required. Because funds can be applied to projects already constructed (most often if there is a proven financial hardship) NEPA would not be necessary in that case. However, if funding is requested while the project is in design, or before construction has begun, a NEPA review may be required. The need for a NEPA review is determined on a case-by-case basis.

Application time frames:

Typically, funding requests must get to the Congressional delegation by the first week in February of the prior year. For example, to receive funding in fiscal year (FY) 2012, a funding request would need to be presented to the State Congressional Delegation before January 2011.

The request then goes to the Appropriations Committee for the type of request (agricultural, EPA, etc.) in June/July. If it comes out of that Committee process, it moves forward in the Appropriations package for approval by October of that year for the following year’s budget.

Financing Terms and criteria:

Grant funding can range from $500,000 to millions of dollars. The maximum funding level is 55 percent of total project cost. Any ultimate appropriation depends upon other pressing needs within the State, political issues, and the presentation and summary project package information.

7.5.1.4. Community Development Block Grants

This federal program is funded by the U.S. Department of Housing and Urban Development and administered through DOLA. The grant program has a maximum grant amount of $400,000 and is often used for small wastewater capital improvements.

Colorado's Community Development Block Grant (CDBG) Program is a federally-funded competitive grant program designed to help communities with their most critical community development needs. The program was established by Congress in 1974 and is administered nationally by the U.S. Department of Housing and Urban Development (HUD). By agreement, DOLA administers the CDBG Program for local governments who do not receive funding on an “entitlement” basis directly from HUD.

Under federal law, all CDBG projects must principally benefit low and moderate income persons. In public facility projects, this is accomplished by making improvements to public facilities that serve communities or neighborhoods that are mostly low or moderate income families.

Eligible Projects:

A wide variety of community development projects are eligible for grant funding, divided into three basic categories: (1) Economic Development; (2) Housing and Community Revitalization; and (3) Public Facilities.

Eligible Applicants:

Eligible applicants are limited to general purpose local governments: incorporated towns and cities typically under 50,000 population and counties. Local governments can apply to DOLA on behalf of private businesses, private nonprofit corporations such as a local economic development corporation, or special purpose governmental agencies such as a housing authority or a water or sewer district. In all cases the local government assumes ultimate responsibility for administration of the federal funds and compliance with all federal and state requirements. Each local government can apply for one housing project and one public facility project each program year. Applications for economic development projects are accepted continuously as long as funding is available.

Funding Requirements:

CDBG provides grants to local governments up to $400,000. For the public facilities category, local governments must provide a match of at least 25 percent of the CDBG funds requested. The match may be waived in cases of extreme financial hardship and where a serious public health or safety problem exists. In the case of sewer projects, analysis of financial need focuses on a community's projected sewer rates measured against the community's median household income and other economic factors. Projected sewer rates are compared to a "target rate" based on local median household income.

Funds Available:

Over the last several years, approximately $8 million each year has been available for grant awards for all four CDBG grant award categories.

Application Process:

DOLA accepts applications until December 1 of each year. Funding is available the following April. DOLA has a specific application which must be completed. A copy is available at- http://www.dola.state.co.us/dlg/fa/cdbg/docs/application.doc. The DOLA contact is:

Becky Murray, DOLA
1313 Sherman Street, Rm 521
Denver, CO 80203
Becky.murray@state.co.us

7.5.2. Local Sources of Funds for Wastewater Capital Improvements

Local funding of capital improvements rely on private financing through bond sales or cash-based financing through use of system development fees, capital reserves, and rates.

7.5.2.1. Revenue Bonds

Revenue Bonds are long-term municipal bonds guaranteed solely by the dedication of project income or sewer funds (e.g., user fees) rather than by a general tax. Current interest rates on bonds range from 4 to 5 percent.

7.5.2.2. General Obligation Bonds

General Obligation Bonds are long-term municipal bonds that are backed by the full faith and credit of the local government. This means that the City of Evans would pledge to use all of its taxing and other revenue-raising powers to repay bond holders. General obligation bonds require voter approval through a local bond election.

7.5.2.3. Utility Local Improvement Districts (ULID)/Special Assessment Districts

When system improvements are needed in one specific location and benefit only those customers adjacent to the improvement, this funding approach can be used to finance the improvements. A good example would be a collection line installation or replacement of a failing line for a particular service area. Typically, the City would finance the improvements through a loan or bond to be paid back over time (15 – 20 years) by the beneficiaries of the improvement(s) who are assessed a charge on a monthly or annual basis until the financing is paid. Again, this type of financing is used when the system improvement does not have wide spread benefits across the service area.

7.5.2.4. Latecomer's Agreement

This type of financing is similar to a ULID except that a third party, usually a developer, pays for the improvement. The City serves as the intermediary, accepting the payments from new customers connected to the improvement (eg. collection line) and forwards those payments on to the developer, less an administrative charge. The incentive for developers to participate is that, without this type of financing and without the infrastructure in place, they cannot develop. In some cases, however, developers do not fully recover the cost of the improvement.

7.5.2.5. Capital Reserves or Cash Basis

Capital improvements may be funded through cash reserves developed through system development fees and user fees. The obvious advantages of this approach are the avoidance of debt and interest payments.

7.5.2.6. Rate Funded Capital

The City has not traditionally planned for a specific amount of project costs to be borne by current ratepayers through capital improvements funded from rates. A general rule of thumb for funding utility renewal and replacement projects is to fund a minimum level of annual depreciation expense. This "source of revenue" should be planned into the City's financial strategy to ensure that current ratepayers are helping to fund the deterioration on the infrastructure they use while receiving service. This planning philosophy has the added benefit of improving debt service coverage requirements that are required by bonding agencies if revenue bond funding is used to pay for the project costs.

The City currently charges for sewer service in a flat monthly fee. A flat monthly fee is a fairly traditional approach to sewer rate design, however, as utilities attempt to more closely tie the cost of providing service to specific customer groups and the manners in which they impact the system, a number of other rate designs are available. The most common rate design next to a flat rate for sewer service, is to charge for sewer service on

a volumetric basis. This requires, however, knowledge of a customer's contributing flows, which is often difficult, if not impossible, to get for each and every customer of a sewer utility system. For this reason, it is common practice to charge a volume-based rate on an assumed amount of sewer flow. There are a number of ways to estimate sewer flow, most of which are based upon water consumption by the same customer. This, however, requires that consumption information is available for each and every sewer customer.

Recognizing that not all water that enters a connection makes its way to the sewer system as wastewater, adjustments can be made to the consumption figure. One method for making an adjustment is to base the volumetric charge on a specified percent of water consumption. Another alternative is to base the volumetric charge on an average of winter water use. For example, in this calculation, a winter period would be defined and an average consumption would be developed which would then be applied to all months of the year. While this is still a form of flat fee, it allows a utility to provide for distinctions between customer classes (i.e. commercial vs. residential) based upon water usage characteristics. The use of winter water consumption as a surrogate for wastewater flows provides for recognition that during peak periods – generally speaking during the summer – a portion of the domestic water use never makes it to the wastewater system.

As a utility becomes more familiar and comfortable with a more intricate rate design for charging for sewer service, more sophisticated rates can be developed. For example, if a utility has access to data from specific customers that provides those customers' loading factors, charges can be developed based upon contributions to the sewer system as a whole. A cost of service study is an important first step that should be taken prior to instituting any form of more detailed rate design to ensure that whatever rate design is selected is fairly and accurately assigning revenue generation to the various customers of the utility.

7.6. Recommended Financial Plan

Chapter 6 presented a summary of estimated capital costs for a variety of alternative wastewater collection and treatment plant management programs and summarized the costs for the recommended plan. These recommended program elements and estimated capital costs were presented in Table 6-10. Implementation of the recommended plan will require a coordinated effort on the part of the City of Evans to see that the various elements of the plan are implemented within the community and the larger study area.

The recommended capital improvement plan suggests these be implemented in multiple phases. As previously stated, it is recommended the City of Evans pursue grant and low-interest loan funding to reduce the impact of the Phase 1 improvements to rate payers. To meet the long term financial needs of the sewer utility, it is recommended the City raise sewer rates $2/month for the next five years to a final rate of $21.12 by 2018. This will provide the required funding to meet the upcoming Phase 1 improvements as well as the long term operations and maintenance requirements. After this point, rates can be raised on an inflationary path until the Hill-n-Park WWTF improvements are required either from a capacity or regulatory need. At which time, the rates will again need to be raised to accommodate the required improvements.

Appendix A – NPDES Permit

Appendix B – Hydraulic Profile

Appendix C – Detailed Cost Estimates

Appendix D -

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