City of Silverton

FINAL
Wastewater System Facility
Master Plan

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

List of Acronyms and Abbreviations

7dadm	7-day average of daily maximum
ACDP	air contaminant discharge permit
ADWF	average dry weather flow
BFP	belt filter press
BOD	biochemical oxygen demand
CAA	Clean Air Act
CBOD	carbonaceous biochemical oxygen demand
CEC	Compounds of Emerging Concern
CMOM	Capacity Management Operations and Maintenance
CWA	Clean Water Act
DAFT	dissolved air flotation thickener
DEQ	Department of Environmental Quality
DO	dissolved oxygen
EDC	endocrine disrupting compound
EPA	Environmental Protection Agency
EQ	exceptional quality
EQC	Environmental Quality Commission
gpcd	gallons per capita per day
gpm	gallons per minute
IMD	Internal Management Directive
mg	million gallons
mgd	millions of gallons per day
MH	manhole
MLSS	mixed liquor suspended solids
MMDWF	maximum month dry weather flow
MMWWF	maximum month wet weather flow
MWVCOB	Mid-Willamette Valley Council of Governments
NPDES	National Pollutant Discharge Elimination System
PCL	Primary Clarifier
PDAF	peak daily average flow
PE	Primary Effluent
PFRP	Process to Further Reduce Pathogens
PIF	peak instantaneous flow
PSL	Primary Sludge
PSRP	Process to Significantly Reduce Pathogens
PSU	Portland State University
RAS	return activated sludge
RPA	Reasonable Potential Analysis
SC	service connection
SCADA	Supervisory Control and Data Acquisition
SDC	system development charges
SFO	Stipulation and Final Order
SRT	solids retention time
SSE	Sanitary Sewer Evaluation
SSMP	Sanitary Sewer Management Plan
SSO	sanitary sewer overflow
SVI	sludge volume index
TKN	total kjeldahl nitrogen
TMDL	total maximum daily loads
TPAD	temperature-phased anaerobic digestion
TPS	thickened primary sludge
TSS	total suspended solids
TWAS	thickened waste activated sludge
UGB	urban growth boundary
UV	ultraviolet
VAR	vector attraction requirements
VCP	vitreous clay pipe
WAS	waste activated sludge
WLA	waste load allocation
WWTP	Wastewater Treatment Plant

Chapter 1 - Executive Summary

The following provides a summary of the analysis, conclusions and recommendations contained in this master plan. In particular the following components of the report are summarized in this chapter:

- Planning Projections
- Water Quality and Regulatory Issues
- Existing Collection System
- Existing Wastewater Treatment Plant and Discharge Facilities
- Collection System Master Planning
- WWTP Master Planning
- Recommendations

Planning Projections

Population

In order to accurately determine future flows and loads (Table 1-1) for the 2030 design target date, it is necessary to make an educated prediction of the Silverton residential population and the degree to which it will increase over the next 25 years. The following approaches were considered to estimate the 2030 Silverton population.

- Census data and City of Silverton population estimates provided by the Mid-Willamette Valley Council of Governments (MWVCOG) and the Portland State University (PSU) Center for Population Research and Census for the years 2000-2005
- Utilizing the same 2000-2005 census and population estimates as Projection 1, but applying the average net growth of 152 persons per year in place of the average percent growth
- Extrapolation from the City's 2001 Comprehensive Plan
- Recent City data on a high spike in construction activity/permits was used to calculate an increased growth rate since 2001
- Assuming the 2005 construction boom was an anomaly, the permit spike noted in

Projection 5 is assumed to be filled that year with the overall growth rate returning to approximately 2% for future growth

A population projection of 14,000, with high initial growth and slower growth later in the planning period, is recommended for future planning.

Table 1-1: Population Projections

Method	Data Used	Projected Population (2030)
Projection 1	Census/Population Estimates	13,400
Projection 2	Census/Population Estimates	12,000
Projection 3	2001 Comprehensive Plan (Population)	12,000
Projection 4	2001 Comprehensive Plan (Land Use/Zoning)	13,900
Projection 5	City Housing Permit Records, 2001 Comprehensive Plan (Land Use/Zoning)	17,700
Projection 6	City Housing Permit Records, 2001 Comprehensive Plan (Land Use/Zoning)	14,200

Flow and Loadings

This section provides estimates of the future wastewater flows and loads based on calculations from recent plant data (Sept. 2002 – Feb. 2006), as well as flow and loading information for Bruce Pac and Quest International for the year 2005. The methods by which each value was calculated are presented in detail in Chapter 4.

The following flow conditions were calculated for this analysis:

- Average Dry Weather Flow (ADWF)
- Maximum Month Dry Weather Flow (MMDWF)

Maximum Month Wet Weather Flow (MMWWF)

Peak Daily Average Flow (PDAF)

- Peak Instantaneous Flow (PIF)

The following contributing load types were used to determine future loading projections:

- Influent Solids Loading
- Septage Flows and Loads

- Industrial Flows and Loads

A summary of the total projected flows and loadings is provided in Table 1-2 below and the recommended facility plan flow and loading is presented in Table 1-3.

Table 1-2: Projected 2030 Total Flow and Loading

	Projected Flow (MGD)	Current Design Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day)	NH3 (lb/day)
Dry Weather						
ADWF	1.71*	2.5	7,765	5,788	1,313	821
MMDWF	2.65	4.3	9,158	8,525	1,504	940
MWDWF	3.06	N/A				
MDDWF		6.0				
Wet Weather						
AWWF	2.54	4.6				
MMWWF	4.17	6.6	9,158	8,525	1,504	940
MWWWF	6.62	N/A				
PDAF	10.87	10.0				
PIF	15.73	12.0				

* The average dry weather flow was also adjusted by adding 0.2 MGD to account for baseline infiltration in the measured plant effluent (on average, measured plant effluent exceeds influent by approximately 0.2 MGD).

Table 1-3: Recommended Facility Plan Flow and Loading

	Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day	NH3 (lb/day)
Dry Weather					
ADWF	2.5	7,800	5,800	1,300	820
MMDWF	4.3	9,200	8,500	1,500	940
MWDWF					
MDDWF	6.0				
Wet Weather					
AWWF	4.6				
MMWWF	6.6	9,200	8,500	1,500	940
MWWWF					
PDAF	11				
PIF	16				

Water Quality and Regulatory Issues

The City currently operates under a National Pollutant Discharge Elimination System (NPDES) permit that allows discharges to Silver Creek and the Oregon Garden. The primary constituents addressed in the NPDES permit are BOD, TSS, ammonia-nitrogen, and temperature, with the ammonia and temperature limits effective upon expiration of the permit.

Based on the City's current NPDES permit, guidance from DEQ, and potential waste load allocations (WLAs) from the Molalla-Pudding TMDL, potential future water quality requirements are described below.

- **BOD/TSS**: Current mass load limits will establish future treatment requirements.

- **Ammonia-nitrogen**: The current NPDES permit establishes a monthly effluent ammonia limit of 0.88 mg/l effective upon expiration of the permit or completion of the Molalla-Pudding TMDL. This limit would increase to 3.0 mg/l if the Environmental Protection Agency (EPA) accepts Oregon's revised water quality criteria for ammonia.

- **Temperature**: The current NPDES permit includes excess thermal load limits for effluent discharged to Silver Creek. The limits are based on biological conditions required to support endangered salmonids, allowing the City to discharge 5.2 million kcal/day during the summer and 21 million kcal/day during the winter (based on 7-day average of the daily maximum temperature). These limits could be modified by the Molalla-Pudding TMDL, but are used as the basis for analysis in the Facility Plan.

- **Turbidity**: The DEQ is currently in the process of revising the turbidity standard, which could result in numerical turbidity limits in NPDES permits. Based on measured effluent turbidity and available background turbidity measurements from Silver Creek in the vicinity of the City's outfall, the new standard could result in permitted effluent concentrations of approximately 4-5 NTU on a monthly average basis and 7-8 NTU maximum.

- **Toxics**: A Reasonable Potential Analysis (RPA) conducted as part of the last permit renewal cycle indicated that cadmium, copper, cyanide, lead, mercury, selenium, silver, and zinc are all parameters of concern. However, the RPA was based on a very limited data set for both the effluent and receiving waters, and in many cases the metals were detected at or near detection levels. The City will continue to gather data as directed in the DEQ's Internal Management Directive to support a more robust RPA as part of the next permit renewal cycle.

- **Compounds of Emerging Concern (CEC)**: CECs include pesticides, pharmaceuticals and endocrine disrupting compounds (EDCs), and industrial chemicals. These compounds are not commonly monitored in wastewater effluent or natural water bodies, but they may have the potential to cause ecological or human health effects. Significant research efforts are currently under way to build an understanding of the sources, fate, and impacts of CECs. It is unclear whether or how CECs will be regulated at a state or federal level.

Biosolids Management

Biosolids management is governed by the Code of Federal Regulations (40 CFR 503), implemented in Oregon in OAR 340 Division 50. From a biosolids treatment perspective, major impacts of the 503 regulations include pathogen reduction requirements, vector attraction requirements (VAR), limits on metals content, and operation and performance requirements for treatment processes.

The City currently produces a Class B biosolids product, which allows it to be beneficially reused on approved sites at agronomic application rates and according to the practices of a DEQ-approved biosolids management plan.

Effluent Reuse

Water quality requirements for recycled water are defined in the Oregon Reuse Rules (OAR 340 Division 55) adopted in 1990. DEQ classifies reclaimed water in four categories: Level I through Level IV. Level IV treatment requirements are the most stringent.

The DEQ is currently in the process of revising the Division 55 reuse rules, and has established a Water Reuse Task Force to make recommendations to DEQ to reduce regulatory barriers and encourage effluent reuse.

Groundwater Regulations

Any discharge that may impact groundwater must meet Oregon standards for groundwater protection. These standards are outlined in Division 40 of the Oregon Administrative Rules (OAR 340-040-0001 through 340-040-0210). The City’s current operation has been determined by the DEQ to have a low potential for adversely impacting groundwater quality.

Air Quality Regulations

Air pollutant emissions are regulated under the Clean Air Act (CAA), the Clean Air Act Amendments of 1990, and Oregon air contaminant discharge permit (ACDP) and Title V programs. Silverton’s WWTP does not currently have an air quality permit, and future expansion is not anticipated to trigger permitting action during the horizon of this Facility Master Plan.

CMOM

CMOM is a program that was proposed to prevent sanitary sewer overflows (SSOs) and WWTP overloading through proactive management of the collection system. The primary purpose of a CMOM-type program is to require system owners to take a proactive approach to preventing sewer overflows.

Much of the work being completed as part of the Facility Master Plan would help the City comply with a CMOM-type regulation.

Existing Collection System

Silverton's wastewater collection system is a conventional gravity system dating back to 1910. Major additions to the collection system were made in 1923, 1939, 1964, and 1983. The collection system now services approximately 910 acres of the 2570 acres within the UGB. Eight pumping stations convey wastewater to the WWTP.

Typical of comparable systems, Silverton's system includes different types of pipe materials including vitrified clay (VCP), PVC and concrete. Most of the pipe installed prior to 1939 was VCP. In the 1960's concrete pipe was installed in the system. Recent construction has consisted primarily of PVC pipe.

As a result of a Sanitary Sewer Evaluation (SSES) completed in 1978, a major rehabilitation of Silverton's wastewater collection system was undertaken in the early 1980s.

Collection System Condition Assessment

Leak Busters, Inc. carried out an electro-scan study of approximately 6,000 feet of sanitary sewer pipe using the Metrotech Focused Electrode Leak Location system (FELL-41™) to assist with leakage assessment of sanitary sewers in connection with the Wastewater System Facility Master Plan.

The pipes tested were 8-to 18-inch diameter vitreous clay pipe (VCP) sanitary sewers. Access to the sewers was through manholes (MH) with an average separation of 350 feet and depth of 8 feet.

Electro-scan testing showed that most of the pipe sections have defects that are potential leaks; however, analyses of the results show that the number, size, and type of the defects vary considerably between pipe sections.

To prioritize the severity of pipe conditions, each anomaly type was given a corresponding weight. Anomalies determined to be large were given a weight of 5; medium anomalies were given a weight of 3; and small anomalies were given a 1. The scores were then summed to produce a total score.

Five of the segments analyzed were determined to fall in a "high rehab priority" category. Nine segments were determined to fall in a "medium rehab priority" category. The remaining five segments were determined to fall in a "low rehab priority" category.

Details on the locations and condition of these pipe segments are presented in Chapter 6.

Existing Wastewater Treatment Plant and Discharge Facilities

Wastewater is primarily comprised of domestic sewage, with 9.1 percent attributed to industrial sources. The facility consists of headworks, primary clarification, secondary treatment and settling, ultraviolet (UV) disinfection, and post treatment aeration. The following design parameters for the treatment facility are based on a 2015 design year:

• Average Dry Weather Flow:	• 2.5 mgd
• Maximum Month Wet Weather Flow:	• 6.6 mgd
• Peak Hour Capacity:	• 12 mgd
• Design Biochemical Oxygen Demand Loading:	• 7,900 lb/day

Historical Plant Performance

The liquid stream treatment process has performed well since commissioning of the new activated sludge facility. The plant has had two effluent permit violations since startup, but these were determined to be data anomalies and no enforcement actions were taken.

Process data on internal solids handling at the plant is limited. Interviews with plant staff were conducted to determine the plant's performance of solids processing. Primary sludge is approximately 0.25 to 0.5 percent, which is appropriate for Silverton's sludge grit removal process. Gravity thickening of primary sludge results in TPS solids concentration between 3 and 4 percent. Similarly, dissolved air flotation thickening of WAS results in a thickened WAS solids concentration between 3 and 4 percent. The anaerobic digesters achieve a volatile solids destruction efficiency of approximately 60 percent. After anaerobic digestion, the solids concentration is approximately 1.5 to 2 percent.

Unit Process Assessment – Liquid Treatment

The following provides a summary of detailed information provided in Chapter 7 on the liquid treatment unit process assessment for the WWTP.

- Headworks

Presently only a single screen is available; however, a bypass channel is available if the primary screening channel requires service. The influent screening facility is not contained, has no odor control, and is only a few feet from the nearest residential building. Headworks enclosure and odor control should be included in future capital improvement planning.

- Primary Treatment

  The clarifiers were constructed as part of the 1984 upgrade and are in good condition. Based on a typical life cycle for this type of equipment, the mechanisms will require replacement within the next 10 years. The structural concrete appears to be in good condition and does not require replacement within the planning horizon of this facility plan. The primaries are currently not covered and are, therefore, a source of odor. Given the close proximity of residents, installation of covers and foul air treatment should be considered for the future.

- Secondary Treatment

  The secondary treatment system currently operates at 45 percent of its design capacity. The system was designed conservatively; therefore, without a performance history of an activated sludge plant at Silverton WWTP, re-rating the secondary treatment to a higher capacity is possible; however there is no immediate need for re-rating. The aerating basin was designed as a high rate activated sludge system but is currently operated in an extended aeration mode to minimize the WAS yield.

- UV Disinfection

  After some initial startup problems the system has been working properly and without major issues. Due to the equalization basin capacity to store peak hours flow the existing UV disinfection capacity is sufficient for 2030 flows.

- Flow Equalization

  The equalization basin has a total volume of 4 MG. There are two submersible return pumps. Under normal operation (one pump) it takes 2 days to empty the equalization basin.

- Effluent Pump Station

  The effluent pump station consist of two service pumps that pump effluent to the Oregon gardens, one pump is available for equalization basin wash down and one pump is available to pump plant effluent to the Silver Creek outfall during high water levels in the creek. The existing flood level pump has sufficient capacity for 2030 flows. The equalization basin wash down pump does not require redundancy or expansion.

Unit Process Assessment – Solids Treatment

- WAS Thickening

  The DAFT receives WAS from the RAS/WAS pump station at approximately 5,000-8,000 mg/L solids concentration depending on the aeration basin mixed liquor concentration and RAS rate. The DAFT currently utilizes approximately 25 percent of its design capacity and is in very good condition; however, there is currently no backup for WAS thickening. The DAFT is not covered and can be a source for odor. Covering and connecting it to the foul air system is recommended for the future.

- PSL Thickening

  The thickener receives degritted sludge at approximately 0.5 percent solids concentration and is adequately sized for current and future loadings; however, there is currently no backup for primary sludge thickening. The gravity thickener skimmer/sludge collector drive has been recently replaced, and the structure and weir are in adequate condition. The thickener is not covered and can be a major odor source. Adding a cover and connection to foul air treatment is recommended for the future.

- Anaerobic Digestion

  The digesters are overloaded and provide no redundancy. Despite operating beyond capacity, the volatile solids destruction in the digesters average approximately 60 percent. This is very good performance. The existing digesters floating steel covers are in fair shape. Because the digesters always operate at maximum capacity, maintenance and repair is often difficult

- Solids Dewatering, Storage, and Disposal

  The existing plant does not have a solids dewatering process other than the solids lagoons, which do not have adequate storage for seasonal limitations on biosolids land application. The two original lagoons only provide 158 days of storage at average 2005 conditions. An abandoned trickling filter (rocks removed) is used to increase the storage capacity.

Biosolids Management

Silverton faces imminent challenges in the area of biosolids storage and land application. Sludge storage is near capacity, requiring the addition of on-site biosolids storage or modifications to the biosolids treatment scheme.

The biosolids land application program is based on having a willing farmer (or farmers) accept the biosolids; the City does not own the property on which biosolids are applied, nor do they have formal agreements with the land owners, ensuring sites will be available for future land application. Currently, only one customer receives Silverton's biosolids, and application can take place only during an approximate two week period.

Collection System Master Planning

The collection system master planning was performed based on a combination of system hydraulic modeling and analysis of the existing system characteristics.

Conveyance System Modeling

Mike URBAN from DHI was used to simulate the hydraulics of the conveyance system.

Using the calibrated model, the hydraulic capacity of the existing collection system was analyzed based on year 2006, 2030 and ultimate build-out flow conditions. For all future model conditions model runs, the I/I rates and sanitary flows were increased accordingly.

The following conclusions were drawn from the system modeling:

- Upgrade to the Oregon Garden Pump Station will be necessary to accommodate flows from the new Oregon Gardens hotel.
- Capacity improvements are needed at various locations in the system. Some capacity improvements may be combined with improvements identified in the condition assessment.
- Additional capacity issues may arise due to poor pipe condition and/or direct connections to stormwater facilities. These locations should be identified through an ongoing condition assessment program.

Table 1-4 lists pipeline improvement projects that are recommended to address capacity issues identified in the hydraulic modeling analysis.

Table 1-4: Recommended Capacity Related Pipeline Improvements for 2030

Improvement ID	Capacity Issue ID	Improvement Location	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-1	CP-1	Westfield Street	Upsize 6-inch to 8-inch	910	$229,800	2008
IMP-2	n/a	Oregon Gardens Pump Station and force main	Increase pump station firm capacity from 200 gpm to 400 gpm	2 new 400 gpm pumps (1 stand-by)	$18,600	2007 - 2008 (completed before hotel opening)
			Upsize force main from 4-inches to 6-inches	909	$182,500	
IMP-3	CP-3	S. James Street	Upsize 12-inch to 18-inch	576	$214,600	2020-2030
IMP-4	CP-4	Sherman Street	Upsize 12-inch to 18-inch	175	$70,000	2020-2030
IMP-5	CP-5	Adams Street	Upsize 8-inch to 12-inch	850	$283,900	2020-2030

In addition to the pipeline improvements identified in Table 1-4, the City has identified the locations for three new pump stations to serve future growth areas within the Urban Growth Boundary. These pump stations are described in Table 1-5.

Table 1-5: Additional Pump Stations

Improvement ID	Pump Station	Description	Estimated Cost	Project Timing
PMP-1	James Street	New pump station & 8-inch forcemain. Including 18-inch and 12-inch trunk lines on James and Jefferson to connect to existing system. Decommission James & Florida Drive & Second & Jefferson Street Pump Stations	$928,400	2008
PMP-2	Pine Street	New pump station & forcemain	$162,100	2009
PMP-3	Setness Lane	New pump station & 6-inch forcemain and associated 8-inch collector pipes.	$1,038,000	2020

Collection System Assessment Expansion

A limited condition assessment was completed (see Chapter 6) as part of the overall system analysis. Based on the condition assessment, the need for rehabilitation was characterized as high, medium, or low. It is recommended that high priority rehabilitation projects be included in the City's capital improvement plan. Table 1-6 lists high priority pipeline improvement projects.

Table 1-6: Recommended Condition Assessment Related Pipeline Improvements

Improvement ID	Improvement Location	Existing Diameter (in)	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-6	Schlador Street	18	Slipline/pipeburst	572	$70,000	2007
IMP-7	Lone Oaks Street	15	Slipline/pipeburst	355	$40,000	2007
IMP-8	Third St.	15	Slipline/pipeburst	770	$85,000	2008
IMP-9	Meat Packers/High School Area	18	Slipline/pipeburst	385	$46,000	2008

In order to develop a systematic condition assessment approach, a complete analysis was performed on the collection system that utilized all known physical and historical information available. The primary source of information was the City's GIS database with supplementary information provided by the City's 1986 Sanitary Sewer Inventory. The purpose of this effort was to determine a prioritized schedule for expansion of the sanitary sewer condition assessment program.

The following criteria (in order of importance) were used in order to rank the numerous sewer segments for prioritized condition assessment:

1. Pipe Material
    a. Clay
    b. Unknown material
    c. Concrete (excluding Water St.)
    d. Ductile iron
    e. PVC

Within each of the pipe material classes listed above, suggested priority was given to larger diameter pipes over smaller diameter pipes. For example, a 15-inch diameter concrete pipe would have been given suggested priority over a 10-inch diameter concrete pipe. Also, within each diameter classification, high priority was given to long reaches of pipe over short reaches. The overall recommended condition assessment program is summarized in Table 1-7 below.

Table 1-7: Prioritized Program for Future Condition Assessment

Pipe Material	Total Length Required for Assessment (ft)	PW Cost	Year(s) to be Performed
Clay	6,080	$6,080	2007
Unknown	63,530	$51,163	2008-2019
Concrete (excluding Water St.)	24,830	$16,662	2019-2020
Ductile Iron	1,780	$1,177	2020
PVC	52,080	$29,830	2020-2030
Total	**148,300**	**$104,913**	

Wastewater Treatment and Disposal Master Planning

This section describes recommended improvements related to wastewater treatment, effluent disposal, and biosolids reuse. A site plan showing recommended improvements is included as Figure 1-1.

Liquid Stream

Headworks and Primary Treatment

The headworks and primary clarifiers are rated for the current and future design flows, and no improvements are required prior to 2030. The existing mechanism is over 40 years old, but is still working well; however, due to its age, cost for replacement should be anticipated between 2020 and 2030. However, it may remain in service while repair and maintenance efforts are within acceptable levels.

Secondary Treatment

Based on projected BOD and TSS loadings, the secondary treatment capacity will be reached when maximum month dry weather flows reach 2.2 MGD, meaning additional capacity would be required in 2020 and planning should begin in 2015.

Based on the process review conducted as part of this Facility Plan, it appears likely that the treatment process can be optimized to gain additional treatment capacity. In order to optimize the process for improved performance and increased capacity, some process control improvements are necessary. Phase 1 improvements include a series of optimization enhancements and equipment upgrades, ultimately resulting in rerating the facility to a design capacity that will serve the City beyond 2030. The Plan also examined options to provide new secondary treatment capacity, but these improvements will not likely be required during the planning horizon.

Phase 1 – Process control upgrades and optimization

Currently, the secondary treatment system is equipped with basic process control and monitoring equipment. While this level of control is adequate under current flows and loads, once influent flow and loading begin to approach design values, the lack of better control will be limiting to both effluent quality and treatment capacity. The recommended process upgrades (which include the necessary SCADA upgrades) are:

- Online alkalinity control
- Aeration control based on multi-point aeration basin DO measurement and online effluent ammonia analyzer
- Automated SRT with Online MLSS meter

The control upgrades are an important element in the process optimization as they provide the necessary tools for the operator to fine tune the activated sludge process. This would entail;

- adjusting control loops and SCADA programming
- controlled variance of key control parameters such as SRT, target DO, anoxic zone size, effluent ammonia concentration
- expanded water quality parameter and process monitoring program

Once the process and its operation is optimized, under current conditions, the aeration system should be upgraded to provide additional aeration capacity to treat higher influent loads. Finally, when this is completed, full scale stress testing would be conducted ideally in conjunction with secondary process simulation. The results of the stress testing and process simulations can ultimately be used to rerate the secondary treatment facility to its true capacity in order to refine the implementation timeframe for the secondary process expansion.

Phase 2 – Capacity expansion

It is expected that the process control upgrades and process optimization will increase the plant capacity to be sufficient for the 2030 flows and loads. The Facility Plan evaluated expanding the secondary treatment capacity using conventional activated sludge treatment, membrane bioreactor (MBR) or integrated fixed-film/activated sludge (IFAS). Because the latter two alternatives are emerging and undergoing technological advances, and because the improvements are not required until close to or after the end of the planning horizon, the decision regarding future secondary treatment expansion technologies can be deferred until the next facility plan update.

To provide the City with all options in the future, the site master plan reserves room for either a third conventional treatment train or and an MBR system (the hybrid technology would not require additional space).

Effluent Filtration

The implementation of an effluent reuse program may be required to comply with the City's thermal load limit. For planning purposes, this chapter evaluates options for providing effluent filtration to provide 1 MGD of reuse quality water.

The following filtration alternatives were reviewed:

- Continuous Backwash Filters
- Pulsed Bed Filters
- Cloth Media Disk Filters

The cloth media filter is the least expensive. The O&M costs for all three technologies are very similar ($10,000 - $13,000) and would not change the ranking based on cost. These are capital costs, and do not include engineering or administration fees.

While the cloth media filter appears to be the most cost-competitive, the capital costs are comparable enough that the City could refrain from choosing a desired technology and instead allow the various filter vendors to bid head-to-head. With this approach, it is recommended that the City budget around the median capital cost ($400,000) to provide flexibility in selecting the best filtration equipment.

Effluent Pump Station

The flood level pumps and equalization basin washdown pumps have sufficient capacity and do not require improvements. The high service pumps require a third pump to increase the firm capacity to 1200 gpm (1.7 mgd) to provide sufficient redundancy.

Solids Stream

Primary Sludge Pumping

The Primary Sludge Pump Station has numerous operational issues and should be demolished and replaced with a new primary sludge pump station with multiple pumps. The new pump station will be located closer to the primary clarifiers in an underground vault.

Primary Sludge Grit Removal

Classified grit is collected in a haul-off container and periodically taken to a local landfill for disposal. The cyclone was replaced in 1998, but the classifier is corroded and beyond its service life and should be replaced.

Primary Sludge Thickening

The thickener receives degritted sludge at approximately 0.5 percent solids concentration. The gravity thickener skimmer/sludge collector drive has been recently replaced, and the structure and weir are in adequate condition.

Assuming a primary sludge concentration of 0.5 percent, the gravity thickener is adequately sized for current and future loadings; however, there is currently no backup for primary sludge thickening. A second gravity thickener should be constructed in the future to provide redundancy for primary sludge thickening. For the interim, a thickener mechanism should be kept onsite.

WAS Thickening

A single 20-foot-diameter dissolved air flotation thickener (DAFT), constructed in 1998, thickens WAS to approximately 3 to 4 percent, depending on loading and influent solids concentrations.

The DAFT has adequate capacity to handle current and 2030 flows and loads assuming no changes in WAS solids concentration. However, there is currently no backup for WAS thickening. A second backup DAFT is recommended in the future to provide adequate redundancy for WAS thickening.

Recycle of Sidestream Flows

Currently, a single 6-foot-diameter manhole with two submersible pumps returns the following flows to the headworks:

- Gravity thickener overflow
- DAFT underflow
- Drain from grit classifier
- Drains from anaerobic digestion facilities

Plant staff stated that both pumps are running on a relatively continuous basis to match flows into the manhole. Concrete inside the manhole is badly corroded and spalled. It is recommended that a new recycle pump station be constructed as part of the solids handling improvements described below.

Sludge Stabilization

Currently, two 30-foot-diameter anaerobic digesters stabilize thickened primary sludge (TPS) and thickened WAS (TWAS) to Class B biosolids standards. The volatile solids destruction in the digesters averages 60 percent, which is very good performance and is adequate to meet vector attraction reduction requirements:

- Digester Structure

  Due to fire code issues, the existing building could not easily house new mixing, heating, and gas handling equipment without a variance from the local fire marshal or appropriate fire code enforcement official.

- Cover

  The existing digesters have floating steel covers that are in fair shape. Plant staffs indicated the covers travel up and down with no difficulties.

- Mixing

  If the existing digesters continue to be operated, it is recommended the gas mixing system be replaced.

- Foaming Issues

  The existing anaerobic digesters have experienced foaming problems in the past. Some advanced digestion processes such as acid-phase digestion, thermophilic digestion, and temperature-phased anaerobic digestion (TPAD) can mitigate foaming issues.

- Heating

  There is one existing combination boiler and heat exchanger unit for heating both digesters. The unit is sufficient to heat both digesters to 95°F at current loading conditions during winter.

- Recirculation Pumping

  Currently, temporary piping is used for recirculation as the original piping had a long vertical run and the recirculation pumps had air binding problems. This piping should be replaced with a permanent system.

- Gas Handling System

  The existing digester gas flare and gas piping is beyond its service life (installed in 1982) and should be replaced.

<u>Storage</u>

The two solids storage lagoons have a combined capacity of 640,000 gallons. This storage volume would be adequate if Silverton was able to apply biosolids for approximately 5 to 6 months out of the year. It is currently inadequate, however, because biosolids application is limited to a two week period during late summer.

Biosolids Management

The biosolids land application program is based on having a willing farmer (or farmers) to accept the biosolids; the City does not own the property on which biosolids are applied, nor does the City have formal agreements with the land owners ensuring that sites will be available for future land application. The current biosolids management program is not sustainable.

Solids Dewatering

Dewatering will provide the greatest flexibility for on-site solids storage and is recommended due to the currently overloaded and under capacity solids storage lagoons. Several proven solids dewatering technologies are available and are summarized below. Centrifuges have higher maintenance requirements than a screw press and the risk of potential odors is high; however, they are similar in cost and performance. For these reasons, a screw press is recommended for dewatering solids at the Silverton plant.

Solids Stabilization, Storage, and Management Alternatives

The most critical element of the solids handling process that requires improvements is the solids stabilization system. The following three alternatives were evaluated to meet the City's biosolids stabilization, dewatering, and storage needs:

- Alternative 1: Anaerobic Digestion, Dewatering, Cake Storage, Land Application
- Alternative 2: Thickened Sludge Blending, Lime Stabilization, Dewatering and Storage
- Alternative 3: Anaerobic Digestion, Dewatering, Drying

Detailed analyses of each alternative were completed, and the alternatives were evaluated based on life cycle cost (including capital and O&M cost), and non cost factors such as biosolids marketability, ease of O&M, reliability and odor potential. These non-cost factors together determined a "benefit score" for each alternative. Table 1-8 shows a cost benefit analysis.

Table 1-8: Cost Benefit Analysis of the Three Alternatives

	Alternative 1	Alternative 2	Alternative 3
Life cycle cost	$11,497	$8,841	$14,837
Benefit score	11	12	12
Cost benefit ratio	$1,045	$737	$1,236

Based on this analysis, it is recommended that the City convert from its existing Class B digestion and liquid sludge storage program to a process that incorporates screw press dewatering and lime stabilization. The initial analysis was based on construction of an enclosed biosolids storage building, however to reduce initial capital cost, it is recommended that the City initially convert one existing sludge lagoon into an open-air dewatered sludge storage facility. Therefore, the recommended biosolids handling improvements include:

- Conversion of the existing digesters to thickened sludge blend tanks
- Construction of a new building to house new screw press dewatering and lime stabilization equipment
- Conversion of an existing sludge storage lagoon to an open-air dewatered biosolds storage facility

Laboratory and Administrative Facilities

Improvements to the lab and administrative building are required to support the staff functions required for efficient long-term operation and maintenance of the treatment plant. Recommended improvements include:

- Adding a new laboratory space with a dedicated HVAC system
- Remodeling the existing laboratory to provide office space for operations and records storage
- Providing new male and female locker room facilities

It is assumed that the renovated facilities would be approximately 1,000 square feet; however, the City should conduct a Schematic Design effort to determine specific facility needs, adjacencies, and layout.

Effluent Management

The recommended effluent management strategy is driven by the need to meet an excess thermal load limit during the summer season. Recommendations are based on the calculated thermal load limits that will become effective upon expiration of the City's permit, but may be modified through implementation of the Molalla-Pudding TMDL. The City has been actively following the development of the TMDL, and should continue to monitor its progress and potential impacts on the City's program. It is recommended that Silverton initiate activities to facilitate compliance with a waste load allocation similar to the excess thermal load in the current NPDES permit, but refrain from making significant capital investments until the TMDL is completed.

Winter Discharge

The existing year-round limits on thermal load to Silver Creek are based on statewide criteria and not on specific conditions or natural thermal potential in Silver Creek. It is extremely difficult to achieve reductions in winter excess thermal load discharges, since there are no consumptive uses for treated effluent. A prior study by Fishman Environmental suggested that removal of the treatment plant effluent from the stream would not impact the likelihood of salmonid spawning or rearing downstream of the outfall. Therefore, if the final Molalla-Pudding TMDL includes a winter thermal load limit that appears unattainable based on existing data, it is recommended that the City conduct a biological evaluation to determine actual impacts on salmonids and assess whether a variance can be granted.

Summer Discharge

- A number of options were evaluated for compliance with the anticipated summer excess thermal load limits. Recommended near-term activities include the following:
- Budget for installation of a third pump in the effluent pump station to allow increased flow to the Oregon Garden
- Conduct a study to optimize performance of the Oregon Garden Wetland for increased temperature reduction and water quality improvement.
- Update the 1998 thermodynamic model of subsurface discharge on the property adjacent to the wastewater treatment plant to evaluate potential temperature reduction based on current effluent and stream temperatures.
- Initiate discussions with the Silver Falls School District regarding irrigation of school property with reclaimed water.
- Initiate a public outreach program to identify additional potential users of reclaimed water.
- Continue to monitor activities of the Willamette Partnership to identify opportunities to buy or sell temperature credits.

Summary of Project Costs and Implementation Schedule

Table 1-9 summarizes recommended collection system and treatment plant improvement projects, costs, and timing. Five discrete wastewater treatment plant projects were identified, incorporating various elements of the overall treatment improvement recommendations. The projects are described below.

- **Project 1: Phase 1 Biosolids Expansion, Phase 1a Process Optimization, Effluent Pumping.** This project includes the Phase 1 capacity-related biosolids improvements (blend tank, dewatering/lime stabilization facility, odor control, recycle pump station improvements, sludge storage), addition of the third effluent pump, and installation of alkalinity feed control, aeration control, and online ammonia analyzers associated with Phase 1a of the secondary treatment improvements. Ongoing process optimization will begin at the completion of Project 1.

- **Project 2: Phase 2 Biosolids Handling, Lab & Admin Facilities.** This project includes upgrading the primary sludge pump station and replacing the grit classifier, as well as expansion of the lab and administrative facilities.

- **Project 3: Aeration System Upgrade.** This project provides additional blower and aeration capacity to support treating higher loads in the secondary treatment process. This project will be required when maximum month influent flows approach 2.2 mgd, which is anticipated to occur after 2015.

- **Project 4: Secondary Treatment Stress Testing/Rerating.** The secondary treatment system stress testing and rerating will be completed following the aeration system upgrade.

- **Project 5: Effluent Filtration/Subsurface Discharge/Reuse.** This project includes capital improvements required to meet temperature TMDL requirements or support development of an effluent reuse program. The timing and cost of this project will depend on the final thermal load allocation in the Molalla-Pudding TMDL, and/or opportunities to use effluent for beneficial reuse applications.

Table 1-9: Recommended Capital Improvements for Silverton Collection System and Treatment Plant Improvements ($1,000s

	2007	2008	2009	2010	2011	2015	2020-2030	Cost ($1,000s)
COLLECTION SYSTEM IMPROVEMENTS								**$ 1,345**
IMP-1 (Westfield Street Capacity)								$ 230
IMP-2 (Oregon Garden Lift Station Capacity)								$ 201
IMP-3 (S. James Street Capacity)								$ 215
IMP-4 (Sherman Street Capacity)								$ 70
IMP-5 (Adams Street Capacity								$ 284
IMP-6 (Schlador Street Condition)								$ 70
IMP-7 (Lone Oaks Street Condition)								$ 40
IMP-8 (Third Street Condition)								$ 85
IMP-9 (Meat Packers/High School Condition)								$ 46
Condition Assessment Program								$ 105
ADDITIONAL PUMP STATIONS								**$ 2,128**
PMP-1 James Street Pump Station								$ 928
PMP-2 Pine Street Pump Station								$ 162
PMP-3 Setness Lane Pump Station								$ 1,038
WASTEWATER TREATMENT PLANT IMPROVEMENTS								**$ 7,018**
Studies								*$ 85*
Thermodynamic Model Update								$ 35
Wetland Optimization Study								$ 25
Laboratory/Admin Facility Schematic Design								$ 30
Project 1 - Phase 1 Biosolids Expansion; Phase 1a Process Optimization; Effluent Pumping								***$ 5,507***
Solids/Effluent Pumping Expansion								$ 5,232
Predesign								
Design								
Construction								
Phase 1a Process Optimization								$ 275
Ongoing Process Optimization								
Project 2 - Phase 2 Biosolids Handling/Lab & Admin								***$ 1,023***
Design								
Construction								
Project 3 - Aeration System Upgrade								***$ 325***
Project 4 - Secondary Treatment Stress Testing/Rerating								***$ 163***
Project 5 - Effluent Filtration/Subsurface Discharge/Other Reuse								

Chapter 2 - Introduction

Background

Over the past 10 years, the City of Silverton has implemented many improvements to provide quality service to ratepayers and protect the sensitive natural environment that contributes to the area's scenic beauty. The City planned for and built a state-of-the-art treatment plant that supports beneficial reuse of effluent at the Oregon Garden site.

Now, almost a decade after these improvements, the City faces new drivers. These drivers include:

- An expanding population in this scenic community which maintains a quaint rural character while being close to employment centers in Salem and Portland
- New regulatory considerations, including a thermal load limit and waste load allocations included in the pending Molalla-Pudding Total Maximum Daily Load (TMDL)
- I/I contributions to the wastewater collection system, reducing available capacity for growth
- Limited capacity for biosolids treatment and storage, and limited options for biosolids final disposal

This Wastewater Facility Master Plan addresses these drivers and balances short- and long-term needs to effectively meet treatment requirements and support future growth while minimizing the impact on ratepayers.

Planning Period

The planning horizon for this facility master plan is the year 2030, which provides a 25-year planning period.

Goals

The primary objective of this report is to provide the City of Silverton with an updated wastewater facility master plan that will identify capital needs through FY 2030, given likely population growth and regulatory changes.

The second major objective of this master plan is to provide the City with a detailed preliminary design report for its wastewater treatment/bio-solids handling system will meet its expected needs for approximately the next 20 years.

Other goals as stated by City employees and TAC members are the following:

- The outcome of the project should be positive for the City in that the time and money invested in the project were well-spent
- The recommended projects should be justifiable to taxpayers
- Project should achieve the best end result for the City
- Recommendations should consider financial impacts on ratepayers
- The Facility Master Plan should include a comprehensive look at the wastewater utility, including operations and long-term needs.
- The Master Plan should include options for Council to consider
- The Master Plan should clearly explain regulatory drivers and other circumstances over which the City has no control
- The Master Plan should contain good growth projections so that the City can determine that growth is sustainable
- The Master Plan should clearly identify drivers to demonstrate to the public why any recommended plant expansions are being made.

Chapter 3 - Planning Area Description[1]

Planning Area

Silverton is located in Marion County, approximately 14 miles east of Salem on the western slope of the Cascade Mountains and the eastern edge of the Willamette Valley (see Figure 3-1)

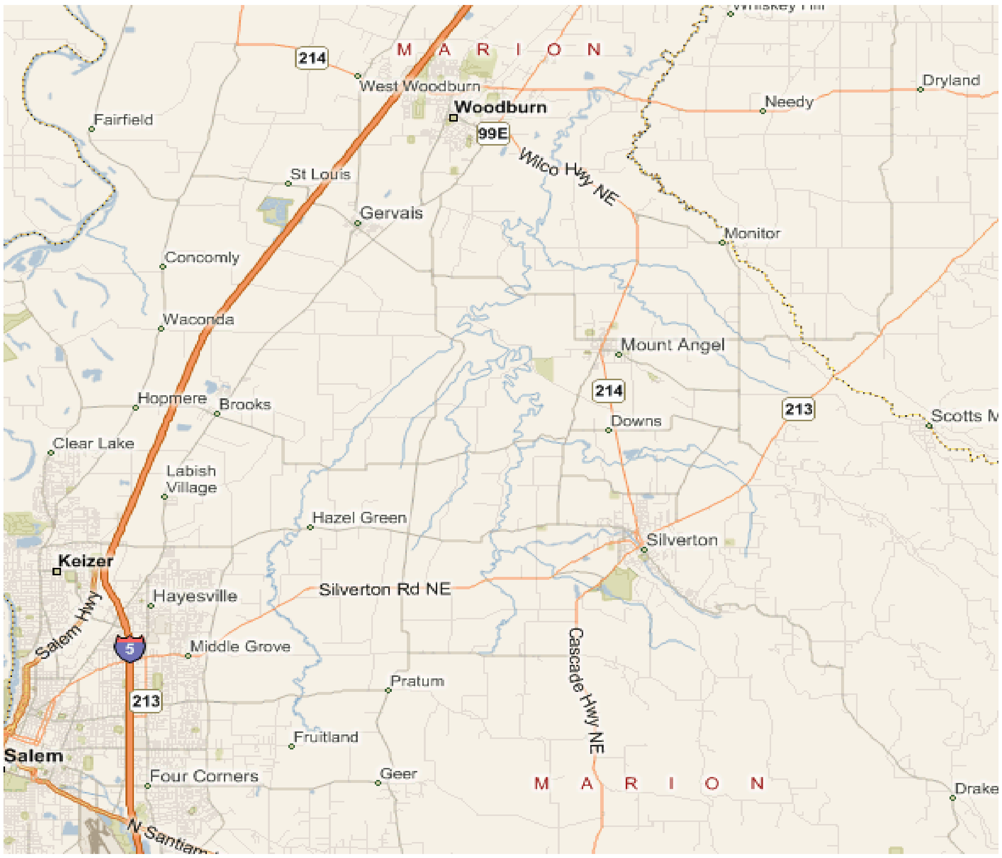


Figure 3-1: Silverton Vicinity Map

The Planning Area for this Facilities Plan is defined as the area that may impact, or be impacted by, modifications to the wastewater facilities. As part of the City's ongoing planning efforts, an Urban Growth Boundary (UGB) has been designated and adopted by the City Council. The UGB and its associated Comprehensive Land Use plan were adopted in 2001. For purposes of this Facilities Plan, the planning area is comprised of all areas within the UGB and city limits. Chapter 7 shows the planning area boundary.

[1] Information in this chapter was taken largely from the 1995 City of Silverton Oregon Sewerage System Facilities Plan Final Report by HDR Engineering, Inc.

Physical Environment

Topography

Silverton is located on the level alluvial plain of Silver Creek between high ridges to the east and west. The general topography of the area slopes downward to the northwest. The elevation of the City is approximately 230 to 250 feet, whereas hills immediately to the southeast rise to nearly 900 feet. North of the city, the topography opens out from the ridges to a relatively flat area.

Climate

Silverton's weather is characterized by wet, mild winters and warm, dry summers. The mean winter temperature is 41 ^{o}F and the mean summer temperature is 65 ^{o}F. Historical temperatures based on City records are shown in Table 3-1 below.

Table 3-1: Seasonal Temperatures1

	Dec–Feb	Mar–May	Jun–Aug	Sept–Nov
Average High (F)	47	59	76	63
Average Low (F)	33	40	52	43
Mean (F)	41	50	65	53

1. Source: www.silverton.or.us

Average annual precipitation is just under 50 inches with most of the precipitation occurring as rain from October to May. Prevailing winds are from the southwest in the winter and the north in the summer. Table 3-2 shows average monthly precipitation for the area as measured at the National Weather Service Silverton Station located at the City's water treatment plant.

Table 3-2: Silverton's Average Monthly Precipitation1

Month	Rainfall, inches
January	6.9
February	5.0
March	4.9
April	3.6
May	2.8
June	1.9
July	0.7
August	0.9
September	1.8
October	3.7
November	7.0
December	7.9
TOTAL	47.2

1. Oregon Climate Service, Station No. 357823, 1962-2006

Air Quality

Silverton is located on the eastern edge of the Willamette Valley air shed. Natural ventilation is restricted by the Cascade and Coast mountain ranges and is limited during periods of atmospheric stagnation in the late summer and early fall. No air quality monitoring has been performed in the City. There are no major pollution sources within the UGB, although severe short-term pollution events occur during the summer and early fall from smoke associated with agricultural field burning.

Geology and Soils

Silverton is located on relatively level alluvial deposits of the Sifton-Salem Association that occur on either side of Silver Creek. The level lands to the north and west of the City are comprised of Willamette Silts which are derived from the Columbia River Basalt. To the east and west of the City, the soils are comprised of weathered Columbia River Basalt and are relatively impermeable.

Nearly all soils in the area are classified as Class I-IV soils as defined by the U.S. Soil Conservation Service (SCS). Classes I-IV soils are those suitable for agricultural use. Table 3-3 lists the SCS soil descriptions of the primary soils predominantly located in the Silverton area, and describes soil suitability for irrigation or rapid infiltration of wastewater on a general basis. Suitability estimates are made based on dominant soil conditions, but site-specific investigation would be required to confirm suitability or limitations with respect to irrigation or rapid infiltration.

Table 3-3: Soils in the Silverton Area[1, 2]

Map Symbol	Description	Suitability for Effluent Irrigation	Suitability for Rapid Infiltration
AbA	Abiqua silty clay loam, 0 to 3 percent slope	Somewhat limited	Very limited
Am	Amity silty loam	Very limited	Very limited
Ca	Camas gravely sandy loam	Very limited	Very limited
Ck	Clackamas gravely loam	Very limited	Very limited
Cm	Cloquato silt loam	Somewhat limited	Very limited
Co	Concord silt loam	Very limited	Very limited
Da	Dayton silt loam	Very limited	Very limited
MaA	McAlpin silty clay loam, 0 to 3 percent slope	Somewhat limited	Very limited
Mb	McBee silty clay loam	Somewhat limited	Very limited
NeB	Nekia silty clay loam, 2 to 7 percent slope	Somewhat limited	Very limited
NeC	Nekia silty clay loam, 7 to 12 percent slope	Very limited	Very limited
NeD	Nekia silty clay loam, 12 to 20 percent slope	Not available	Not available
NeE	Nekia silty clay loam, 20 to 30 percent slope	Not available	Not available
NeF	Nekia silty clay loam, 30 to 50 percent slope	Very limited	Very limited
NsE	Nekia very stony silty clay loam, 2 to 30 percent slope	Not available	Not available
NsF	Nekia very stony clay loam, 30 to 50 percent slope	Very limited	Very limited
Nu	Newberg fine sandy loam	Somewhat limited	Somewhat limited
Nw	Newberg silt loam	Somewhat limited	Somewhat limited
Sa	Salem gravelly silt loam	Very limited	Very limited
SIB	Salkum silty clay loam, basin, 0 to 6 percent slope	Not available	Not available
SuC	Silverton silt loam, 2 to 6 percent slope	Very limited	Very limited
SvB	Stayton silt loam, 0 to 7 percent slope	Not available	Not available
Te	Terrace escarpments	Very limited	Very limited
Wc	Wapato silty clay loam	Very limited	Very limited
WIA	Willamette silt loam, 0 to 3 percent slope	Not available	Not available
WIC	Willamette silt loam, 3 to 12 percent slope	Not available	Not available
WtE	Witzel very stony silt loam, 3 to 40 percent slope	Not available	Not available
WuA	Woodburn silt loam, 0 to 3 percent slope	Very limited	Very limited
WuC	Woodburn silt loam, 3 to 12 percent slope	Very limited	Very limited

1. From US Department of Agriculture, Natural Resources Conservation Service Selective Soil Interpretations for Marion County, Oregon, 2006; and City of Silverton Sewerage System Facilities Plan, 1995.
2. Includes soils comprising 1% or more of the Silverton area.

Earthquakes

Northwestern Oregon is subject to earthquake activity from three sources: crustal earthquakes, intraplate earthquakes and great subduction earthquakes. The Scotts Mills earthquake of March, 1993 (magnitude 5.6) is an example of a crustal earthquake, which is the mildest of all three types. The Scotts Mills earthquake was suspected to be caused by movement along the Mt. Angel fault, located about three miles northeast of Silverton. Geologists indicate that similar earthquakes with magnitudes up to 6.5 can occur at any time. They further warn that because of the location of the region atop the Cascadia subduction zone, even larger earthquakes are possible. The Oregon Department of Geology and Mineral Industries has developed earthquake hazard maps for the region. Figure 3-2 shows the relative earthquake hazard for Silverton and the surrounding areas based on the combined effects of ground-shaking amplification, liquefaction, landsliding. Silverton is generally in a low hazard area, with low to medium and medium to high risk in limited areas of greater slopes due to hazards associated with earthquake-induced landslides. Any construction (including wastewater treatment plant improvements) should take into account the potential earthquake hazard.

Water Resources

The major water features of the area are Webb Lake, Silver Creek, Brush Creek, and Abiqua Creek. Silver and Abiqua Creeks are tributary to the Pudding River, which flows northward to the Molalla River, which in turn discharges into the Willamette River at river mile 36. Brush Creek is tributary to Silver Creek, flowing out of Pettit Reservoir.

Silver Creek is the receiving water for effluent from the City's wastewater treatment plant. Flow in Silver Creek varies throughout the year, with low flow in the dry summer and fall months, and higher flow in the winter and early spring when rains and snowmelt contribute to increased flows.

Both Silver Creek and the Pudding River experience violations of water quality, and DEQ is in the process of developing a Total Maximum Daily Load (TMDL) to address water quality violations in the Pudding River basin. Water quality issues associated with Silver Creek are discussed in Chapter 5. Groundwater availability varies throughout the region and depends on local geology. Generally, wells in the Columbia River basalt to the east and west of the City have very low yields, while wells in the alluvium to the north of town have typical yields of 100-200 gpm.

Flood Plain

The flood plain consists of the floodway and the flood fringe as designated by the Corps of Engineers. Within the floodway, structures could potentially restrict floodwaters and cause greater flooding upstream. Consequently, building in these areas is prohibited. The flood fringe is the area between the floodway and the 100-year flood plain. Building construction is allowable within the flood plain, but Silverton Municipal Code requires that finished floor elevations be at least three (3) feet above the 100-year flood elevation; and the floodway depth and breadth cannot be adversely affected.

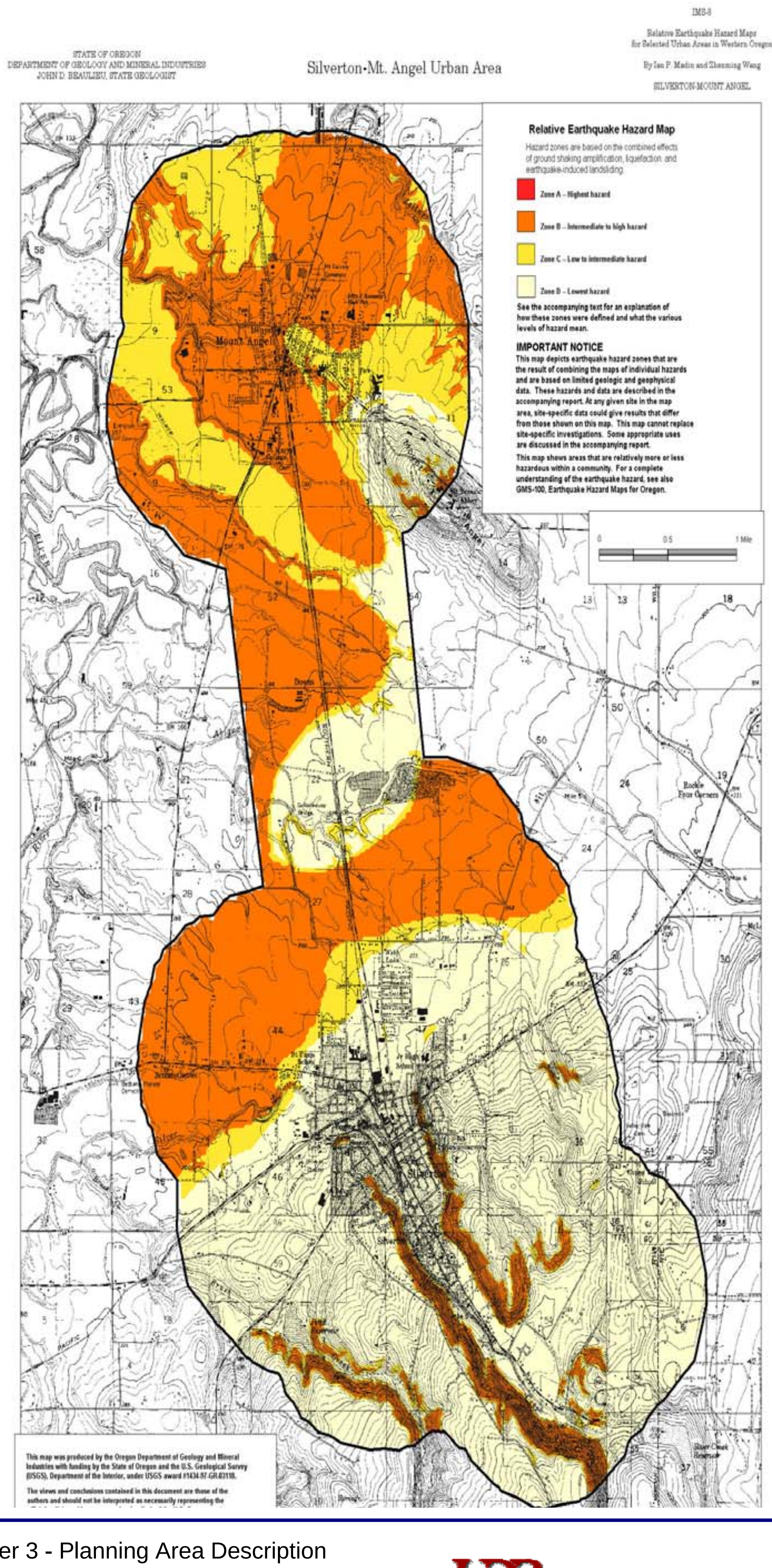


Figure 3-2: Earthquake Hazard Map (Source: State of Oregon Department of Geology and Mineral Industries)

Plants and Animals

Fisheries and Aquatic Life

The Oregon Department of Fish and Wildlife inventoried Silver Creek in July 1993. The Department's conclusion was that the area near the treatment plant was a transition zone and did not support a significant number of game fish year-round. They further concluded that the lower two-mile reach of Silver Creek is a migratory route for winter steelhead and cutthroat trout. Consequently, the stream is classified as "salmonid" with respect to DEQ's established water quality standards, and is protected for salmon and trout rearing and migration.

Sensitive, Threatened and Endangered Species

A complete review of sensitive, threatened, and endangered species in the area was conducted as part of the 1995 Facilities Plan. Appendix C of the 1995 Plan identified federally-listed and proposed endangered and threatened species and candidate species that may occur in the area of the Silverton Wastewater Treatment Plant, and included a list of sensitive species that occur in western Oregon.

Cultural Environment

Land Use and Employment

A land use inventory was performed by the City of Silverton as part of the 2001 Comprehensive Plan. Land use in the area of the treatment plant is predominantly single-family residential, with multi-family residential located northeast of the plant. Currently, the development in the vicinity of the plant is relatively low density.

Based on information provided on the City's website (www.silverton.or.us), Silverton's major employers include the following:

- Silver Fall School District
- Silverton Hospital
- Champion Homes
- BrucePac
- Mallorie's Dairy

Commercial Development

Commercial development is concentrated in the central area of the city where the state highways, the main arterial routes through town, and the railroad line converge. There is also an area zoned for commercial development along Highway 214 at the south end of town.

Industrial Development

Silverton currently has two major industrial contributors: BrucePac and Qwest. There is a 16-lot industrial park in the northeast part of the city, but wastewater production is small and of similar quality to residential wastewater. The City is in the process of identifying potential sites for future industrial development. Wastewater flow and loading contributions from industries in the City is described in Chapter 4.

Transportation

Silverton is served by State Route 213 from the west and east, and State Highway 214 from the north and south. The Willamette Valley Railroad, Inc. line serves the city from the west and north. A small airport located northwest of the city was privately owned and operated from the 1940s to the mid-1980s. It was Oregon's first airport beginning operation in 1916. It is currently inactive.

Historic and Archaeological Sites

It is generally known that the Silverton area was inhabited by a band of the Kalapuyan tribe before white settlement. As part of the 1995 Facilities Plan, the State Historic Preservation Office reported no record of any prehistoric sites along Silver Creek. Further, a cultural resources survey was performed for the 1978 Facilities Plan for the area adjacent to the treatment plant on the west. No archaeological or historic resources were identified.

The following buildings or districts are listed on the National Register of Historic Places:

- Calvary Lutheran Church and Parsonage
- Gallon House Bridge
- Gordon House
- McCallister-Gash House
- George McCorkle House
- Miller Cemetery Church
- Silverton Commercial Historic District
- Victor Point School

Chapter 4 - Planning Projections

Establishing future flow and loading projections is a critical element in determining required investments in the City's wastewater infrastructure. This chapter examines historical and projected population, wastewater flow, and wastewater influent characteristics, and determines recommended projections to use as the basis of planning future facilities.

Population

In order to accurately determine future flows and loads for the 2030 design target date, it is necessary to make an estimate of the Silverton residential population and the degree to which it will increase over the next 25 years. An accurate prediction provides a reasonable basis for facility sizing and verifying the City's capability to serve the future population. Six approaches were taken to estimate the 2030 Silverton residential population:

- Projection 1:

  Based on census data and City of Silverton population estimates provided by the Mid-Willamette Valley Council of Governments (MWVCOG) and the Portland State University (PSU) Center for Population Research and Census for the years 2000-2005, an average percent growth rate of 2.0 was calculated and used to project a 2030 population of 13,400.

- Projection 2:

  Utilizing the same 2000-2005 census and population estimates as Projection 1, but applying the average net growth of 152 persons per year in place of the average percent growth, a projected 2030 population of 12,000 was estimated.

- Projection 3:

  The City's 2001 Comprehensive Plan utilized a growth rate of 1.9 percent that was used for projections from 2001-2020. The Comp Plan estimated a 2020 population of 9,965. Further extrapolating this analysis yields an estimated 2030 population of 12,000.

- Projection 4:

  The City's 2001 Comprehensive Plan provided a projection of the residential housing requirements for 2020. Extrapolating this rate of increase to 2030 and utilizing the density approximations and zoning designations provided in the Plan (Figure 4-1), a net population increase was calculated and added to the mid-year

2000 census data, yielding a 2030 population of 13,900. Table 4-1 provides a summary of the housing and density approximations utilized for Projection 4.

- Projection 5:

  Recent City data indicates a high spike in construction activity/permits for the year 2005 (235 single family dwelling permits versus an average of 40-50 permits for the years 1999-2004). This spike was used to calculate an increased growth rate since 2001 (2.9%) and projected a population of 17,700 in 2030.

- Projection 6:

  Assuming the 2005 construction boom was an anomaly, the permit spike noted in Projection 5 is assumed to be filled that year with the overall growth rate returning to approximately 2% for future growth. This estimate yields an approximate population of 14,200.

Table 4-1: Population Projection 4 (Land Use and Density)

Residential Zone	Additional Units (2030)	Density (people/unit)	Population Increase
Single Family	1346	2.7	3,634
Multifamily	894	2.7	2,414
Manufactured Homes	126	2.7	340
Total			**6,388**

Source: Based on City of Silverton 2001 Comprehensive Plan.

Table 4-2 and Figure 4-2 provide a summary of Projections 1-6. It should be noted that an additional projection that assumed the very high 2005-2006 growth rate (7.7%) was also attempted, but the projected population was unreasonable high (50,000+); thus, this projection was not included in the summarized estimates. Projections 1-6 were reasonably close and varied from 12,000 to 17,700, utilizing both population trends as well as future housing needs as predictive indicators.

Projections 5 and 6 predict higher populations based on the 2005-2006 housing/construction permit increase, and with the difficulty of predicting whether this rate of growth is likely to continue, a blended approach would seem prudent. The blended approach assumes higher growth rates in the early years of the projection with a reduced rate later. This approach would account for high growth without over reliance on the 2005 housing/construction permit data. Thus, a population projection of 14,000, with high initial growth and slower growth later in the planning period, is recommended for future planning.

Table 4-2: Population Projections

Method	Data Used	Projected Population (2030)
Projection 1*	Census/Population Estimates	13,400
Projection 2	Census/Population Estimates	12,000
Projection 3	2001 Comprehensive Plan (Population)	12,000
Projection 4	2001 Comprehensive Plan (Land Use/Zoning)	13,900
Projection 5	City Housing Permit Records, 2001 Comprehensive Plan (Land Use/Zoning)	17,700
Projection 6	City Housing Permit Records, 2001 Comprehensive Plan (Land Use/Zoning)	14,200

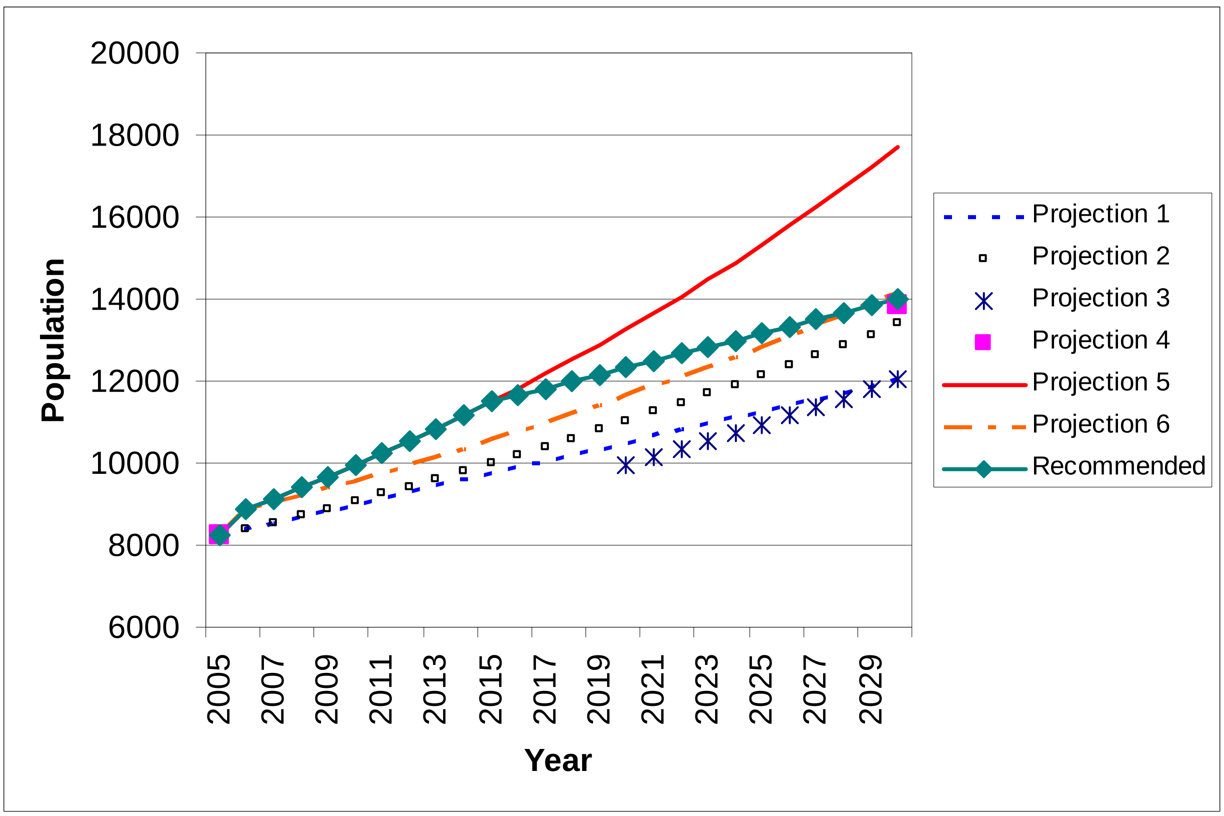


Figure 4-1: Summary of Flow Projections

Flow and Loadings

This section provides estimates of the future wastewater flows and loads based on calculations from recent plant data (Sept. 2002 – Feb. 2006), as well as flow and loading information for Bruce Pac and Quest International for the year 2005. Though the City's primary wastewater source is residential, Bruce Pac and Quest represent the two most significant industrial contributors. Future flow and load predictions will be based on a decoupled estimation of the industrial and residential portions of the flow, with the former being based on approximations of future industrial land use, and the latter relying on the population predictions provided in the previous section.

The only exception to this will be the maximum month loading estimates, which will be based on total influent TSS and BOD. Accurate data was not available to provide an additional separation of residential and commercial applications. Commercial applications were considered to be a part of the residential portion of the flow prediction.

Wastewater Flow Baseline Conditions

The 2003-2005 (calendar year) plant data was utilized to provide approximations of typical baseline flow parameters and current average TSS and CBOD plant loadings. These values were utilized in conjunction with population and land use data to determine the eventual future flows and loads. A summary of the baseline flow parameters is provided in Table 4-3.

Table 4-3: Baseline Flow Parameters (2003-2005)

Year	ADWF (MGD)	MMDWF (MGD)	MMWWF (MGD)	PDAF (MGD)	PIF (MGD)
2003	0.69	1.48	2.36	6.89	--
2004	0.76	0.88	2.57	4.61	--
2005	0.91	1.49	2.48	8.42	--
Average	0.79	1.28	2.47	6.64	
Oregon DEQ (Method 2)	--	1.49	2.92	8.89	13.6

Average Dry Weather Flow (ADWF)

The average dry weather flow was calculated for each year (2003-2005) based on the arithmetic mean of the flows from May to October. This value provides the basis for establishing per capita flows, and also for calculating peaking factors as the ratio between design conditions flows (maximum month and peak daily) and the ADWF.

The ADWF ranged from 0.69 to 0.91 MGD, with 2005 being the highest year. In order to convert ADWF to a per-capita flow, the average dry weather flow values were adjusted to average dry weather residential/commercial flows by subtracting the average industrial flow contribution. Average per-capita flows were then calculated by dividing the residential/commercial flow by the historical population to generate gallons per capita per day (gpcd). These results are shown in Table 4-4 below. The average per capita flow of 89 gpcd correlates well with the value of 90 gpcd used in the *City of Silverton System Development Charge Study for the Transportation, Water & Sewer Services* (FCS Group, August 2005).

Table 4-4: Average Dry Weather and Per Capita Flow

Year	ADWF (MGD)	Industrial Flow (MGD)	Residential ADWF (MGD)	Per Capita Flow (gpcd)
2001				106*
2002				80*
2003	0.69	0.096	0.59	74
2004	0.76	0.096	0.66	82
2005	0.91	0.096	0.81	99
Average				**89**

* 2001 and 2002 values based on Annual I&I Monitoring Report (2004 Calendar Year Activities)

Maximum Month Dry Weather Flow (MMDWF)

The maximum month dry weather flow was calculated (Method 1) as the maximum value in a 30-day running average between May 1st and October 31st. The MMDWF ranged from 0.88 to 1.49 MGD, with 2005 being the highest year. Peaking factors (MMDWF/ADWF) ranged from 1.15 to 1.95, with 2005 providing the largest ratio.

A second method (Method 2) for calculating the MMDWF was also employed, utilizing a statistical correlation between plant flow and precipitation data per the recommendations of the Oregon Department of Environmental Quality (DEQ). A plot of cumulative rainfall versus monthly average flow was developed (Figure 4-2) for the months of January to May from 2003 to 2005.

Based on climatology charts recommended by the DEQ (DEQ, 2005a) the 10-year cumulative rainfall (90% probability) for May, which is determinative as the wettest dry weather month, is 4.42 inches. This yields an MMDWF of 1.49 MGD based on Figure 4-2, which correlates with Method 1 and supports the associated peaking factors.

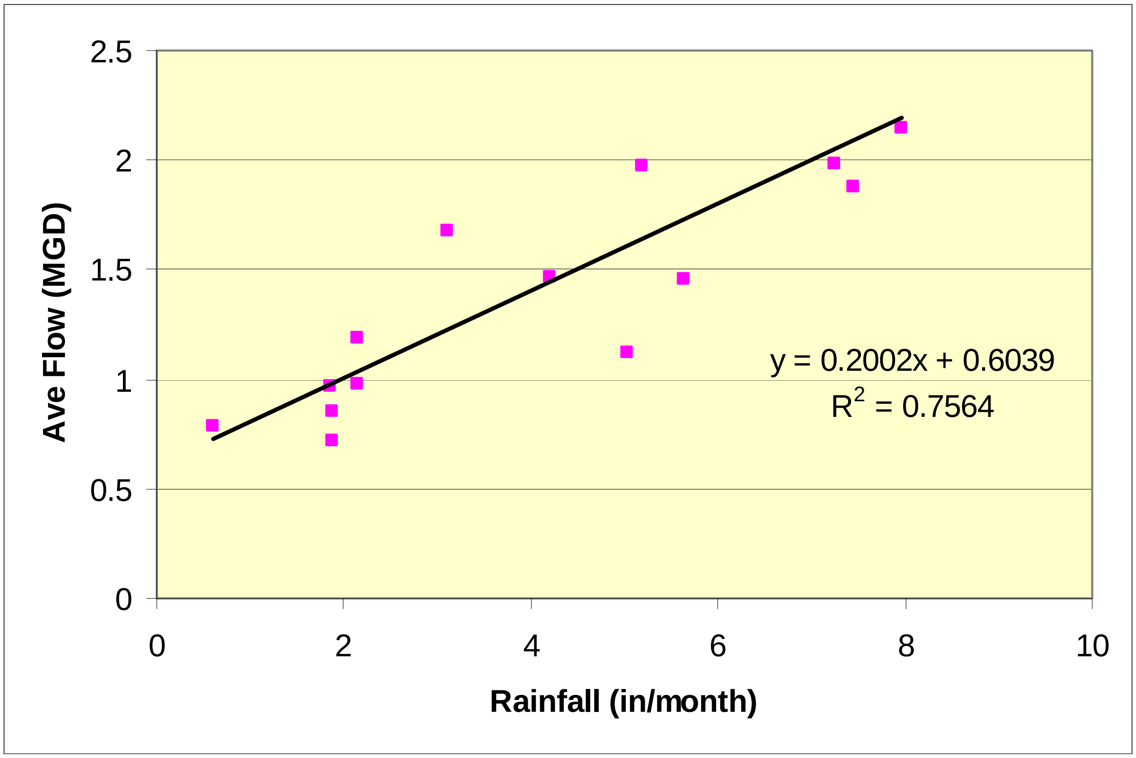


Figure 4-2: Average Monthly Plant Flow vs. Monthly Rainfall for Jan-May, 2003-2005.

Maximum Month Wet Weather Flow (MMWWF)

The maximum month wet weather flow was calculated (Method 1) in the same manner (30-day running average) as the maximum month dry weather flow, utilizing the data from the months of the preceding November to April of the year in question (i.e., the 2003 MMWWF is based on the data from November 2002 to April 2003). The MMWWF ranged from 2.36 to 2.57 MGD, with 2004 being the highest year. Peaking factors (MMWWF/ADWF) ranged from 2.73 to 3.42, with 2003 providing the largest ratio.

The second calculation method (Method 2) for the MMWWF utilizes the same Figure 4-2 as was used for the MMDWF. However, the normative climate data is the five-year January rainfall (80% probability), which is estimated at 11.56 inches and yields a MMWWF of 2.92 MGD.

This is slightly higher than the values predicted for Method 1, but reasonably close considering the added variability of the statistical approach and rainfall data.

Peak Daily Average Flow (PDAF)

The peak daily average flow (Method 1) was taken as the maximum value for each calendar year and ranged from 4.61 to 8.42, with 2005 having the highest daily value. Peaking factors (PDAF/ADWF) ranged from 6.07 to 10.0, with 2003 providing the largest ratio.

Method 2 for calculating the PDAF requires that the plant flow be correlated to the 5-year, 24-hour storm event. Figure 4-3 displays the daily plant flow data (for the Jan through May months, 2003-2005). Weather Bureau records (NOAA Atlas 2, Volume X) estimate the five-year storm event at 3 inches per day. This corresponds to a PDAF of 8.89 MGD, which correlates with the higher range predicted by Method 1 and supports using a peaking factor of approximately 10 for prediction of future flows and loads.

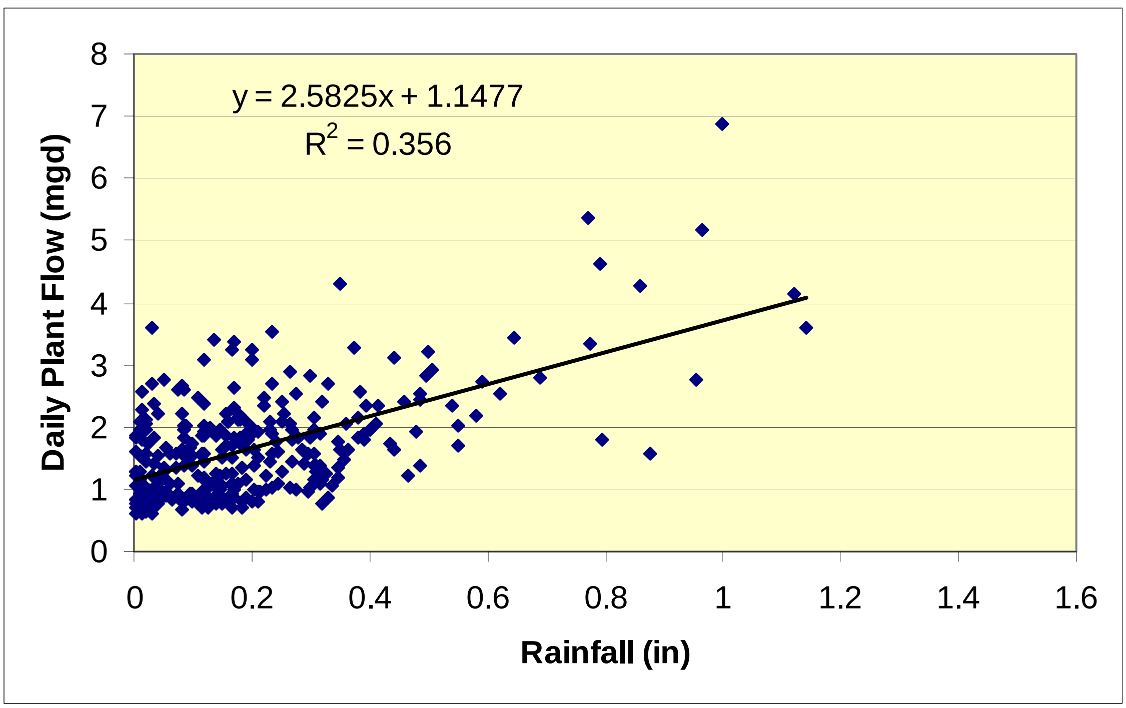


Figure 4-3: Daily Plant Flow vs. Daily Rainfall for Jan-May, 2003-2005.

Peak Instantaneous Flow (PIF)

The peak instantaneous flow, which represents the peak flow resulting from a five-year storm during high groundwater periods, was calculated using a probability graph per DEQ recommendations (Method 2). The method assumes a particular probability of exceedence for the annual average flow (50%), the MMWWF (8.3%) and the PDAF (0.27%). Graphing these values (Figure 4-4) allows for the determination of the PIF at 0.011% probability of exceedence per a logarithmic fit of the data. The value predicted by Figure 4-4 is 13.6 MGD.

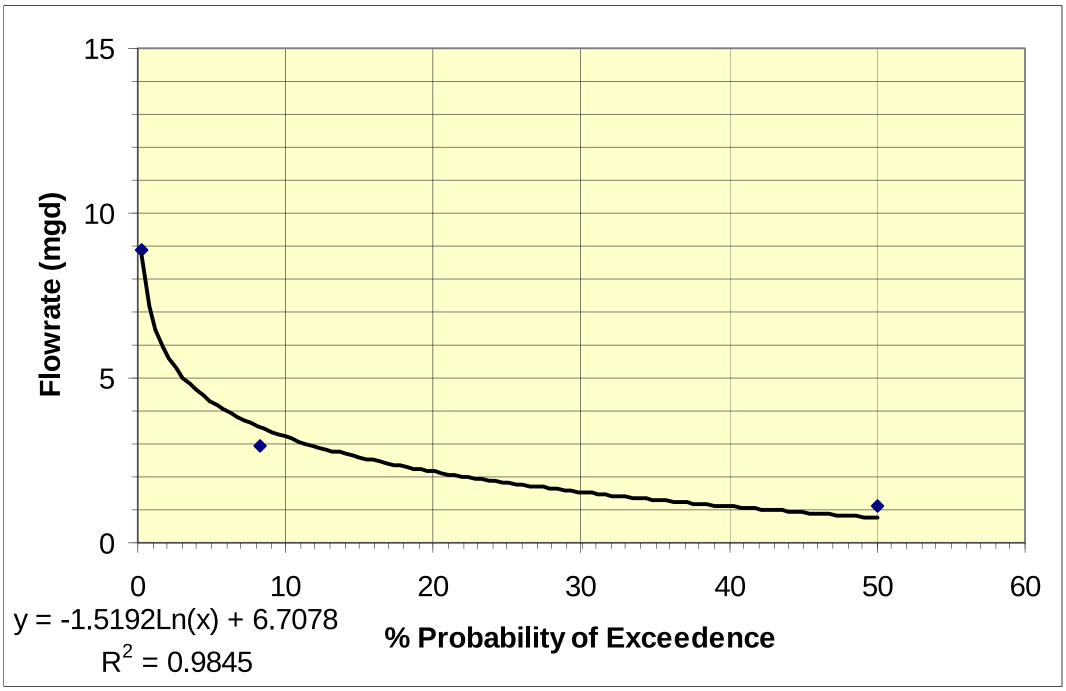


Figure 4-4: Plot of Average Annual flow, MMWWF (Method 2) and PDAF (Method 2) vs. % Probability of Exceedence per DEQ Methodology for Determining PIF.

Influent Solids Loading

Table 4-5 provides a summary of the TSS and CBOD loading for the past three years, based on plant data that provided approximately four daily concentration samples per month. Though 2004 maintained the highest average solids loading, CBOD was slightly higher in 2005. Thus, the 2004 total TSS loading and 2005 total CBOD loading were used for extrapolating 2030 plant loadings. Maximum month TSS and BOD loading is based on the maximum 30-day running average of the periodic TSS and BOD measurements from 2003 to 2005 (typically 3-5 independent samples per month).

Based on the values shown in Table 4-6, the average per capita CBOD and TSS loadings are 0.33 pounds per day and 0.24 pounds per day, respectively.

Table 4-5: Silverton Influent Organic Loading (TSS and CBOD)

Year	Average TSS (lb/d)	Max. Month TSS (lb/d)	Average CBOD (lb/d)	Max. Month CBOD (lb/d)
2003	1530	2180	2250	2870
2004	2300	5790	2780	3800
2005	2020	2880	2910	3740
Maximum Month Peaking Factor		**1.80**		**1.30**

Table 4-6: Summary of Peaking Factors (2003-2005)

Year	MMDWF	MMWWF	PDAF	PIF
2003	1.48	3.42	10.0	--
2004	1.15	3.38	6.07	--
2005	1.64	2.73	9.29	--
Average	1.65	3.18	8.45	
Oregon DEQ (Method 2)	1.90	3.72	11.3	17.3

Septage Flows and Loads

The amount of flow, TSS, and CBOD from septage is estimated to be 500 gpd, 30,000 mg/L, and 8,000 mg/L respectively. These are the same approximations as those used in the 1995 Facilities Plan, which were expected to remain constant as the City limited septic tank usage. Thus, loadings from septage yield constant values of 30 lb/day CBOD and 125 lb/day TSS.

Industrial Flows and Loading

Bruce Pac and Quest International effluent data were utilized as representative of the industrial flow component of the treatment plant influent. Table 4-7 provides a summary of the average flow, TSS, and CBOD loadings based on the City of Silverton's sanitary sewer utility bills for December 2004 to January 2006.

For comparison to the plant influent data, the ADWF was also calculated for each facility (utilizing the same technique as described previously), though it should be noted the industrial flow data indicated relative consistency throughout the year and was not subject to the same wet/dry seasonal fluctuations that affected the plant influent.

Table 4-7: Bruce Pac and Quest International (Dec 2004 – Jan 2006)*

Facility	Flow (gpd)	TSS (lb/d)	CBOD (lb/d)	ADWF (gpd)
Bruce Pac	73,300	520	888	63,600
Quest International	2,300	9.5	16	2,700

* Source: Based on Dec. 2004 – Jan. 2006 City of Silverton sanitary sewer utility bills.

These values represent average monthly contributions, and do not account for peak contributions associated with activities such as cleaning. Representatives of Bruce Pac indicate that cleaning operations currently can generate approximately 160,000 gallons per day of flow, and that under future conditions this could grow to 200,000 gallons per day.

Projected Future Flows and Loadings

Projected future flows and loadings are based on the following wastewater contributions:

- Future residential/commercial average and peak contributions
- Contributions from existing industries (including septage haulers)
- Potential contributions from future industries (including septage haulers)

Residential/Commercial Projections

Residential/commercial flow and loading projections were calculated based on the 2030 projected population, per capita flow and loading values given in the previous section, and average peaking factors listed in the previous section. Influent total kjeldahl nitrogen (TKN) and ammonia (NH_3) loadings are based on industry average per capita loading. Future wet weather peaking factors were reduced by 15% compared to the historical average peaking factors, based on the assumption that ongoing inflow and infiltration (I/I) control programs and improved construction materials and practices will continue to reduce inflow and infiltration. This assumption is consistent with the 1995 Facilities Plan.

Resulting projections for the year 2030 are shown in Table 4-8 below.

Table 4-8: Projected 2030 Residential/Commercial Flow and Loading

	Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day)	NH3 (lb/day)
Dry Weather					
ADWF	1.44 *	4,643	3,421	635	396
MMDWF	2.38	6,036	6,158	825	515
MWDWF	2.80				
Wet Weather					
AWWF	2.28				
MMWWF	3.90	6,036	6,158	825	515
MWWWF	6.36				
PDAF	10.37				
PIF	15.47				

* Includes 0.2 MGD to account for infiltration at the wastewater treatment plan (on average, measured plant effluent exceeds influent by approximately 0.2 MGD).

Industrial Projections

Future industrial contributions were calculated for average and maximum day conditions. Average industrial contributions are based on the City's 2005 Industrial Survey, and include an allowance of 150,000 gpd for new industrial development (with average influent concentrations similar to those from BrucePac)[2]. Maximum day industrial projections apply a peaking factor of 2.0 to flows from both BrucePac and the new industry. This results in a maximum day flow from BrucePac of 171,000 gallons per day, which is consistent with BrucePac's estimation that future peak flows could be between 160,000 gallons per day and 200,000 gallons per day. No peaking factors were applied to the industrial loading contributions. Anticipated 2030 industrial contributions are shown in Table 4-9 below.

Table 4-9: Projected 2030 Industrial Flow and Loading

	Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	NH3 (lb/day)
Average				
BrucePac	0.09	1,072	786	140
Quest	0.02	143	79	19
Septage	0.01	30	125	20
New Industry	0.15	1,877	1,376	245
Total – Average	**0.26**	**3,122**	**2,367**	**424**
Max. Day				
BrucePac	0.17	1,072	786	140
Quest	0.04	143	79	19
Septage	0.01	30	125	125
New Industry	0.30	1,877	1,376	245
Total – Max. Day	**0.52**	**3,122**	**2,367**	**529**

[2] This assumption supports the development of relatively water-intense industries on vacant industrial land included in the City's current Comprehensive Plan.

Recommended Future Projections

Combining the residential/commercial and industrial projections from Table 4-8 and Table 4-9 results in the total future projected flows and loadings in Table 4- below. The table also includes the current facility design flow for comparative purposes.

As Table 4- illustrates, the projected 2030 flows are lower than the current design flow capacity for all conditions other than peak day and peak instantaneous flows. Current design BOD, TSS, and ammonia loadings are not shown in Table 4-, as they are lower than the projected 2030 loadings.

Table 4-10: Projected 2030 Total Flow and Loading

	Projected Flow (MGD)	Current Design Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day)	NH3 (lb/day)
Dry Weather						
ADWF	1.71	2.5	7,765	5,788	1,313	821
MMDWF	2.65	4.3	9,158	8,525	1,504	940
MWDWF	3.06	N/A				
MDDWF		6.0				
Wet Weather						
AWWF	2.54	4.6				
MMWWF	4.17	6.6	9,158	8,525	1,504	940
MWWWF	6.62	N/A				
PDAF	10.89	10.0				
PIF	15.73	12.0				

The differences between projected and design flows stem from the analysis of baseline flow conditions. The average dry weather flow projection in the 1995 Facility Plan included a baseline sanitary flow component of 90 gpcd, and a “baseline I/I” component of 78 gpcd, resulting in a total per capita flow under average dry weather conditions of 168 gpcd. The baseline sanitary flow of 90 gpcd correlates with the analysis of recent flow records and with the 2005 System Development Charge (SDC) study; however, the recent data does not support including a “baseline I/I” contribution as part of the ADWF.

Because the design capacity has already been provided as part of the previous facility upgrades, the current design capacity will be used as the future planning basis for all flow conditions other than PDAF and PIF. For those two flow conditions (as well as CBOD, TSS, and nutrient loadings) the projected 2030 values will serve as the future planning basis. This approach results in the recommended facility plan flow and loading values shown in Table 4-10.

Table 4-10: Recommended Facility Plan Flow and Loading

	Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day	NH3 (lb/day)
Dry Weather					
ADWF	2.5	7,800	5,800	1,300	820
MMDWF	4.3	9,200	8,500	1,500	940
MWDWF					
MDDWF	6.0				
Wet Weather					
AWWF	4.6				
MMWWF	6.6	9,200	8,500	1,500	940
MWWWF					
PDAF	11				
PIF	16				

Chapter 5 - Water Quality and Regulatory Issues

Wastewater treatment and the discharge and reuse of effluent and residuals are controlled under the Clean Water Act, with regulations administered by the Environmental Protection Agency (EPA). In Oregon, regulatory programs related to wastewater treatment, disposal, and reuse are implemented and monitored by the Department of Environmental Quality (DEQ), with limits established for the City of Silverton through a National Pollutant Discharge Elimination System (NPDES) permit.

Regulatory requirements continue to evolve through an array of federal, state, and local programs, leading to new requirements for the City of Silverton. This chapter summarizes these trends and their implications on the City.

Effluent Discharge Limitations

The federal Water Pollution Control Act is the primary legislation that protects surface waters, including lakes, rivers, and coastal areas. This 1972 legislation, which became known as the Clean Water Act (CWA), provides the foundation for monitoring and reducing water pollution. There are several programs under the CWA that either directly regulate or contribute to the regulation of WWTP effluent quality. These programs include:

- Section 402: National Pollutant Discharge Elimination System (NPDES) Discharge
- Section 303(d): Identification and Protection of Surface Water Uses
- Total Maximum Daily Load (TMDL): Point and Non-Point Loads for Pollutants
- Sanitary System Overflow (SSO) Rule: Capacity, Management, Operations, and Maintenance of Sanitary Sewer Systems

NPDES Discharge Permit

Discharging treatment plant effluent to surface water requires an NPDES permit from DEQ. This discharge method is governed by OAR 340-41. The City's existing NPDES permit, included in Appendix A, was issued on August 2, 2005 and expires on December 31, 2009. This permit stipulates water quality criteria for all regulated discharges, which include the City's outfall to Silver Creek at River Mile 2.35, an emergency overflow from the existing surge basin, and discharge to the Oregon Garden wetlands. This permit reflects compliance with current water quality standards, but may be modified by the Molalla-Pudding TMDL currently under development. The current permit limits are summarized in Table 5-1 and Table 5-2 below.

Table 5-1: NPDES Permit Limit Effluent Discharge Limitations (Outfall 1 – Silver Creek)

Parameter	Average Monthly Concentration (mg/l)	Average Weekly Concentration (mg/l)	Monthly Average (lb/day)	Weekly Average (lb/day)	Daily Maximum (lbs)
May 1 – October 31					
$CBOD_5$	10	15	300	330	420
TSS	10	15	300	330	420
Ammonia-nitrogen[1]	Shall not exceed monthly average concentration of 0.88 mg/l and a daily maximum concentration of 2.0 mg/l				
Excess thermal load[2]	Shall not exceed a weekly average of 5.2 million Kcals/day				
November 1 – April 30					
$CBOD_5$	25	40	830	1100	1500
TSS	30	45	1300	1700	2200
Excess thermal load[3]	Shall not exceed a weekly average of 21 million Kcals/day				
Year-Round					
Dissolved oxygen	Shall not be less than 6.5 mg/l as a daily average				
E. coli bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.				
pH	Shall be within the range of 6.5 – 9.0				
$CBOD_5$ and TSS removal efficiency	Shall not be less than 85% monthly average for $CBOD_5$ and TSS				

[1] Permit limit becomes effective upon *expiration* of the permit or four years following approval of the Molalla-Pudding TMDL, whichever is sooner.

[2] Excess thermal load limit becomes effective upon expiration of the permit or four years following approval of the Molalla-Pudding TMDL, whichever is sooner. Compliance period for summer excess thermal load limit is May 16 through October 14.

[3] Excess thermal load limit becomes effective upon expiration of the permit or four years following approval of the Molalla-Pudding TMDL, whichever is sooner. Compliance period for winter excess thermal load limit is October 15 through May 15.

Table 5-2: NPDES Permit Limit Effluent Discharge Limitations (Outfall 2 – Oregon Garden)

Parameter	Average Monthly Concentration (mg/l)	Average Weekly Concentration (mg/l)	Monthly Average (lb/day)[1]	Weekly Average (lb/day)[1]	Daily Maximum (lbs)[1]
$CBOD_5$	10	15	300	330	420
TSS	10	15	300	330	420
Ammonia-nitrogen	Temperature dependent, ranging from 1.3 mg/l monthly average and 3.0 mg/l daily maximum at monthly average effluent temperature < 12°C to 0.84 mg/l monthly average and 1.9 mg/l daily maximum at monthly average effluent temperature > 24°C.				
Dissolved oxygen	Shall not be less than 5.5 mg/l as a daily average.				
E. coli bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.				
pH	Shall be within the range of 6.5 – 9.0				

[1] The mass load of CBOD5 and TSS in the combined discharge from Outfalls 001 and 002 shall not exceed the seasonally appropriate CBOD5 and TSS mass load limits for Outfall 001

Discharge from Outfall 003 (surge basin overflow) is prohibited unless it is due to storm events as allowed under OAR 340-041-0120 (13) and (14), which are defined as a one-in-five-year, 24-hour duration storm during the period of November 1 through May 21, and a one-in-ten-year, 24-hour storm during the period of May 22 through October 31.

303(d) Listing

The City discharges to Silver Creek, which is within the Molalla-Pudding subbasin of the Willamette basin. The Willamette basin supports numerous designated beneficial uses and has related water quality standards specified in OAR 340-041 to protect these beneficial uses.

Silver Creek is currently on DEQ's list of Water Quality Limited Water Bodies (303(d) list) for violations of the rearing and migration temperature criteria during the non-spawning period (May 16 to October 14) and for fecal bacteria during the summer. These violations occur from river mile 5.0 to the mouth. DEQ indicates that the City's discharge "likely has a minor if any impact on the water quality limited status of the river (DEQ, 2005b)." The permit limits contained in the current NPDES permit are considered the City's Bacteria Control Management Plan for managing impacts to the current 303(d) listing.

TMDL Development

Oregon DEQ is in the process of developing a TMDL to address 303(d) water quality limitations in the Molalla-Pudding subbasin. Listed parameters to be addressed in the TMDL include:

- Arsenic
- Chlordane
- DDT
- Dissolved oxygen
- Bacteria
- Iron
- Manganese
- Nitrates
- Temperature

The TMDL is currently scheduled for completion by the end of 2007. The City has six months following completion of the TMDL to notify DEQ regarding whether the facility can comply with the ammonia limits in the permit and with any Waste Load Allocation (WLA) established in the TMDL. If the City cannot consistently comply with the limits, the following compliance schedule will apply:

- No later than one year following TMDL approval, the City must submit an evaluation of alternatives for required facility improvements.
- No later than two years following TMDL approval, the City must submit final engineering plans and specifications for required improvements.
- No later than three years following TMDL approval, the City must submit documentation of award of construction contracts for necessary improvements.
- No later than four years following TMDL approval, the City must complete construction and comply with ammonia limits and TMDL.

EPA Peak Flow Policy

For several years, the EPA has been working to develop a policy implementing requirements regarding wet weather blending (diverting a portion of the plant flow around biological treatment processes) at municipal WWTPs. A proposed policy was issued in December 2005, and EPA is now reviewing public comments. Key provisions of the proposed policy (as described by the EPA) are:

- All diverted flows will receive a minimum of primary treatment.
- All effluent limits would continue to be met.
- Diversions will not be approved if peak flows are “largely due to poor collection system maintenance or lack of investment in or upgrades to treatment capacity” (EPA, 2005).

Potential Future Water Quality Requirements

Based on the City’s current NPDES permit, guidance from DEQ, and potential WLAs from the Molalla-Pudding TMDL, potential future water quality requirements are described below.

BOD/TSS

Increases in mass load limits in Oregon require approval by the Environmental Quality Commission (EQC). Such approval is unlikely to be granted; therefore, future planning should assume that allowable mass load discharges in the current permit will be carried forward in future permit cycles.

Ammonia-Nitrogen

The ammonia-nitrogen (ammonia) limits in the City’s current permit are based on EPA’s 1986 Quality Criteria for Water and OAR 340-41, under which toxic concentrations of ammonia are both pH and temperature dependent. Based on the 1986 criteria, the City’s discharge has the potential to create toxic conditions in the receiving water; therefore, effluent ammonia limits were applied.

In 1999, the EPA revised its criteria for evaluating ammonia toxicity. Oregon has adopted new ammonia criteria based on the EPA’s revised criteria, and is now waiting for the EPA to approve the State’s revised criteria. While the City’s discharge is still toxic under the 1999 criteria, the allowable effluent ammonia concentrations are significantly higher than under the 1986 criteria. The effluent limits that would be imposed under the new criteria are listed below, and would become effective upon EPA approval of the 1999 criteria without a formal permit modification.

- Outfall 1: May 1 to October 31 – Monthly average concentration of 3.0 mg/l and daily maximum concentration of 7.8 mg/l
- Outfall 2: Temperature dependent, ranging from 4.4 mg/l monthly average and 10.0 mg/l daily maximum at monthly average effluent temperature > 12°C to 2.0 mg/l monthly average and 4.6 mg/l daily maximum at monthly average effluent temperature > 24°C.

The alternatives analysis included in this Facility Plan will identify recommended improvements to meet the effluent limits required under the 1986 criteria; however, implementation of any improvements should not occur until the EPA has acted on the State's proposed revisions.

Temperature

The Oregon Temperature Standard (OAR 340-041-0028) establishes temperature requirements for Oregon streams based on biological conditions required to support endangered salmonids, the natural thermal potential of the stream, protection of cold water fisheries, and allowable increases due to human use. The Oregon Gardens wetlands do not support salmonids and waters from the wetlands are used primarily for irrigation, so the Temperature Standard only applies to the City's discharge to Silver Creek.

The TMDL process includes evaluation of "natural conditions" in a water body, and allowable thermal discharges will ultimately be based on this criterion. Prior to completion of the TMDL, however, the allowable discharge is determined based on the designated biological criteria for the receiving stream. Designated uses for Silver Creek and associated allowable temperature increases are shown in Table 5-3 below.

Table 5-3: Oregon Temperature Standard Implications for Silver Creek*

Period	Designated Beneficial Use	Human Use Allowance
Winter (October 15 – May 15)	Salmon and steelhead spawning	0.3°C
Summer (May 16 – October 14)	Salmon and steelhead rearing	0.5°C

* OAR 340-041-0028

The City's current discharge has a reasonable potential to exceed the allowable increases listed in Table 5-3; therefore, the current NPDES permit established excess thermal load limits for the two periods. The excess thermal load limits are considered interim, and can be modified based on the outcome of the Molalla-Pudding TMDL process.

The recently-completed Willamette TMDL may provide some guidance regarding potential temperature limits for the Molalla-Pudding Basin. Modeling completed for the Willamette Basin showed that "*the river naturally exceeds standards for protecting salmon during warmest months. When this occurs, the natural condition is used to set pollutant limits*[3]." However, in some sub-basins, the TMDL allocates only a portion of the human use allowance to point source discharges. Overall, for planning purposes, it is assumed that future thermal load limits developed in the Molalla-Pudding TMDL will be similar to the excess thermal load limits in the current permit.

[3] Willamette Basin Total Maximum Daily Load, Oregon Department of Environmental Quality, September 2006.

Turbidity

The DEQ is currently in the process of revising the turbidity standard described in OAR 340-041-0038. The current draft criteria would impose a numerical limit allowing increases of no more than 5 NTUs maximum or 3 NTUs on a monthly average basis. Under the draft guidelines, sources that have a reasonable potential to cause or contribute to an exceedence of the turbidity criteria will be assigned numerical effluent limits calculated to meet the turbidity criteria at the edge of the permitted mixing zone (DEQ 2005d). Based on measured effluent turbidity and available background turbidity measurements from Silver Creek in the vicinity of the City's outfall, the new standard could result in permitted effluent concentrations of approximately 4-5 NTU on a monthly average basis and 7-8 NTU maximum.

The proposed changes have not yet been finalized by DEQ staff. Once finalized, the revised standard will be reviewed and approved by the EQC, and finally become effective upon approval by the EPA.

Based on historical effluent data, the City's ability to comply with potential future permit limits is marginal. The Facility Plan alternatives will examine process improvements to provide additional effluent filtration; however, implementation of any recommendations will be contingent upon DEQ finalizing the revised turbidity standard.

Toxics

The DEQ uses a Reasonable Potential Analysis (RPA) to evaluate whether potentially toxic compounds in a discharger's effluent have a reasonable potential to result in an exceedance of a water quality criterion. During the City's last permit renewal cycle, an RPA was completed for ammonia, cyanide, metals, and toxic organics discharged to Silver Creek and the Oregon Garden wetlands. The RPA indicated that cadmium, copper, cyanide, lead, mercury, selenium, silver, and zinc are all parameters of concern. However, the RPA was based on a very limited data set for both the effluent and receiving waters, and in many cases the metals were detected at or near detection levels.

The DEQ has developed an Internal Management Directive (IMD) on Reasonable Potential Analysis (DEQ 2005c). This IMD establishes effluent and receiving water monitoring data that must be submitted by NPDES permittees with permit renewal dates after January 1, 2007. Silverton was notified of these new monitoring requirements on January 6, 2007.

Other TMDL Constituents

Of the identified TMDL constituents, only temperature and bacteria are anticipated to be addressed through the City's NPDES permit. As described above, the temperature listing will be addressed through a thermal load limit in the City's permit, which may be slightly different than the limit included in the current permit. Based on input from DEQ's TMDL author, compliance with the current E. Coli bacteria limit (126 organisms per 100 mL) will result in compliance with the requirements of the TMDL.

The other constituents in the TMDL are not related to point source discharges, or are difficult to address in a point source waste load allocation. High levels of iron, manganese, nitrates, and arsenic in the watershed are due to background contributions from natural sources, and therefore limits for these constituents will not be established in the TMDL. Legacy pesticides (Chlordane, DDT) are primarily introduced to waterways through runoff, so limits on discharges of these constituents may be addressed through the development of load allocations or waste load allocations for TSS. It is anticipated that compliance with any potential TSS limits generated through the TMDL will be achievable through conventional wastewater treatment technology.

DEQ recently issued the Willamette Basin TMDL (Willamette TMDL) for approval by EPA. This document includes a Mercury TMDL, which will apply to all discharges in the Willamette Basin. The TMDL allocates mercury loads to point and nonpoint sources in general, but does not include limits for specific point or nonpoint source dischargers. For purposes of this Facility Plan, it is assumed that any required mercury reduction measures would first focus on nonpoint sources and industrial discharges, and would not impact the evaluation or recommendation of process improvements at the wastewater treatment plant.

Compounds of Emerging Concern

In addition to the traditional measures of water quality, there is increasing interest in a group of synthetic or naturally-occurring chemicals collectively known as Compounds of Emerging Concern (CEC). These compounds are not commonly monitored in wastewater effluent or in natural water bodies, but may have the potential to cause ecological or human health effects. CECs include pesticides, pharmaceuticals and endocrine disrupting compounds (EDCs), and industrial chemicals.

Significant research efforts are currently underway to build an understanding of the sources of these CECs, their individual and collective impacts on aquatic ecosystems, and their fate and transport in wastewater treatment processes and in biosolids. Due to the emerging nature of this issue and the lack of a complete scientific body of knowledge, it is unclear whether or how CECs will be regulated at a state or federal level. It will be important for the City to track developments related to this issue, and to partner with other dischargers (such as through continued involvement in the Oregon Association of Clean Water Agencies) to help shape and provide feedback regarding future regulatory policies related to CECs.

Biosolids Management

Biosolids management is governed by the Code of Federal Regulations (40 CFR 503), implemented in Oregon in OAR 340 Division 50. The 503 regulations are broad-based, addressing general requirements, pollutant limits, management practices, operational standards, monitoring frequency and record-keeping requirements, reporting requirements, and pathogen and vector attraction reduction requirements for treatment and disposal of municipal wastewater sludge. All common disposal practices including land application, surface disposal, and incineration are all covered in the regulations. From a biosolids treatment perspective, major impacts of the 503 regulations include pathogen reduction requirements, vector attraction requirements (VAR), limits on metals content, and operation and performance requirements for treatment processes.

Pathogen Requirements

The 503 regulations create two categories of biosolids with respect to pathogens: Class A and Class B. Class A biosolids are an essentially pathogen-free product that can be used without restriction. Class B biosolids are not a pathogen-free product, but can be applied to agricultural land, forest land, or reclamation sites approved by the DEQ. Regulations require that crop harvesting, animal grazing, and public access be restricted for specific periods of time after the application of Class B biosolids.

To meet Class B pathogen reduction measures, biosolids must be treated with a Process to Significantly Reduce Pathogens (PSRP), or an equivalent process. Approved PSRPs include aerobic digestion, anaerobic digestion, composting, lime stabilization, and air drying. Anaerobic digestion such as that currently used at Silverton must meet a 15-day solids retention time.

Class A biosolids must be treated using an EPA-approved Process to Further Reduce Pathogens (PFRP) or equivalent process. Approved PFRPs include composting, lime stabilization, heat drying, heat treatment, thermophilic anaerobic digestion, beta ray irradiation, gamma ray irradiation, or pasteurization. There are no site restrictions or additional management practices for Class A biosolids use.

Vector Attraction Reduction Requirements

The 503 regulations also require VAR prior to disposal or land application in order to make the material less attractive to insects, rodents, and other vectors. Table 5-4 summarizes accepted vector attraction methods for biosolids. Exceptional quality (EQ) biosolids can be produced by meeting the Class A pathogen content requirements and using Methods 1 through 8 of Table 5-4 to meet VAR requirements.

Table 5-4: Vector Attraction Reduction Measures for Biosolids

Method	Description
1	Meet 38% reduction in volatile solids
2	Demonstrate vector attraction reduction with additional anaerobic digestion in bench-scale unit
3	Demonstrate vector attraction reduction with additional aerobic digestion in bench-scale unit
4	Meet a specific oxygen uptake rate for aerobically digested biosolids
5	Use aerobic processes at greater than 104°F for 14 days or longer
6	Alkali addition under specified conditions
7	Dry sludge with no unstabilized solids to at least 75% solids content
8	Dry sludge with unstabilized solids to at least 90% solids content
9	Inject sludge beneath the soil surface
10	Incorporate sludge into the soil within 6 hours of application

Trace Elements

Eight trace elements commonly found in biosolids (arsenic, cadmium, copper, lead, mercury, nickel, selenium, and zinc) are regulated through Part 503. The regulations distinguish between biosolids sold or given away in a bag or other container, and bulk sewage sludge. Bulk sewage sludge applied to agricultural land, forest sites, public contact sites, or reclamation sites must comply with either a specified cumulative pollutant loading rate or a monthly average pollutant concentration. Biosolids sold or given away in a container must have pollutant concentrations no higher than the ceiling concentrations stipulated in the 503 regulations, and must be within allowable annual loading rates.

Agronomic Application Rates

One of the general requirements for land application of biosolids is that the application must be performed at an agronomic rate to minimize the migration of nutrients to groundwater. Historically, agronomic rates have been evaluated based on nitrogen uptake, with the goal of preventing the migration of nitrate into groundwater. However, some states are beginning to monitor agronomic uptake based on both nitrogen and phosphorus. Managing biosolids land application to meet agronomic phosphorus uptake rates can have significant impacts on facilities that achieve excess biological phosphorus removal, increasing the amount of land required to maintain a biosolids land application program.

Biosolids Management Plan

Beneficial use of biosolids must be managed in accordance with a current, DEQ-approved biosolids management plan. DEQ describes the function of the Biosolids Management Plan as follows:

"A biosolids management plan is the main administrative tool of Oregon's biosolids program. It is specific to a facility and is used to guide the wastewater treatment facility's solids operations and biosolids land application activities. Together with a facility's water quality permit and land application site authorizations, the plan provides assurance that biosolids processing and management activities are addressed in a comprehensive manner and problems with compliance are minimized" (DEQ 2005e).

Effluent Reuse

Water quality requirements for recycled water are defined in the Oregon Reuse Rules (OAR 340 Division 55) adopted in 1990. DEQ classifies reclaimed water in four categories: Level I through Level IV. Level IV treatment requirements are the most stringent, allowing reclaimed water to be used on areas open to general public contact and allowing unrestricted use for agricultural irrigation. Treatment requirements for use of reclaimed water are described in Table 5-5 below.

Table 5-5: Treatment Requirements for Use of Reclaimed Water

Category	Level I	Level II	Level III	Level IV
Biological Treatment	X	X	X	X
Disinfection		X	X	X
Clarification				X
Coagulation				X
Filtration				X
Total Coliform (Organisms/100 m/L)				
Two Consecutive Samples	N/L	240	N/L	N/L
7-Day Median	N/L	23	2.2	2.2
Maximum	N/L	N/L	23	23
Sampling Frequency	N/R	1 per week	3 per week	1 per day
Turbidity (NTU):				
24-Hour Mean	N/L	N/L	N/L	2
5% of Time During 24-Hr Period	N/L	N/L	N/L	5
Sampling Frequency				Hourly

N/L – No Limit
N/R – Not Required

The DEQ is currently in the process of revising the Division 55 reuse rules, and has established a Water Reuse Task Force to make recommendations to DEQ to reduce regulatory barriers and encourage effluent reuse.

Groundwater Regulations

Any discharge that may impact groundwater must meet Oregon standards for groundwater protection. These standards are outlined in Division 40 of the Oregon Administrative Rules (OAR 340-040-0001 through 340-040-0210). The standard most applicable to wastewater treatment plants is that for nitrate-N, with a limit of 10.0 mg/l total (unfiltered) concentration.

The City's current operation has been determined by the DEQ to have a low potential for adversely impacting groundwater quality; therefore, no groundwater monitoring is currently required.

Air Quality Regulations

Air pollutant emissions are regulated under the Clean Air Act (CAA), the Clean Air Act Amendments of 1990, and Oregon air contaminant discharge permit (ACDP) and Title V programs. Sources emitting regulated pollutants can be classified as either minor or major sources based on total annual pollutant loading. Silverton's WWTP does not currently have an air quality permit, and future expansion is not anticipated to trigger permitting action during the horizon of this Facility Master Plan.

CMOM

CMOM is a program that was proposed to prevent sanitary sewer overflows (SSOs) and WWTP overloading through proactive management of the collection system. While the rule has not been promulgated since a draft was issued in 2003, several state and regional regulatory agencies have implemented CMOM-like requirements. For example, in early 2006, the California State Water Resources Control Board adopted new requirements that all regulated entities complete Sanitary Sewer Management Plans (SSMPs) addressing proper management, operation, and maintenance of sanitary sewer collection systems.

The primary purpose of a CMOM-type program is to require system owners to take a proactive approach to preventing sewer overflows. Implementation of a CMOM program would also help demonstrate adherence to best management practices for utilities seeking to gain approval to blend under the new EPA Peak Flow Policy. Key elements of a CMOM Plan include:

- Summary of the Utility's Sewer Management Program
- Overflow Response Plan
- System Condition and Capacity Analysis
- Communication Plan
- Routine Program Audit

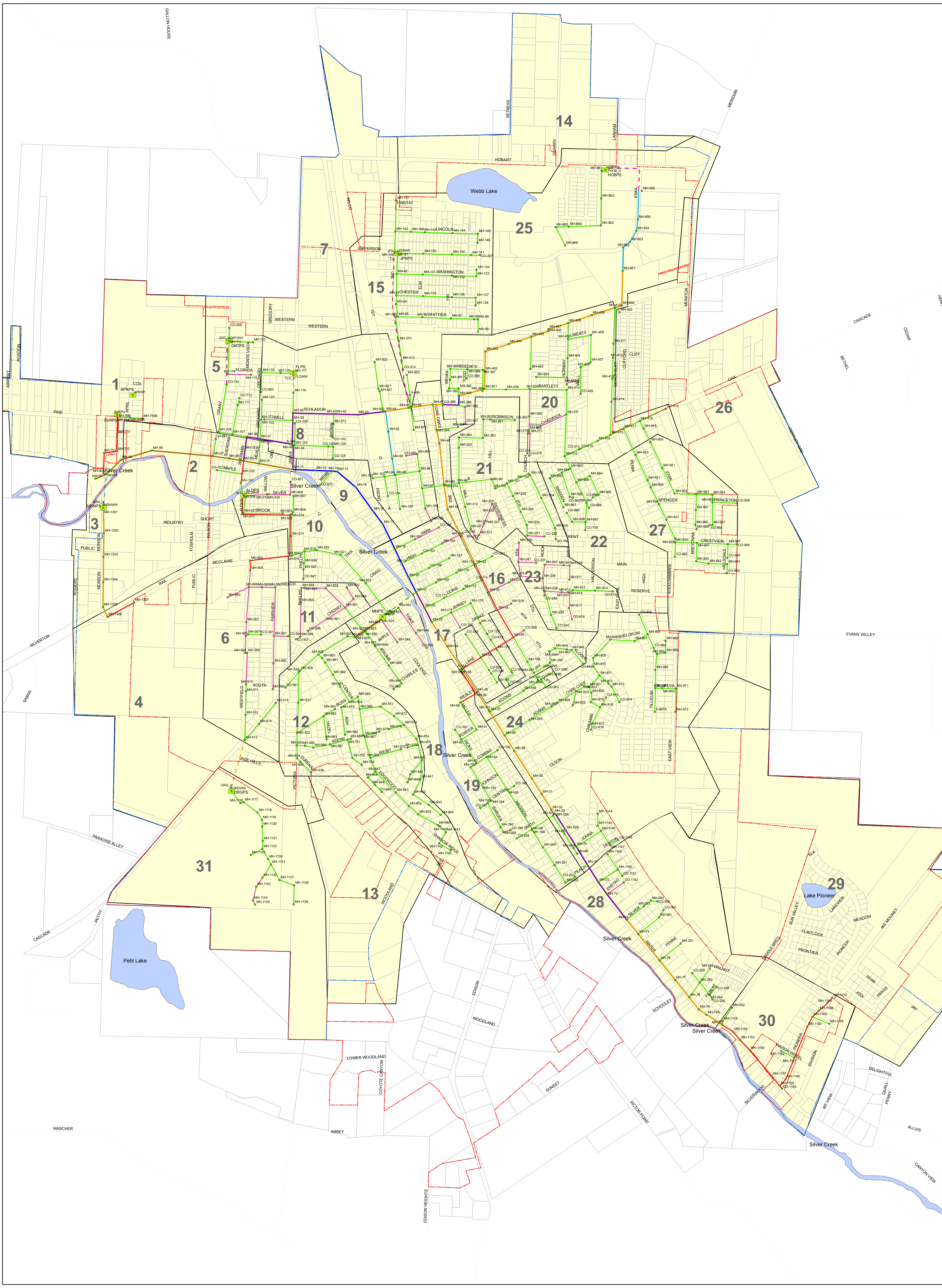
Chapter 6 - Existing Collection System

Background

Silverton's wastewater collection system is a conventional gravity system dating back to 1910. Major additions to the collection system were made in 1923, 1939, 1964, and 1983. In 1983, interceptors and trunk sewers were constructed as part of the improvement program for both the collection system and the wastewater treatment plant (WWTP). Few other collection system additions were constructed in the 1980s. Since 2005 major additions have been made to serve new subdivisions and the industrial park. The collection system now services approximately 910 acres of the 2570 acres within the UGB. (Figure 6-1 shows the present collection system and basin boundaries.

Typical of comparable systems, Silverton's system includes different types of pipe materials. Prior to 1939, the pipe materials consisted of vitrified clay pipe with cement mortar joints. The 1930 additions were constructed with concrete pipe and mortar joints. In 1964, additions made in north and south Silverton were constructed of concrete pipe with rubber-ring gasketed joints. Subsequent additions included the Eureka area and the majority of the 1983 interceptors, which were constructed of rubber-gasketed concrete pipe. Recent additions to the collection system have utilized PVC pipe with rubber gaskets.

As a result of a Sanitary Sewer Evaluation (SSES) completed in 1978, a major rehabilitation of Silverton's wastewater collection system was undertaken in the early 1980s. The City replaced or augmented approximately 9 percent of their trunk lines, root-treated approximately 11 percent of their system, and cleaned approximately 16 percent of the system. The City also undertook the separation of known sources of inflow into the sanitary system. Although the trunk line augmentation removed all direct bypasses into Silver Creek, high rainfall-related flows are still seen in the system.



Gravity Main
- 6 inches
- 8 inches
- 10 inches
- 12 inches
- 15 inches
- 18 inches
- 21 inches
- 30 inches

Pressure Main
- 4 inches
- 6 inches
- 8 inches
- 10 inches

- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies
- Sewer Basin

N
W E
S

0 0.09 0.18 0.27 0.36 0.45
Miles

Figure 6-1
City of Silverton
Sanitary Collection System

HDR | ONE COMPANY *Many Solutions*℠

Sewers

The Silverton service area, both present and future, is divided into 31 basins. The length of each City-owned, active pipe by size are summarized by basin in Table 6-1.

Table 6-1: Lineal Feet of Sewer Main per Drainage Basin

Basin Number	2"	4"	6"	8"	10"	12"	15"	18"	21"	30"	Grand Total
1		360	50	917	526	956				301	3,110
2				474		167		145		2,009	2,794
3				16		20	1,416				1,452
4					0	440	152				592
5			0	1,727				178			1,905
6	458		3,396	1,603		725					6,182
7											
8		0	199	4,850	1,125		1,057	2,302			9,533
9			0	256					2,003		2,259
10	341		921	492		1,518					3,279
11			1,591								1,591
12				11,415							11,415
13											
14											
15				1,411							1,411
16			694	2,668		1,157	3,250				7,768
17			478	454		908	1,496	475	679		4,489
18		31	0	2,179		485					2,695
19			111	6,127		620					6,857
20			1,982	11,306			2,825	206	121		16,440
21		25	1,255	3,279							4,559
22			45	3,656							3,701
23			2,789	1,153							3,941
24				8,451			1,691	846			10,988
25				1,753	1,197		594				4,344
26				96							96
27				6,706							6,706
28			0	3,429			2,010	976			6,415
29											
30				716		2,701	53				3,470
31			353	1,935							2,288
TOTAL	**799**	**416**	**13,864**	**77,069**	**2,848**	**9,697**	**14,544**	**5,128**	**2,803**	**2,310**	**129,478**

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Pump Stations

Eight pumping stations convey wastewater to the WWTP. Table 6-2 summarizes information about the pumping stations.

Table 6-2: Pumping Station Summary

Location	Type	Number of Pumps	Design Capacity of Each Pump (gpm)	Power (hp)
Silver and Alder Ave.	Submersible	2	200	2
April Lane	Submersible	2	150	2
James and Florida Drive	Submersible	2	200	5
Grant Street	Submersible	2	200	5
Hobart Road	Submersible	2	325	5
Second and Jefferson Street	Dry pit	2	500	15
Monson Road*	Submersible	2	400	5
West Main Street	Submersible	2	900	20

* This pump station is expected to be on-line in the Spring of 2007.

All submersible stations are equipped with Flygt pumps. The four older submersible stations – Silver and Alder Pump Station, Grant Street Pump Station, James and Florida Pump Station, and West Main Street Pump Station – were constructed or reconstructed in 1983. The Second and Jefferson Street Pump Station was constructed in 1964; however, the pumps in the station were replaced as part of the 1983 improvements. It is equipped with two Allis-Chalmers centrifugal pumps.

Currently, the Monson Road Pump Station is not fully developed. The wet well and force main are in place; however, no pumps have been installed. It is expected that this pump station will be on-line in the Spring of 2007.

There are other privately owned pump stations that contribute flow to the system. One of note is operated by and located at the Oregon Gardens. This pump station transfers water from their facility to the collection system. Plans for a new hotel in this area may necessitate the conversion of this pump station from private to publicly owned. This will be discussed further in Chapter 8, Collection System Master Planning.

Condition Assessment

Leak Busters, Inc. carried out an electro-scan study of approximately 6,000 feet of sanitary sewer pipe using the Metrotech Focused Electrode Leak Location system (FELL-41™) to assist with leakage assessment of sanitary sewers in connection with the Wastewater System Facility Master Plan.

Sewer Description

The pipes tested were 8-to 18-inch diameter vitreous clay pipe (VCP) sanitary sewers. Access to the sewers was through manholes (MH) with an average separation of 350 feet and depth of 8 feet.

Manhole names of the electro-scanned sewer sections are shown on the sewer plans supplied by HDR. The manhole-to-manhole distances (measured from the center of the manholes) are shown in the results: Each manhole-to-manhole test section is referenced by the upstream manhole and the street name. The sewer segments tested are shown in Table 6-3**.**

Table 6-3: Sewer Segments Inspected

Start Manhole	End Manhole	Start Manhole	End Manhole
MH-114 Grant	MH-111 Grant	MH-052 Third	MH-051 Third
MH-115 Florida	MH-114 Grant	MH-051 Third	MH-050 Third
CO-206 Grant	PS	MH-050 Third	MH-049 Third
MH-113 Monte Vista	MH-112	MH-049	MH-048
MH-112	MH-111 Grant	MH-048 Third	MH-398 Loan Oaks
MH-063 Hicks	MH-062 Porter	MH-394 Loan Oaks	MH-047 Loan Oaks
MH-062 Porter	MH-061 Miller	MH-047 Loan Oaks	MH-046 Roths
MH-061 Miller	MH-060 Wesley		**Total for day**
	Total for day	MH-046 Roths	MH-044 Meat Packers
MH-055 Third	MH-054 Third	MH-044 Meat Packers	MH-043 High School
MH-054 Third	MH-053 Third	MH-043 High School	MH-041 Schlador
MH-053 Third	MH-052 Third	MH-041 Schlador	MH-040 JAMES
MH-056 Lane	MH-055 Third	MH-060 Wesley	MH-059 Water
	Total for day		

Methodology

Technology

The sewer electro-scan carried out by the FELL-41™ utilizes the variation of electric current flow through a sewer pipe wall to locate pipe defects that are potential water leakage paths.

Most sewer pipe materials such as clay, plastic, concrete, asbestos-reinforced concrete, and brick are electrical insulators; thus, have high resistance to electrical current. A defective pipe that leaks water will also leak electrical current, whether or not water infiltration is occurring at the time of the test.

The sewer electro-scan is carried out by applying an electric voltage between an electrode in the pipe (called a sonde) and an electrode on the surface (usually a metal stake pushed into the ground). A simplified electrical circuit for this procedure is shown in Figure 6-2. The water in the pipe is at a level ensures that the pipe is full at the sonde location. The electrical resistance of the current path between the sonde and the surface electrode is very low, except through the pipe wall. The high electrical resistance of the pipe wall prevents electrical current from flowing between the two electrodes unless there is a defect in the pipe, such as a crack, defective joint, or faulty service connection.

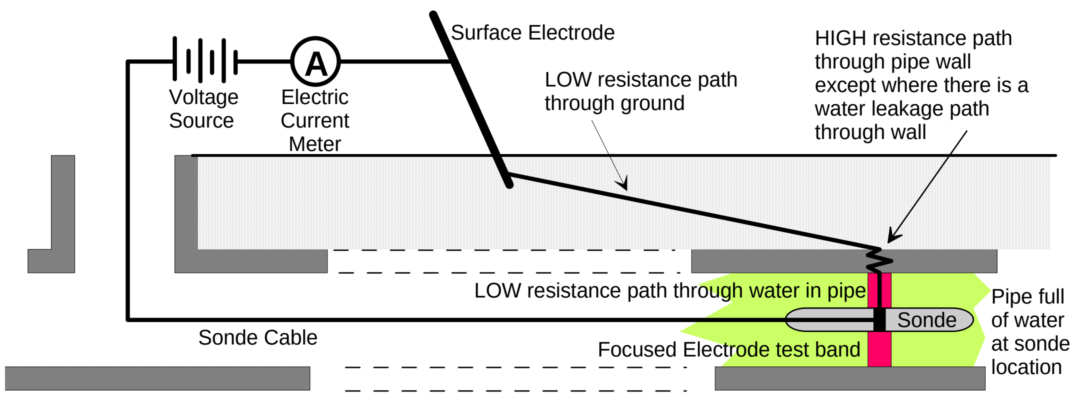


Figure 6-2: Electro-Scan Electrical Schematic

To detect defects around the complete circumference of the pipe wall, the sewer needs to be completely full of water in the sonde region. If the pipe is only partly full in the sonde region, then only that part of the pipe that is covered with water is tested.

The sewer electro-scan is carried out by pulling the sonde through a pipe at a speed of 30 ft/min. The current flow between the surface electrode and the sonde is recorded at approximately 0.5 inch intervals along the pipe. Most sewer pipe materials have high resistance to electrical current and there is only a small current flow except where there is a pipe defect. As the center of the sonde approaches within approximately 1 inch of a pipe defect, the current from the focused electrode increases, reaching a maximum when the center of the sonde is radially aligned with a defect.

Data Collection

As the sonde is pulled through the pipe, the electrical current flow through the pipe wall and the position of the sonde in the pipe are recorded and displayed in real time as an electro-scan on a notebook computer.

A region on the electro-scan where the sonde current level is above the threshold level is called an anomaly. The threshold level is shown as the lowest unbroken horizontal line on the electro-scan.

Data Analysis

The electro-scan is analyzed using a computer program in the following steps:

- Processing the data to remove the current offset above zero.

  This process enables a computer program to automatically pick and grade the electro-scan anomalies (see below).

- Setting a sonde current threshold level.

  The value of the threshold level was selected to provide discrimination between what might be “slightly” leaking joints or defects and other defects.

  For this study, the threshold level selected was 1.0 and is shown as the lowest unbroken horizontal line on the electro-scans. This threshold level was based on past experience of electro-scanning full pipes up to a diameter of 18 inch.

  Further testing or investigation may lead to modification of this threshold level.

- Grading the anomalies as Large, Medium, or Small according to the maximum value of the electro-scan anomaly.

  The Large-Medium and Medium-Small current level boundaries of 7.0 and 4.0 respectively are shown as unbroken horizontal lines on the electro-scan. The location and length of an anomaly is the location and longitudinal length of the electrical defect along the pipe. The maximum current level of the anomaly is a measure of the amount of current flow through the defect and is related to the size of the defect.

  For this study, the grading levels were selected from past experience of electro-scanning full pipes up to a diameter of 18 inch.

  The boundaries between Large, Medium, and Small may be refined using the results of other types of testing or investigation.

  These grades provide a means of establishing priority for further pipeline investigation and/or repair.

- Plotting joint locations.

  Anomalies that occur at regular intervals are usually caused by joint defects. To assist with the identification of these joint anomalies, the analysis program can be used by the operator to plot “+” marks on the electro-scan at a regular interval. The analysis program can then select anomalies that occur at the “+” marks and plot a “◇” over the “+”. These anomalies are considered to be associated with a joint defect. Other anomalies are usually due to structural faults or faulty service connections (SC).

- Tabulating anomalies and calculating relative anomaly occurrence.

  The analysis program detects, measures, and grades the size and type (joint or other) of the anomalies and calculates the total length of anomalies for each test section. This is a measure of the potential relative leakage for each manhole-to-manhole pipe section.

Results

The length of pipe electro-scanned is shown in Table 6-4.

Table 6-4: Length of Pipe Electro-Scanned

Pipe Diameter (in)	Length (ft)
8	1,538
12	1,939
15	2,439
18	1,296
Total	**7,212**

The pipe sections electro-scanned each day are shown in Table 6-5. Each test section is referenced by the upstream manhole number and street name. The distances shown on the electro-scans are in the downstream direction and begin from the upstream start of the pipe test section. The electro-scans have been plotted so the left-hand manhole on the electro-scan is the upstream manhole.

The processed electro-scans of the sewer segments tested are shown in Appendix D. Manhole names and comments concerning particular anomalies are also shown on the electro-scans.

All test sections were analyzed using the same threshold level of 1.0 and the same anomaly grade levels of 7.0 for the Large-Medium and 4.0 for the Medium-Small current level boundaries. These levels may be refined using selective joint pressure testing or other investigation methods.

Table 6-5: Pipe Sections Electro-Scanned Each Day

Date/Time	Start Manhole	End Manhole	Length (ft)
August 14, 2006 / 9:30 AM	MH-114 Grant	MH-111 Grant	455
	MH-115 Florida	MH-114 Grant	
	CO-206 Grant	PS	
	MH-113 Monte Vista	MH-112	
	MH-112	MH-111 Grant	
	MH-063 Hicks	MH-062 Porter	393
	MH-062 Porter	MH-061 Miller	250
	MH-061 Miller	MH-060 Wesley	440
August 14, 2006 / 3:40 PM	**Total for Day**		**1,538**
5 hours, 10 minutes			298
August 15, 2006 / 9:00 AM	MH-055 Third	MH-054 Third	438
	MH-054 Third	MH-053 Third	280
	MH-053 Third	MH-052 Third	438
	MH-056 Lane	MH-055 Third	533
August 15, 2006 / 3:40 PM	**Total for Day**		**1,689**
5 hours, 40 minutes			298
August 16, 2006 / 9:00 AM	MH-052 Third	MH-051 Third	436
	MH-051 Third	MH-050 Third	417
	MH-050 Third	MH-049 Third	353
	MH-049	MH-048	
	MH-048 Third	MH-398 Loan Oaks	186
	MH-394 Loan Oaks	MH-047 Loan Oaks	355
	MH-047 Loan Oaks	MH-046 Roths	320
August 16, 2006 / 3:20 PM	**Total for Day**		**2,067**
5 hours, 20 minutes			388
August 17, 2006 / 9:00 AM	MH-046 Roths	MH-044 Meat Packers	372
	MH-044 Meat Packers	MH-043 High School	385
	MH-043 High School	MH-041 Schlador	339
	MH-041 Schlador	MH-040 JAMES	572
	MH-060 Wesley	MH-059 Water	250
August 17, 2006 / 2:50 PM	**Total for Day**		**1,918**
4 hours, 50 minutes			397
August 18, 2006			
21 hours, 0 minutes	**PROJECT TOTAL**		**7,212**

Anomalies that occur at regular intervals are usually caused by joint defects. To assist with the identification of these joint anomalies, the analysis program can be used by the operator to plot “+”marks on the electro-scan at a regular interval. The analysis program can then select anomalies that occur at the “+” marks and plot a “◇” over the “+”. These anomalies are considered to be associated with a joint defect (See Appendix D). Other anomalies are usually due to structural faults or faulty service connections.

Data Discussion

Electro-scan testing (Table 6-6) has shown that most of the pipe sections have defects that are potential leaks; however, analyses of the results show that the number, size, and type of the defects vary considerably between pipe sections.

Table 6-6: Summary of Electro-Scanning Results and Corresponding Weighted Scores (All pipes in this table are VCP)

Pipe Information			Defect Scores							Rehab Priority
Start MH	End MH	Pipe Dia (")	Large	Score	Medium	Score	Small	Score	Total	
041 Schlador	MH-040	18	8	40	13	39	106	106	185	High
394 Lone Oaks	MH-047	15	0	0	5	15	126	126	141	High
51 Third	MH-050	15	0	0	1	3	124	124	127	High
50 Third	MH-049	15	2	10	2	6	107	107	123	High
044 Meat Packers	MH-043	18	3	15	7	21	82	82	118	High
060 Wesley	MH-059	12	0	0	6	18	70	70	88	Medium
043 High School	MH-041	18	0	0	11	33	48	48	81	Medium
054 Third	MH-053	12	0	0	0	0	78	78	78	Medium
114 Grant	MH-111	8	3	15	2	6	55	55	76	Medium
061 Miller	MH-060	8	2	10	2	6	55	55	71	Medium
48 Third	MH-394	15	0	0	4	12	53	53	65	Medium
52 Third	MH-051	15	0	0	0	0	60	60	60	Medium
055 Third	MH-054	12	2	10	2	6	37	37	53	Medium
047 Lone Oaks	MH-046	15	0	0	2	6	42	42	48	Medium
062 Porter	MH-061	8	0	0	0	0	28	28	28	Low
063 Hicks	MH-062	8	1	5	0	0	8	8	13	Low
046 Roths	MH-044	15	0	0	0	0	13	13	13	Low
53 Third	MH-052	12	0	0	1	3	6	6	9	Low
056 Lane	MH-055	12	0	0	0	0	2	2	2	Low

To prioritize the severity of pipe conditions, each anomaly type was given a corresponding weight. Anomalies determined to be large were given a weight of 5; medium anomalies were given a weight of 3; and small anomalies were given a 1. As shown in Table 6-6 the numbers of each anomaly were multiplied by the corresponding weight. The scores were then summed to produce a total score.

These totals were sorted to produce a prioritized sewer condition list. High priority was given to segments that fell between weighted scores of 185 to 118; medium priority was given to segments that fell between 88 and 48, and low priority was given to segments that fell between weighted scores of 28 and 2.

Chapter 7 - Existing WWTP and Discharge Facilities

Introduction

This chapter describes the treatment systems employed at the Silverton facility, reviews the plant's record of performance, and summarizes the capabilities, limitations, and condition of major treatment facilities.

The City of Silverton owns and operates a municipal wastewater treatment plant, which is located at 400 Schemmel Lane, with an outfall on the north bank of Silver Creek at River Mile 2.45. Wastewater is primarily comprised of domestic sewage, with 9.1 percent attributed to industrial sources. The facility consists of headworks, primary clarification, secondary treatment and settling, ultraviolet (UV) disinfection, and post treatment aeration. The following design parameters for the treatment facility are based on a 2015 design year:

Average Dry Weather Flow:	2.5 MGD
Maximum Month Wet Weather Flow:	6.6 MGD
Peak Hour Capacity:	12.0 MGD
Design Biochemical Oxygen Demand Loading:	7,900 lb/day

Expansion History

The timeline presented in Figure 7-1 summarizes the history of major plant modifications and upgrades. The plant was originally constructed in 1962 as a trickling filter plant and expanded to a trickling filter/solids contact facility in 1985. The expansion and associated collection system improvements were completed under the EPA Construction Grants program. Failure to meet design performance criteria, however, led the DEQ to issue a Stipulation and Final Order (SFO) requiring the City to bring the discharge into compliance with all water quality standards. To address this need, the City completed a Facilities Plan in 1995, and then constructed major modifications and improvements to bring the facility to its current level of performance. It currently provides nitrogen removal and Class B biosolids, in addition to secondary treatment. New facilities were brought on-line in 1999, which included a new headworks structure, modifications to the existing rectangular primary clarifiers, new activated sludge basins, construction of one new secondary clarifier and modifications to an existing secondary clarifier, addition of UV disinfection, post treatment aeration, and a new dissolved air flotation thickener. The improvements also included a new surge basin for diversion of primary effluent during high flow events.

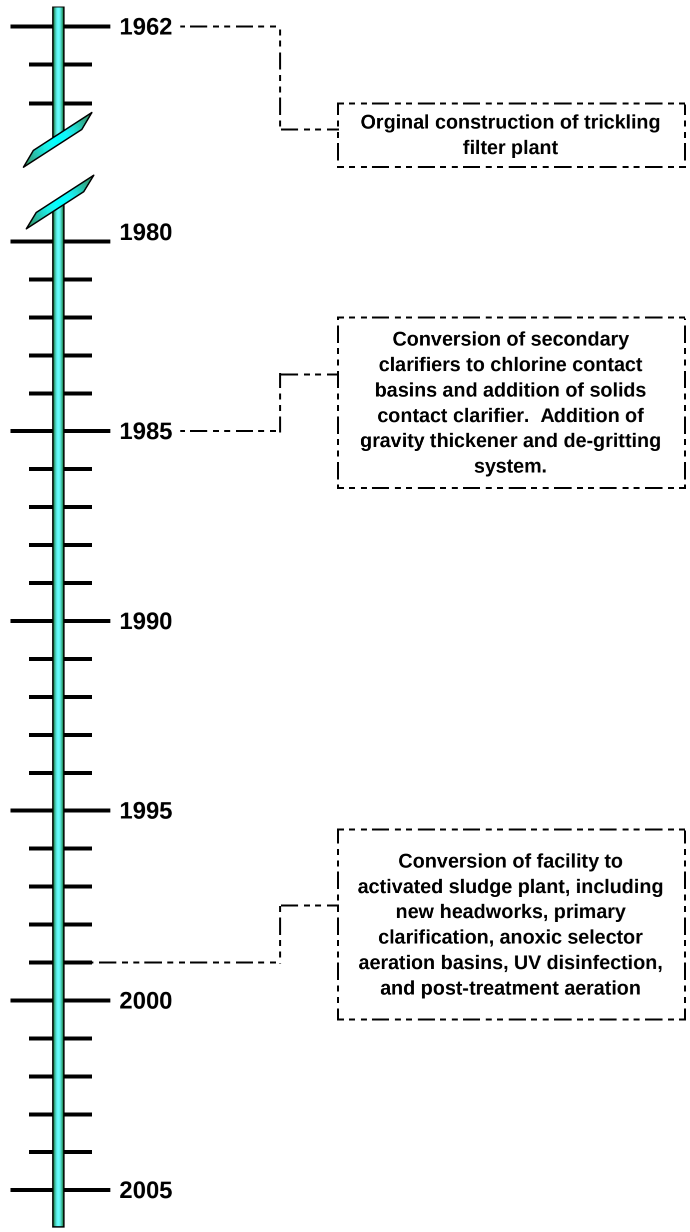


Figure 7-1: Silverton WWTP Facility Construction History (need to identify original construction)

Current Treatment Scheme

Figure 7-2 depicts the process schematic and summarizes major unit processes for Silverton WWTP. Wastewater enters the headworks via a 30-inch ductile iron pipe, which consists of a mechanical bar screen and two comminutors. Following influent screening, the primary sedimentation is provided in two rectangular primary clarifiers.

Primary effluent is equally distributed between two high-rate activated sludge aeration basins. The two carousel-shaped basins operate in alternating air on/off mode for nitrogen removal. They also have a small aerated mixing cell upstream of the carousel basin and a post aeration basin for ammonia polishing. The aeration basin was designed as a high rate activated sludge system, but is currently operated in an extended aeration mode to minimize the waste-activated sludge yield.

Solids in the aeration basins effluent are retained in two circular secondary clarifiers. There RAS is pumped to the aerated mixing cell and WAS is pumped to the DAFT. Secondary effluent is discharged after UV disinfection to either Silver Creek or the Oregon Garden. During summer, the majority of effluent is routed to the Oregon Garden, where it receives further treatment and polishing in a series of three constructed wetlands. The Oregon Garden discharge is discussed in greater detail later in this chapter.

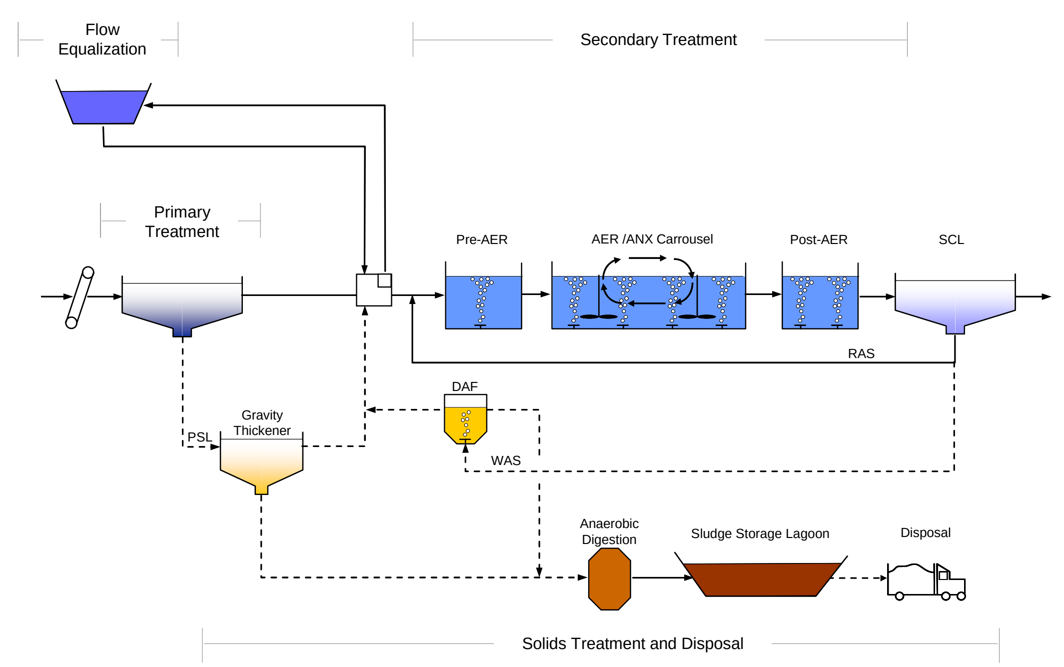


Figure 7-2: Unit Process Flow Schematic

Primary sludge from the primary clarifiers is sent to a cyclone grit removal process prior to gravity thickening. Thickened primary sludge and thickened waste activated sludge are fed to anaerobic digesters. Generated digester gas is used to fuel the digester heating system. Digested solids are stored in either of two lagoons or an out-of-service trickling filter, which are decanted occasionally to maximize solids holding capacity. The decant is returned to the head of the plant. The biosolids are removed on an annual basis (typically in August) for beneficial land application.

Primary effluent flows exceeding the secondary treatment capacity of 7.0 MGD are diverted to a 4.0 mg equalization basin. Flow in excess of the equalization basin storage capacity bypasses secondary treatment and is blended with secondary effluent, disinfected, then discharged.

Current Effluent Disposal Scheme

Treated wastewater not discharged to Silver Creek is pumped through a 16 inch pipe to a series of constructed wetlands at the Oregon Garden site. The maximum pumping rate to the Oregon Gardens is 600 gallons per minute (gpm). Once treated, wastewater enters the first wetland It is considered waters of the state and is no longer regulated as wastewater. Historical effluent flows to the Oregon Garden are shown in Figure 7-3.

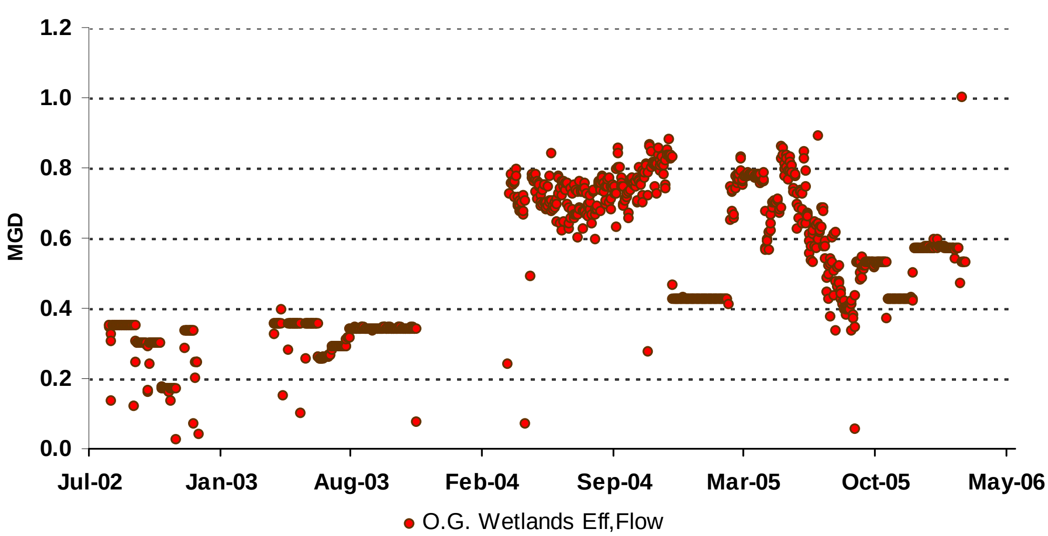


Figure 7-3: Historical Effluent Flow to the Oregon Garden

Historical Plant Performance

Liquid Process

Liquid treatment has performed well since commissioning of the new activated sludge facility. During the first few years, only one of the two trains was being used because flows and loads were still very low. From a flows and loads perspective, one train would be sufficient today; however, there are capacity limitations in the anaerobic digester and sludge storage. The full volume of the two aeration trains is used in extended aeration mode to minimize the sludge yield. Such a shift from a high rate low solids retention time (SRT) to a low rate high SRT process can reduce the yield by over 50 percent, making this an effective tool for operators to mitigate the solids processing bottleneck until new facilities have been constructed. The plant has had no effluent permit violations since startup.

Due to this significant change in operation strategy, the last two years are not representative of the facility's performance under its design operation parameters. This is true especially for secondary clarifier solids retention. Because of the high SRT, the mixed liquor concentrations are also higher (ranging between 3000 mg/L and 5000 mg/L in 2005). This created solids loading in excess of 25 lb/sf/d. Figure 7-4 shows the relationship of secondary clarifier loading and effluent TSS, providing evidence that even under very high solids loading, good performance was maintained.

It should be noted that during peak flow events, flow in excess of 7 MGD is being diverted to the flow equalization basin. Once this storage capacity is exhausted, the flow bypasses secondary treatment to be blended with secondary effluent, disinfected, and discharged to Silver Creek.

In addition, the secondary clarifier was designed for 25 lb/sf/d surface loading for maximum month wet weather flows. The peak flow event, in combination with the high MLSS, pushed the clarifier to its design load. Figure 7-4 shows how well the clarifier performed under these conditions, indicating potential capacity beyond its current design load.

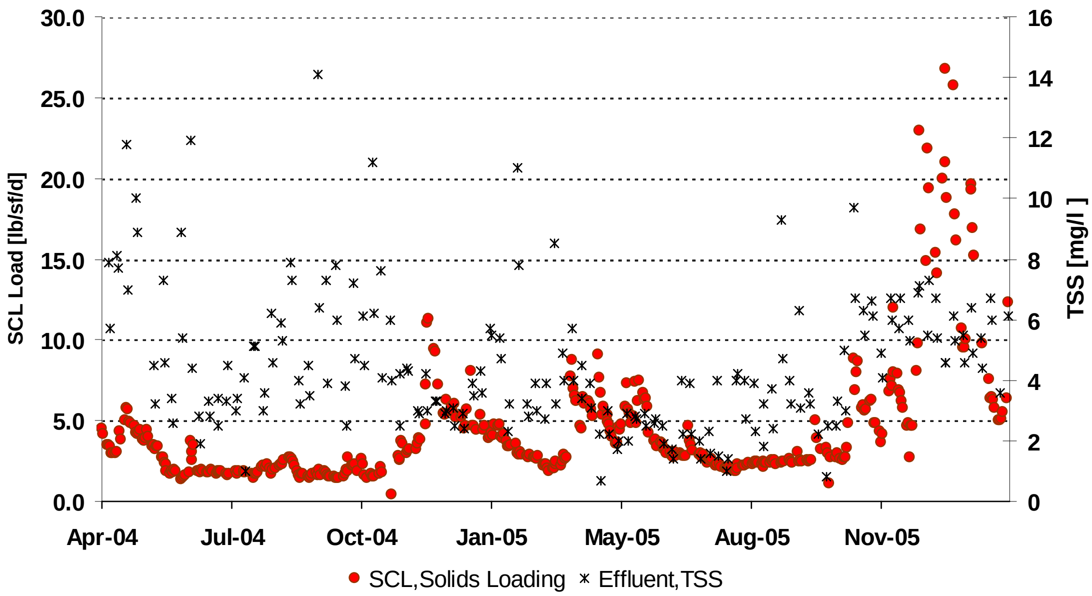


Figure 7-4: Silverton WWTP Secondary Effluent Loading vs. Secondary Effluent TSS

One aspect that aided the clarifier performance during the peak load event in 2005 was the low sludge volume index (SVI). Given the SVI history, it could be considered a fortunate coincidence that the peak event and solids retention friendly low SVI occurred at the same time.

This has significant implications for the capacity of the secondary treatment system. As aforementioned, during the peak event the clarifiers operated very well and showed potential for rating to a higher capacity; however, this performance depends on reasonably low SVI values. The latter have been rare at Silverton WWTP. The data review has delivered few clues for the SVI inconsistencies (Figure 7-5). Theoretically, the air on/off operation mode should create excellent anoxic selector conditions during the denitrification cycle. No relationship is shown, however, between SVI and effluent nitrate (which would indicate a connection between denitrification, anoxic selector effectiveness, and SVI). Conversely, effluent total phosphorus showed a good correlation with SVI.

A possible explanation is that, instead of the anoxic selector effect, an anaerobic selector effect appears to be more successful in filament control. Anaerobic conditions could occur at the end of an air-off cycle when nitrites have been fully denitrified. Under these conditions, the phosphorus-accumulating bacteria population establish and provide sBOD removal under anaerobic conditions, which is one critical measurement of selector effectiveness. Low effluent phosphorus concentrations are an indirect measure of anaerobic selector activity. At the Silverton WWTP it appears that with low effluent phosphorus (thus, good anaerobic selector effectiveness), low SVI values are the result.

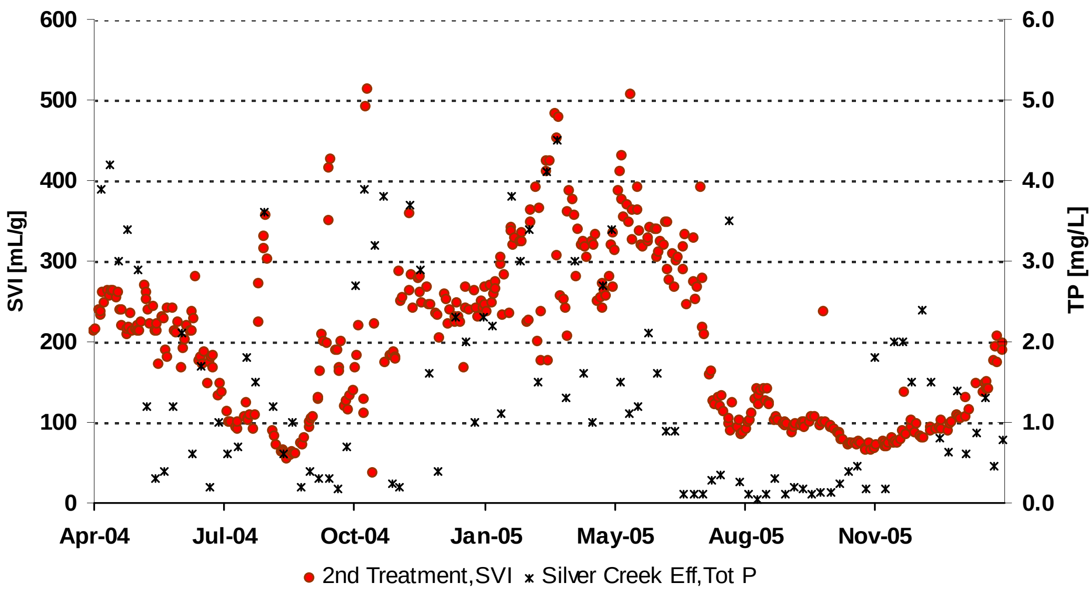


Figure 7-5: Silverton WWTP Relationship of SVI and Effluent Total Phosphorus

Figure 7-6 shows the relationship between effluent alkalinity and pH. It becomes apparent from this plot that the alkalinity supplement feed control could be improved. Ideally, the effluent alkalinity should be more constant and bottom-out at a target value (e.g., 75 mg/L). Instead, the alkalinity data shows a much larger spread. Because influent alkalinity is typically consistent, the large spread of effluent alkalinities are likely due to supplement feed control insufficiencies. The most common result of such control is overdosing, which not only increases the chemical cost, but increases the amount of chemical sludge generated.

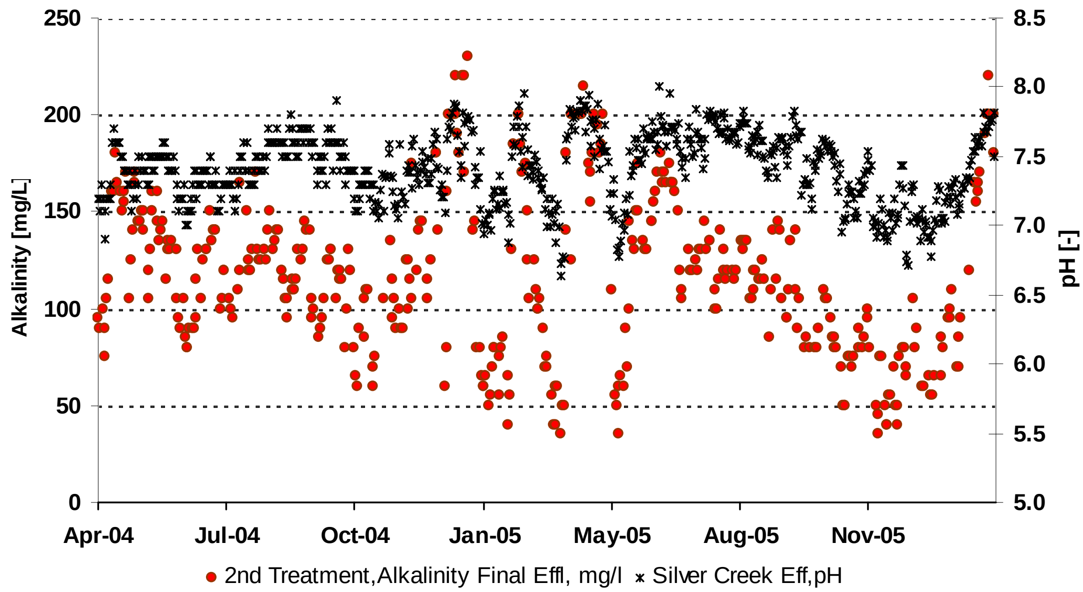


Figure 7-6: Silverton WWTP Relationship Between Alkalinity and Effluent pH

Figure 7-7 through Figure 7-13 show key effluent parameter plots with indications of their permit limits.

Solids Process

Process data on internal solids handling at the plant is limited. Interviews with plant staff were conducted to determine the plant's performance of solids processing. Primary sludge is approximately 0.25 to 0.5 percent, which is appropriate for Silverton's sludge grit removal process. Gravity thickening of primary sludge results in TPS solids concentration between 3 and 4 percent. Similarly, dissolved air flotation thickening of WAS results in a thickened WAS solids concentration between 3 and 4 percent. The anaerobic digesters achieve a volatile solids destruction efficiency of approximately 60 percent. After anaerobic digestion, the solids concentration is approximately 1.5 to 2 percent. Digested solids are stored in one of two lagoons or in an abandoned trickling filter. As solids settle in the lagoons, the lagoons are decanted and the solids are concentrated to approximately three percent prior to removal and land application during summer.

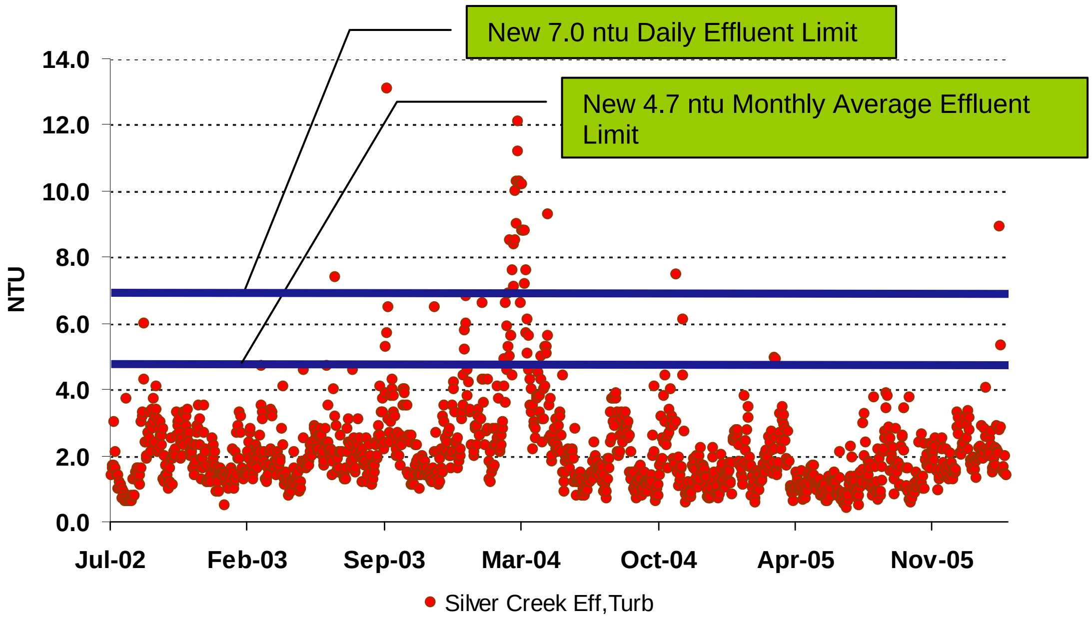


Figure 7-7: Historical Effluent Turbidity

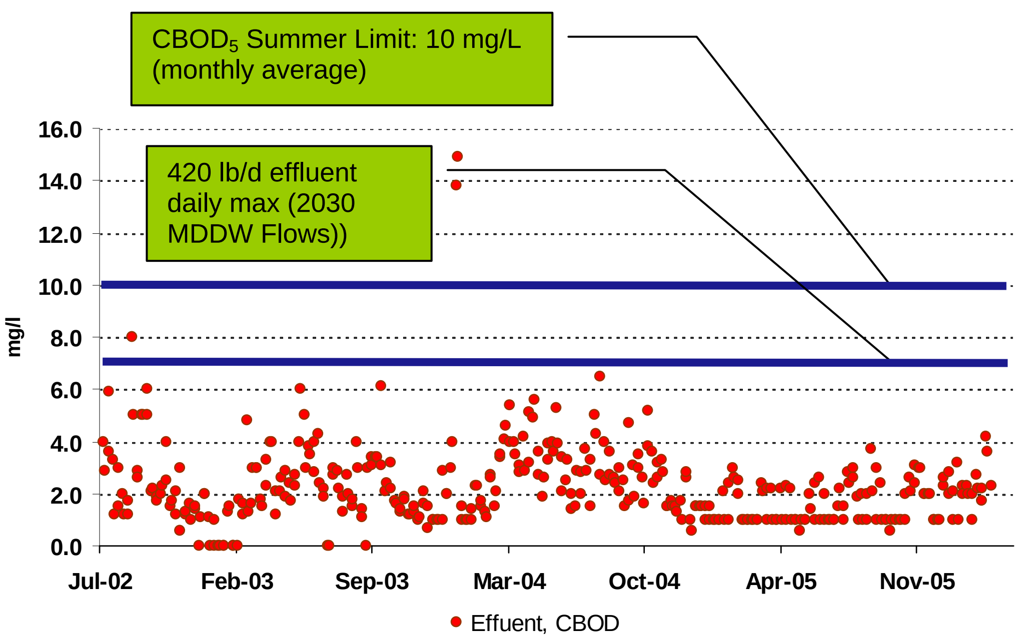


Figure 7-8: Historical Effluent CBOD Concentrations

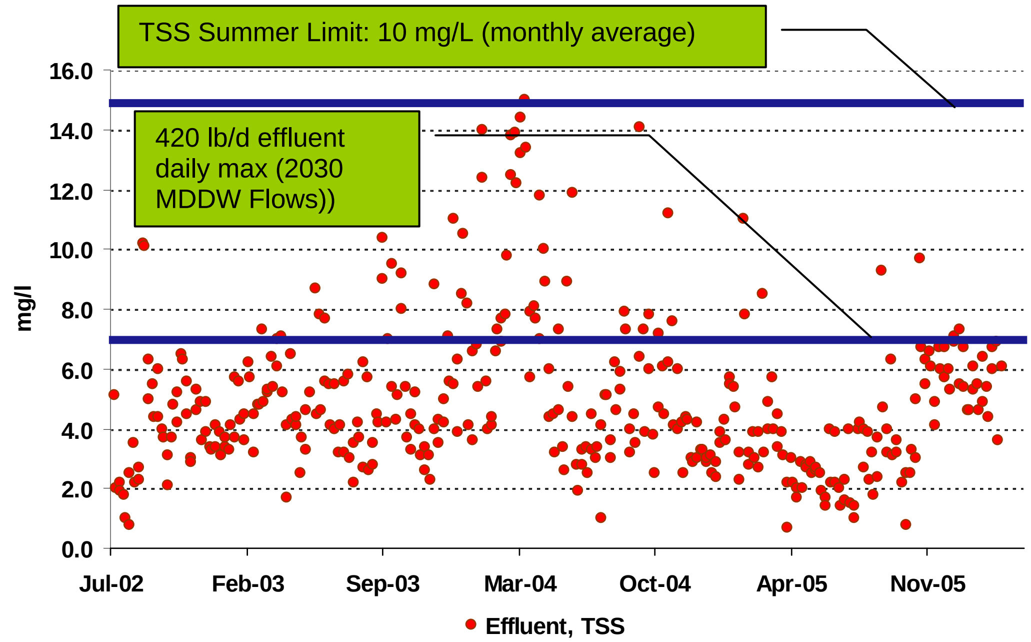


Figure 7-9: Historical Effluent TSS Concentrations

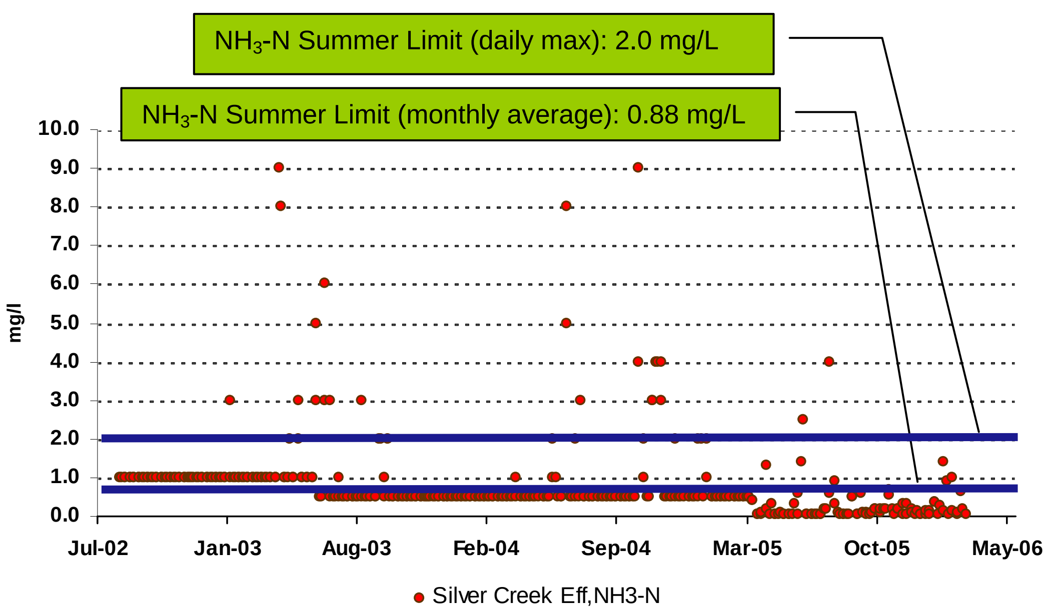


Figure 7-10: Historical Effluent Ammonia Concentrations

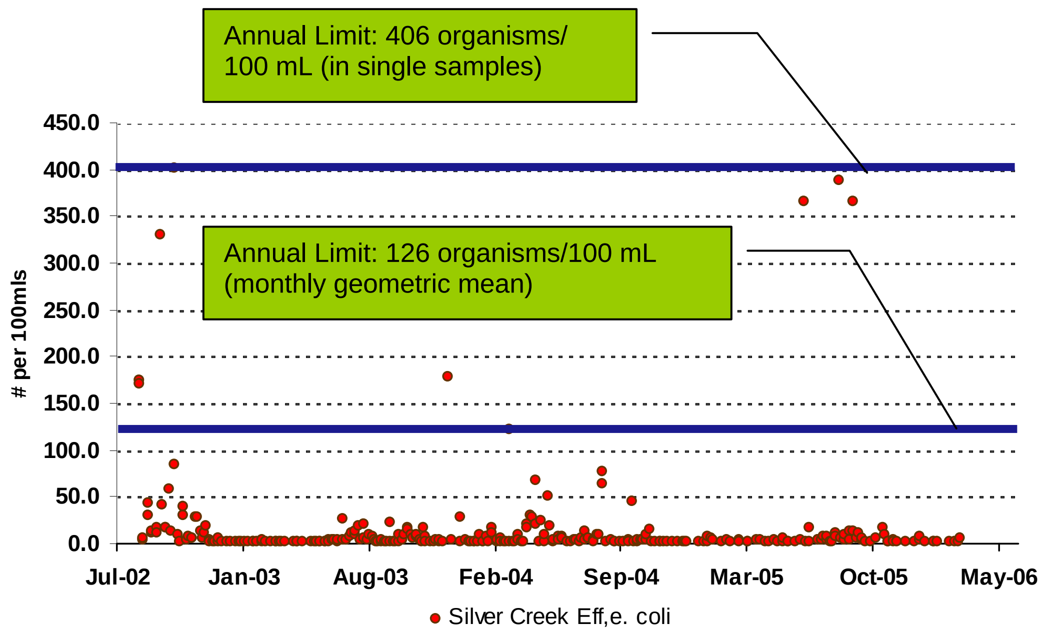


Figure 7-11: Historical Effluent *E. coli*

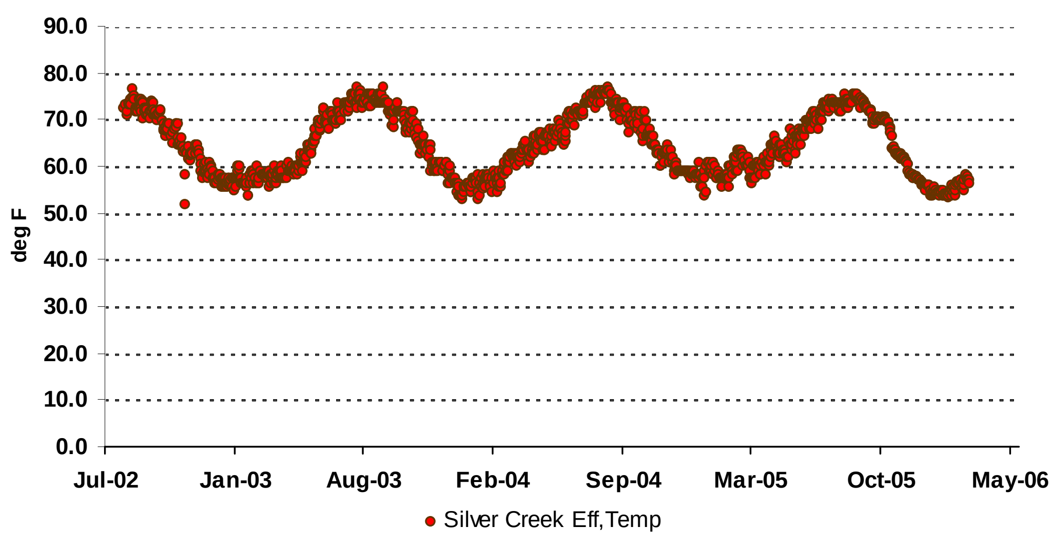


Figure 7-12: Historical Effluent Temperature

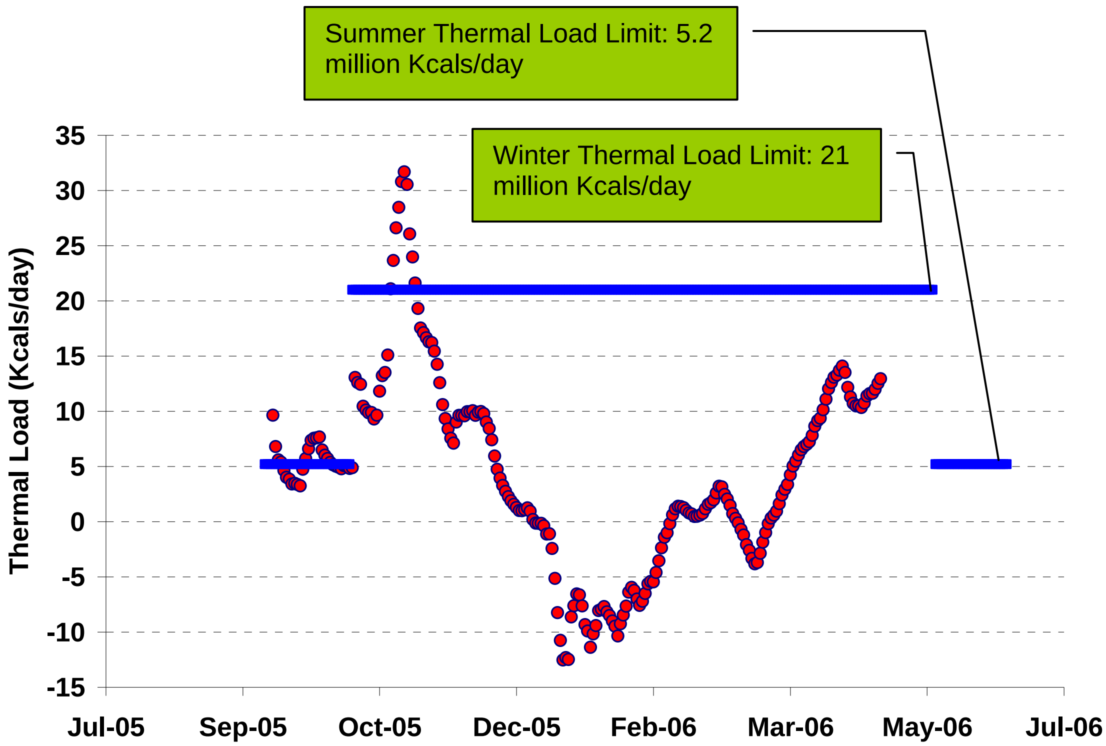


Figure 7-13: Effluent Thermal Load

Unit Process Assessment – Methodology

The next sections of this chapter review the functions and capabilities of individual unit processes and identify key operational, maintenance, or mechanical issues related to plant processes. The discussion is divided into three major process areas: liquids treatment, solids treatment, and support facilities. The findings were developed through meetings with the City, field inspections, review of performance data, and mass balance modeling.

Unit Process Assessment – Liquid Treatment

Headworks

Description

Raw wastewater is conveyed to the plant by gravity through a 30 inch pipeline. Wastewater flows through an influent junction box and a Parshall flume prior to entering the headworks. The headworks facility at the Silverton WWTP consists of a single 0.5 inch mechanical bar screen (Figure 7-14). The screenings deposited automatically into a roll-away dumpster that is residually exchanged and its contents are hauled to a landfill.

Figure 7-14: Silverton Headworks Single Bar Screen

Capacity and Redundancy

Presently only a single screen is available; however, a bypass channel is available if the primary screening channel requires service. The existing mechanical bar screen has a hydraulic capacity of 15 MGD (Table 7-1) which exceed the design maximum day flow for 2030. Thus the existing influent screen has sufficient capacity through the end of this planning horizon.

Table 7-1: Information Summary of Screening Facility and Equipment

Parameter	Value
Maximum Hydraulic Capacity (Peak)	15 MGD
Screen Type	Mechanical bar screen
Bar Spacing	0.5 inch

Operational Issues

The influent screening facility is not contained, has no odor control, and is only a few feet from the nearest residential building. Odor complaints are inevitable and headworks enclosure and odor control should be included in future capital improvement planning.

Primary Treatment

Description

Primary treatment at the Silverton WWTP is currently provided in two rectangular clarifiers (Figure 7-15) that total 2400 square feet. The clarifiers were constructed with in 1984 and are in good condition. From time to time plant staff has to replace various scraper mechanism parts but based on a typical life cycle for this type of equipment, replacement should not be necessary for another 10 – 20 years. The structural concrete appears to be in good condition and does not require replacement within the planning horizon of this facility plan.

Primary sludge is pumped from the primary clarifiers to the primary sludge gravity thickener. Before reaching the gravity thickener, the primary sludge is degritted using a single cyclone and classifier. The removed grit is collected in a roll-away dumpster and hauled to a landfill.

Figure 7-15: Silverton WWTP Primary Clarifiers

Capacity and Redundancy

The capacity of the primary clarifiers is limited by hydraulic loading. The current peak design flow is 12 MGD, which results in a peak hydraulic loading of 5000 gal/sf/day. Under design maximum month hydraulic loading of 6.6 MGD, hydraulic loading with two clarifiers in service is 2750 gal/sf/d. Under design average dry weather flow conditions (2.5 MGD), the hydraulic loading is 1040 gal/sf/d with both clarifiers in service. Dry weather flow in 2006 was approximately 0.75 MGD. Table 7-2 summarizes the primary clarifier design parameters and other related information. The existing primary clarifier capacity is sufficient for the 2030 design flows.

Table 7-2: Information Summary of Primary Clarifiers

Parameter	Value
Number of Clarifiers	2
Surface of Each	1200 sf
SWD	10 ft
Maximum Hydraulic Capacity (Peak)	12 MGD
Peak Hydraulic Loading (2 clarifiers)	5000 gal/sf/d
Max. Month Hydraulic Loading (2 clarifiers)	2750 gal/sf/d
Design Average Loading	1040 gal/sf/d
2006 Average Hydraulic Loading (2 clarifiers)	310 gal/d/sf
Year Installed	1984

Operational Issues

The influent pipe to the primaries has multiple 90° elbows which have a tendency to build up grease, which is difficult to remove.

Primary sludge is pumped from the primary clarifiers by a single recessed impeller pump. This pump is located next to the administration building in a wood frame shelter with aluminum siding, as shown in Figure 7-16. The suction line is too long (approximately 80 feet) and the elevation of the pump is too high, potentially causing plugging and cavitation problems. This pump and shelter should be demolished and replaced with a new primary sludge pump station with multiple pumps and should be located closer to the primary clarifiers in an underground vault.

Currently, primary sludge is degritted using a single cyclone and classifier, which is shown in Figure 7-17. The equipment is not enclosed and is located adjacent to the anaerobic digesters and the gravity thickener. Classified grit is collected in a haul-off container and periodically removed to a local landfill for disposal. The cyclone was replaced in 1998, but the classifier is corroded, beyond its service life, and should be replaced. Consideration should be given to enclosing the process for odor control purposes.

The primaries are currently not covered and are, therefore, a source of odor. Given the close proximity of residents, installation of covers and foul air treatment should be considered for the future.

Figure 7-16: Primary Sludge Pump Shelter (Tan Walls with Blue Roof) (Left) and Primary Sludge Pump (Right)

Figure 7-17: Primary Sludge Degritting Equipment

Secondary Treatment

Description

The secondary treatment facility at Silverton WWTP (Figure 7-18 and Figure 7-19) is a high rate activated sludge plant consisting of two equal trains. Each train has a pre-aerating zone for mixing of RAS and primary effluent, a carousel type aeration basin with only partial diffuser coverage, a post aeration tank, and the secondary clarifier. Both trains share a common blower building, a RAS/WAS pump station, and a lime feed system.

The secondary treatment system was designed for nitrogen removal, utilizing both simultaneous N/DN and alternating air-off cycles between both trains. Simultaneous nitrification and denitrification is accomplished by circulating the basin content around the tank where it passes over the section with diffusers taking up oxygen. As the content travels around the basin, the dissolved oxygen is consumed and ammonia is nitrified. Eventually, the conditions become anoxic and denitrification begins to occur until the content passes through the aeration area again. Operators can adjust the net size of the aerated area by adjusting the target dissolved oxygen (DO) at the control point and the speed at which the content moves around the basin. In addition operators turn the air off for several hours in a 4.5 hour on/1.5 hour off cycle to further improve nitrogen removal. During the air-off cycle, the post aeration basin nitrifies residual ammonia to maintain low effluent ammonia concentrations at all times. The air on/off cycle alternates between the two treatment trains.

The blower building (Figure 7-20) contains four multi stage centrifugal blowers capable of providing a maximum of 3873 scfm. The aeration system uses membrane disc fine bubble diffusers.

The return activated sludge is pumped back to the pre-aeration tank from the RAS/WAS pump station (Figure 7-21). Three 1.5 MGD RAS pumps are available and provide a recycle rate under design wet weather maximum month condition of 70% percent with all three pumps running. Piping connections are in place to accommodate a future fourth RAS pump.

The RAS pumps have a common suction and discharge header, which allows single SRT operation only.

WAS pumping is controlled based on pounds wasted per day to maintain the target SRT and/or MLSS. Based on the RAS TSS the WAS flow is adjusted and runs 24/7. The WAS is pumped to the DAFT, where it is then thickened.

For pH maintenance, supplementary alkalinity is added in the form of lime, which is stored dry in a silo next to the aeration basin. The system produces lime slurry, which is fed on a constant rate basis. The dosage rate is determined based on the laboratory results from the effluent alkalinity sample.

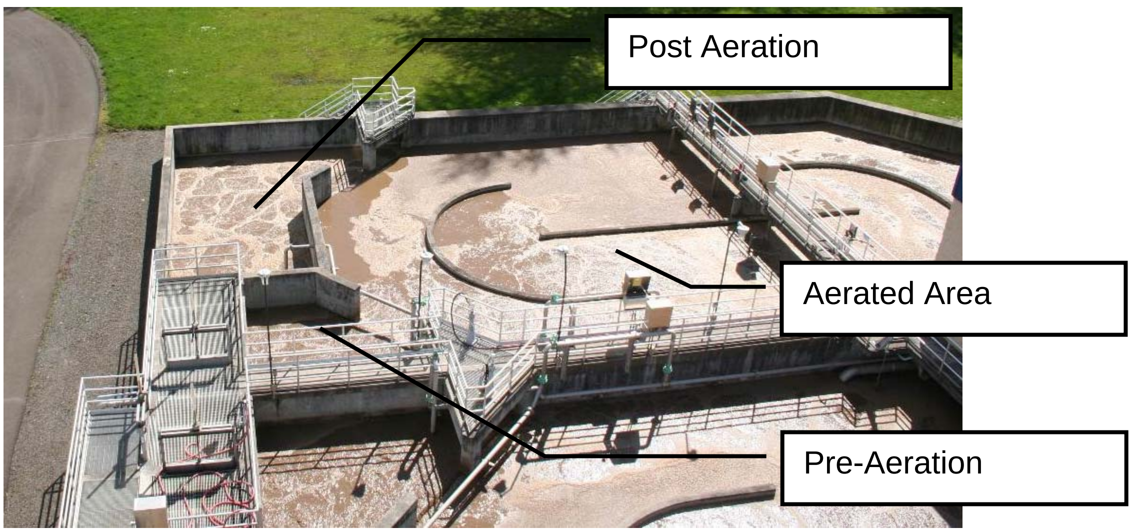


Figure 7-18: Silverton WWTP Aeration Basin

Figure 7-19: Silverton WWTP Secondary Clarifier

Figure 7-20: Silverton WWTP Blower Building

Figure 7-21: Silverton WWTP RAS/WAS Pump Station

Capacity and Redundancy

The secondary treatment system was the focal point of the facility's expansion in 1996. Currently the system operates at 45 percent of its design capacity (Table 7-3). Because of the solids processing bottleneck, operators have changed the mode of operation to extended air, running at a very high SRT to reduce the WAS production. The system was designed conservatively; therefore, without a performance history of an activated sludge plant at Silverton WWTP, rerating the secondary treatment to a higher capacity is possible.

The aeration basins were designed for a mixed liquor concentration of 3000 mg/L. This results in a secondary clarifier loading of 25 lb/sf/d with both clarifiers online under design maximum month conditions (6.6 MGD).

Table 7-3: Information Summary of Secondary Treatment

Parameter	Value
Number of Aeration Trains	2
Number of Secondary Clarifiers	2
Preaeration Volume (Each)	0.023 MG
Carousel Aeration Basin Volume (Each)	0.58 MG
Post-Aeration Volume (Each)	0.043 MG
Total Activated Sludge Volume	1.3 MG
Design HRT (@6.6 MGD MMWWF)	4.2
Design MLSS	3000 mg/L
2005 average MLSS	3000 mg/L*
Secondary Clarifier Surface (Each)	5000 ft
Design SCL Solids Loading (@ 6.6 MGD MMWWF)	25 lb/sf/d
2005 Average SCL Loading	4.4 lb/L/hr**
Design RAS Rate (@ 6.6 MGD MMWWF)	45%
Number of RAS Pumps	2 + 1 standby
Number of Blowers	3 + 1 standby
Total Blower Capacity (Without Standby)	3873 scfm

* Plant currently operated in extended air mode with long SRT to minimize solids production

** with two clarifiers in operation

Operational Issues

The aerating basin was designed as a high rate activated sludge system but is currently operated in an extended aeration mode to minimize the WAS yield. This means that the present plant performance is not representative of the design intent.

The air on/off operation means that during the air off periods at least half the plant flow is only aerated for a very short period of time in the post aeration basin. With not online feedback as to the combined effluent ammonia concentration bleed through of peak load such as from lagoon decanting can occur and may be responsible for the occasional spike in effluent ammonia confrontation.

The secondary system also appears to have problems with high SVI at times exceeding 400 mL/g, which impairs clarifier solids retention performance and limits system capacity.

UV Disinfection

Two 5 MGD medium pressure, high intensity UV systems (Figure 7-22) were installed on one of the existing chlorine contact basins during the last facility expansion. They are located in part of the old chlorine contact tank. After some initial startup problems the system has been working promptly and without major issues. Due to the equalization basin capacity to store peak hours flow the existing UV disinfection capacity is sufficient for 2030 flows.

Figure 7-22: Silverton WWTP UV Disinfection System

Flow Equalization

Description

Silverton WWTP has the option of diverting excess flow during peak storm events to a 4 mg equalization basin. The objective of the equalization basin is to minimize the size of the secondary treatment and at the same time reduce the amount of flow bypassing secondary treatment.

When flows exceed 7 MGD, the excess flow is diverted to the equalization basin downstream of the primary clarifier. Once the basin is full, flow then bypasses secondary treatment, blends with secondary effluent, and is discharged to Silver Creek after disinfection. Flow from the equalization basin is pumped back upstream of the secondary treatment system. The pumps have a capacity of 600 gpm and operate at constant speed. They are controlled by a level sensor though during peak flow events the pump is turned off until while plant flow exceeds the 6 MGD.

Figure 7-23: Silverton WWTP Flow Equalization Basin

Capacity and Redundancy

The equalization basin has a total volume of 4 MG (Table 7-4). There are two submersible return pumps. While one is standby, they can be run together if a higher return flow rate is desired. With one pump running, the maximum flow is 1400 gpm, which increases to 1700 gpm with both pumps running. Under normal operation (one pump) it takes 2 days to empty the equalization basin. E dedicated pump at the effluent pump station provide plant effluent for flow equalization basin washdown.

Table 7-4: Information Summary of the Equalization Basin

Parameter	Value
Volume	4 mg
Depth	11 ft
Area	57,000 st
Number of Return Pumps	2 (1+1 standby)
Return Pump Capacity	1400 gpm

Operational Issues

None known

Effluent Pump Station

The effluent pump station consist of two service pumps that pump effluent to the Oregon gardens, one pump is available for equalization basin wash down and one pump is available to pump plant affluent to the Silver Creek outfall during water levels in the creek. During normal water elevations the effluent flows by gravity to the creek.

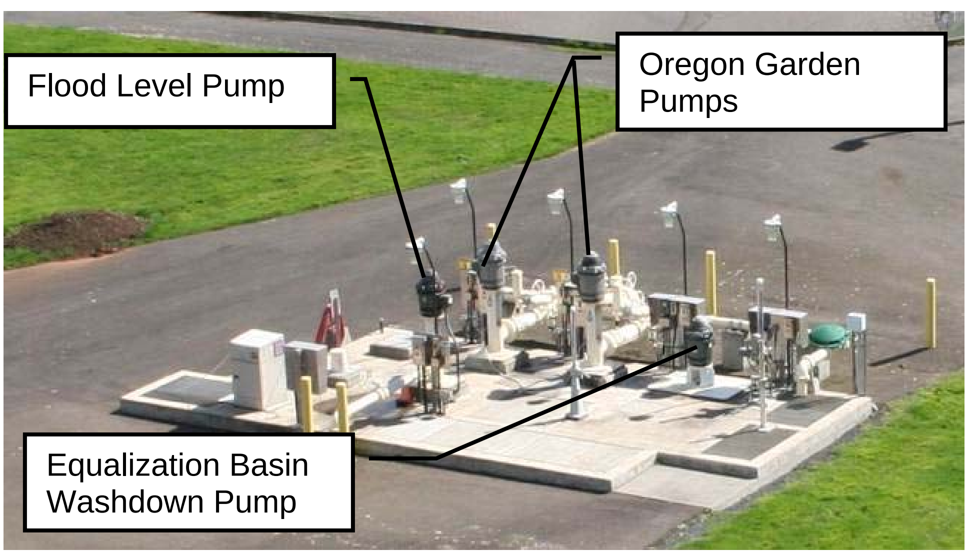


Figure 7-24: Effluent Pump Station at Silverton WWTP

Capacity and Redundancy

The high service pumps have a service capacity of 0.85 MGD with one standby pump (Table 7-5). The flood level pump has a capacity of 10 MGD. Due to the large flow equalization volume effluent peak hour flows can be maintained below 10 MGD. Thus the existing flood level pump has sufficient capacity for 2030 flows. The lack of full redundancy was reviewed but due to the rarity of the event it was deemed to be sufficient.

The equalization basin washdown pump has a capacity of 0.85 MGD and due to its usage nature does not require redundancy or expansion.

Table 7-5: Information Summary of the Equalization Basin

Parameter	Value
High Service Pump Capacity (One Pump Running)	0.85 MGD
Flood Level Pump	10 MGD
Equalization Basin Washdown Pump	0.86 MGD

Unit Process Assessment – Solids Treatment

WAS Thickening

Description

The DAFT (Figure 7-25) receives WAS from the RAS/WAS pump station at approximately 5,000-8,000 mg/L solids concentration depending on the aeration basin mixed liquor concentration and RAS rate. The DAFT is located near the aeration basins and secondary clarifiers.

Figure 7-25: Silverton WWTP Dissolve Air Flotation Thickener

Capacity and Redundancy

The single 20-foot-diameter DAFT, constructed in 1996, thickens WAS to approximately 3 to 4 percent depending on loading and influent solids concentrations. Table 7-6 shows the current design solids and hydraulic loadings to the DAFT. The DAFT currently utilizes approximately 25 percent of its design capacity and is in very good condition; however, there is currently no backup for WAS thickening.

Table 7-6: Information Summary of Waste Activated Sludge Thickening

Parameter	Value
Area	315 sf
SWD	9 ft
Design Solids Loading	24 lb/sf/d
2005 MM Solids Loading	4.7 lb/sf/d
Design Hydraulic Loading	3600 gal/sf/d

Condition and Operational Issues

The DAFT is not covered and can be a source for odor. Covering and connecting it to the foul air system is recommended for the future.

PSL Thickening

Description

The thickener receives degritted sludge at approximately 0.5 percent solids concentration, and is located adjacent to the anaerobic digesters and primary sludge degritting equipment. The single 20-foot-diameter gravity thickener (Figure 7-26), constructed in 1982, thickens primary sludge to approximately 3 to 4 percent, depending on loading and influent solids concentrations.

Figure 7-26: Silverton WWTP Primary Sludge Gravity Thickener

Capacity and Redundancy

Table 7-7 shows the current design solids and hydraulic loadings for the gravity thickener. Assuming a primary sludge concentration of 0.5 percent, the gravity thickener is adequately sized for current and future loadings. The current solids loading of 6.4 lb/sf/d represents 26 percent of its design loading; however, there is currently no backup for primary sludge thickening. The gravity thickener skimmer/sludge collector drive has been recently replaced, and the structure and weir are in adequate condition.

Table 7-7: Information Summary of Primary Sludge Thickening

Parameter	Value
Area	315 sft
SWD	11 ft
Solids Loading Rate (@ 6.6 MGD MMWWF)	24 lb/sf/d
2005 Solids Loading	6.4 lb/ft/d
Hydraulic Loading Rate (@ 6.6 MGD MMWWF)	700 gal/sf/d

Condition and Operational Issues

Being a single gravity thickener, no backup alternatives exist if the thickener has to be taken out of service for maintenance.

The thickener is not covered and can be a major odor source. Adding a cover and connection to foul air treatment is recommended for the future.

The thickened sludge only reaches 3 to 4 percent. Primary sludge thickeners are capable of thickening to 7 percent TSS and more.

Anaerobic Digestion

Description

Two 30-foot-diameter anaerobic digesters (Figure 7-27) stabilize thickened primary sludge and TWAS to Class B biosolids standards. The digesters are gas mixed and have floating steel covers for gas storage. Digester gas is utilized for digester heating. Excess gas is burned off by the digester gas flare.

Figure 7-27: Silverton WWTP Anaerobic Digesters

Capacity and Redundancy

Table 7-8 shows the estimated detention times at current flows and loads. The table shows the digesters are overloaded and provide no redundancy. Despite operating beyond capacity, the volatile solids destruction in the digesters average approximately 60 percent, which is very good performance. The existing digesters floating steel covers are in fair shape. According to plant staff, the covers travel up and down with no difficulty. Currently, temporary piping is used for recirculation as the original piping had a long vertical run and the recirculation pumps had air binding problems. The existing digester gas flare and gas piping is beyond its service life (installed in 1982) and should be replaced. The entire anaerobic digestion facility has zero redundancy.

Table 7-8: Information Summary of Anaerobic Digestion

Parameter	Value
Number of Digesters	2
Diameter	30 ft
SWD	approx. 15.5 ft
Volume (Each)	82,000 gallons
Design HRT (2 Digesters)	20 days
Current HRT (2 Digesters)	13.7 days
Design Solids Loading (2 Digesters)	16 lb/cf/d
2005 Solids Loading (2 Digesters)	11 lb/cf/d

Condition and Operational Issues

The existing anaerobic digesters have experienced foaming problems in the past. Foaming is typically caused by filamentous bacteria from the secondary treatment system and is difficult to control for plants that nitrify due to low ammonia limits.

Because the digesters always operate at maximum capacity, maintenance and repair is difficult.

Temporary piping is currently being used for recirculation as the original piping had a long vertical run and the recirculation pumps had air binding problems.

Solids Dewatering, Storage, and Disposal

Description

The existing plant does not have a solids dewatering process other than the solids lagoons, which do not have adequate storage for seasonal limitations on biosolids land application. To increase the plant's solids storage capacity, one of the abandoned trickling filter structures is currently used. All storage volumes are periodically decanted to further maximize their storage capacity. The decant is returned to the plant influent.

Figure 7-28: Silverton WWTP Sludge Storage Lagoons

Capacity and Redundancy

The two original lagoons have a combined capacity of 640,000 gallons (Figure 7-28, Table 7-9). This provides only 80 days of storage at average 2005 conditions. An abandoned trickling filter (rocks removed) is also used as a lagoon. The trickling filter provides an additional 44 days of sludge storage. Dewatering will provide the greatest flexibility for on-site solids storage and is recommended due to the currently overloaded and under capacity solids storage lagoons. Several proven solids dewatering technologies are available and are presented below.

Table 7-9: Information Summary of Biosolids Storage

Parameter	Value
Lagoon Volume (Each)	0.32 MG
Trickling Filter Storage Volume	0.35 MG*
Total Storage Capacity (only Lagoons)	80 days**
Total Storage Capacity (Lagoons and Trickling Filter)	124 days**

*Trickling filter volume based on 100 ft diameter and 6 ft depth.

**Storage capacity does not account for decanting. Accounting for decant (assuming 3% final solids), the total storage volume is 142 days (220 days with trickling filter storage volume) at average 2005.

Condition and Operational Issues

The solids storage lagoons are a significant odor source. In order to maximize their capacity operators use portable pumps to periodically decant the lagoons. This generates very high recycle ammonia loads.

Major System Deficiencies

The major deficiencies identified in the Silverton WWTP include: biosolids management, primary sludge pumping, and primary sludge grit removal.

Biosolids Management

Silverton faces imminent challenges in the area of biosolids storage and land application. Sludge storage is near capacity, requiring the addition of on-site biosolids storage or modifications to the biosolids treatment scheme.

The biosolids land application program is based on having a willing farmer (or farmers) accept the biosolids; the City does not own the property on which biosolids are applied, nor do they have formal agreements with the land owners, ensuring sites will be available for future land application. Currently, only one customer receives Silverton's biosolids, and application can take place only during an approximate two week period.

Chapter 8 - Collection System Master Planning

Introduction

The purpose of the conveyance system analysis is to characterize the system hydr0061ulics and build a baseline for development of a CIP program. This chapter describes the background, methods and results of the analysis. Results of the analysis include a description of the existing system hydraulics along with general description of the hydraulics for the planning year 2030 and ultimate build-out conditions.

Conveyance System Model

Model Selection

MikeURBAN from DHI was used to simulate the hydraulics of the conveyance system. MikeURBAN is an enhanced version of the U.S. Environmental Protection Agency's Storm Water Management Model that incorporates hydraulic analysis within a GIS environment. This model was selected as the analysis tool because of its ability to model complex hydraulic systems with reliable results and its ability to present those results graphically.

Model Development

The system model generates inflow hydrographs and analyzes the major conveyance components. The conveyance components include eight pump stations, one diversion, and the trunk and interceptor gravity sewers. The pipe and manhole data used for model construction was extracted from a GIS database and used in the MikeURBAN model. After the physical representation of the system was constructed in the model, the dry-weather sanitary flows, inflow and infiltration and major industrial flows (from Bruce Pac, Quest, and future industrial developments) were imported into the model.

Wastewater Flow Generation

Using projected population, land use information and GIS tools, a population factor (people per acre) was estimated for the City of Silverton's residential and commercial land uses. The population flow factor was determined for the existing population, the 2030 projected population as determined in Chapter 4, and the ultimate build-out population. The ultimate build-out population was determined using information in the City of Silverton's Comprehensive Plan dated August 2002 and is estimated to be 20,488.

Table 8-1 shows the calculated population factors used for the three flow conditions evaluated in the model.

A flow factor of 90 gallons per capita per day (gpcd) was assumed based on the analysis described in Chapter 4. A flow contributing area was defined at each loading manhole using GIS tools, and the flow factor was applied to this area to generate the average day flow loads. Table 8-1 also shows the total average flow loaded into the model.

Table 8-1: Modeled Wastewater Flow

Flow Condition	Projected Population	Calculated Population Factor (People per Acre)	Average Daily Wastewater Flow (mgd)*
Current Condition (2006)	8,235	4.1	0.74
2030 Condition	14,000	7.0	1.26
Ultimate Build-Out	20,488	10.2	1.84

* Only residential flow included in total.

The diurnal pattern for Silverton was developed from the flow monitoring data collected by GEOtivity flow monitors that were in place from the beginning of April through May 2006. A weekday and weekend diurnal pattern was developed and these patterns were applied to the average day flow determined as described above. The diurnal patterns are shown in Figure 6-2. These diurnal patterns are similar to those for other similar communities in the Willamette Valley.

For Silverton the highest weekday flow peak occurs in the morning at about 7:00 am. Another, smaller peak occurs in the evening at about 8:00 pm. Note that the weekend diurnal pattern shows that the morning peak is smaller and occurs later, at about 10:00 am.

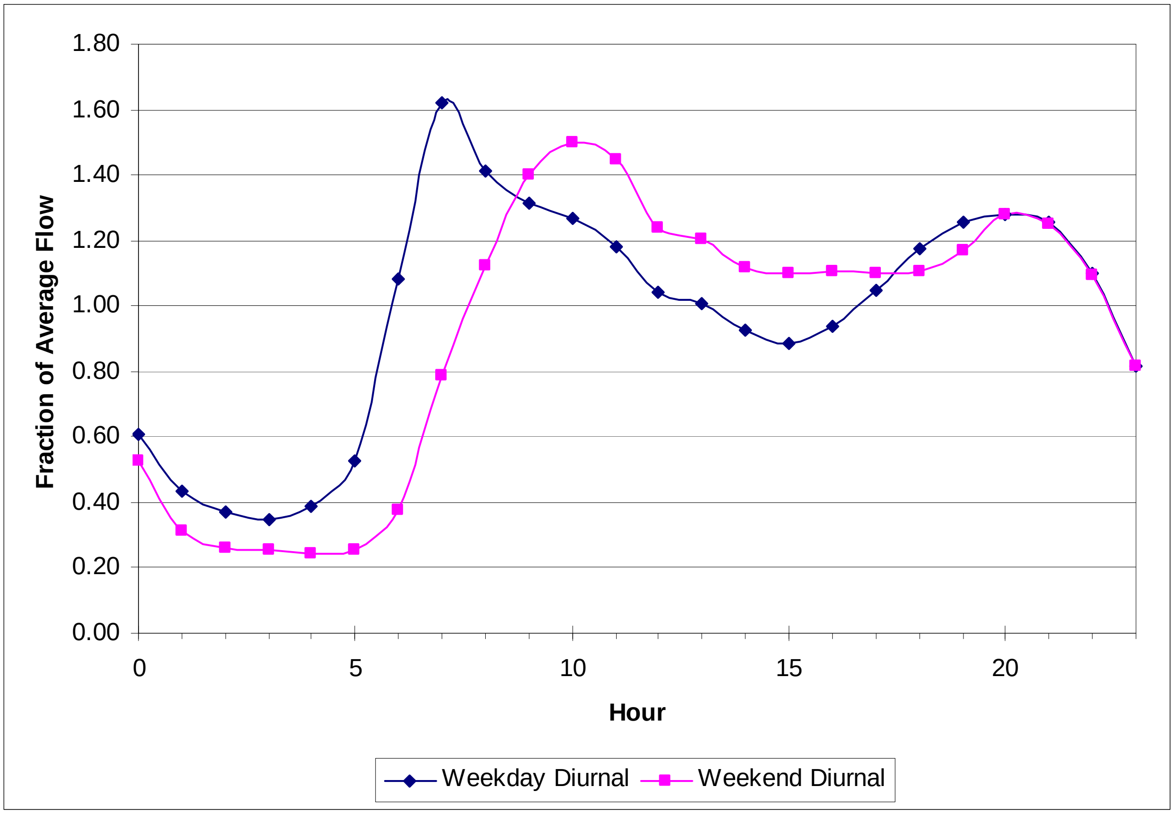


Figure 8-1: Diurnal Pattern

Wet weather flows were simulated using an inflow and infiltration (I/I) triangular hydrograph generated based on sewer flow monitoring and precipitation data. The I/I hydrograph was created using the RTK Hydrograph Method. This hydrograph is based on three parameters:

- R: the fraction of rainfall over the watershed that enters the sanitary sewer system
- T: the time to peak in hours
- K: the ratio of the time to recession to the time to peak

These values are typically determined for three generic storm events – a short duration storm, a medium duration storm and a long duration storm. The combination of the storm event and the R, T and K values help determine the shape of the I/I hydrograph. The R, T, and K values used for the Silverton model were determined during the calibration process.

The design storm used to evaluate wet weather system capacity was the 5-year, 24-hour storm. For the Silverton area this storm has depth of 2.52 inches. A SCS Type 1A unit hydrograph was used with this depth and input into the model to simulate this design storm. The design storm hyetograph is shown in Figure 8-2

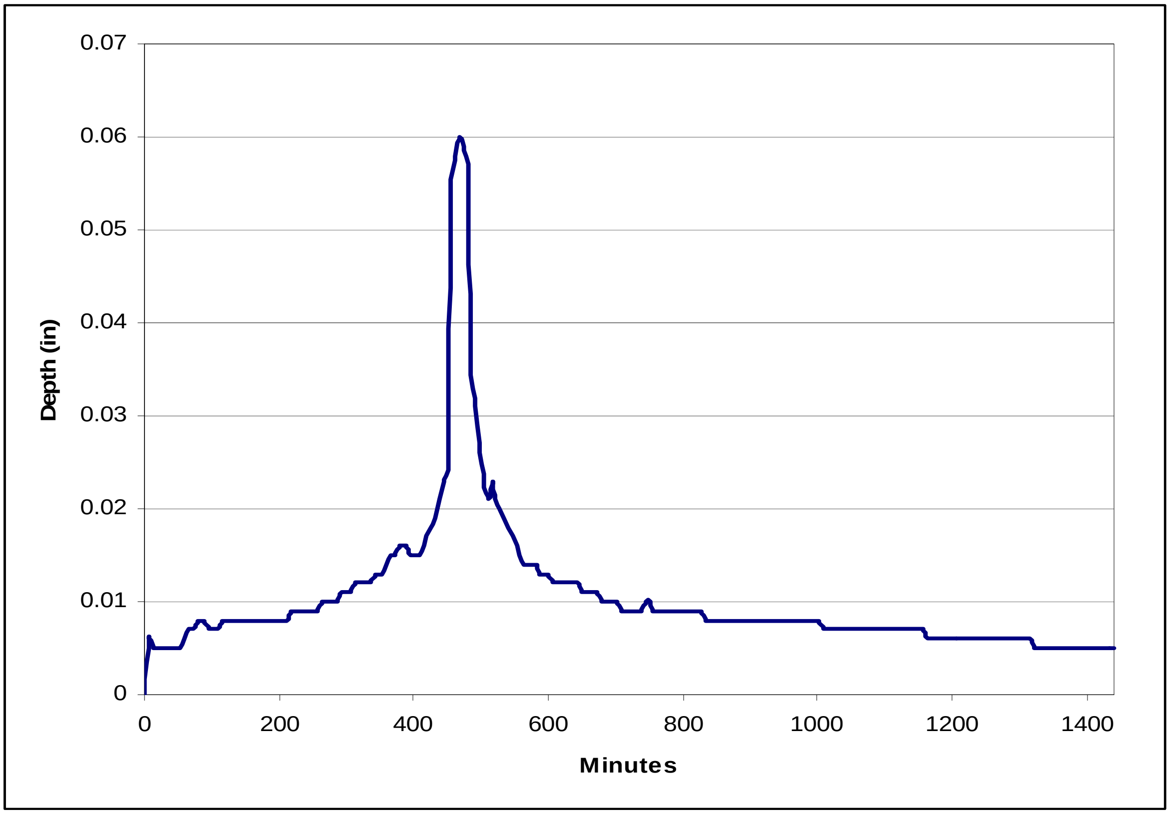


Figure 8-2: Silverton 5-Year 24-Hour SCS Type 1A Rainfall

Sewers and Manholes

The GIS data was used to construct the model representation of the existing conveyance system in MikeURBAN. Hydraulic connectivity was verified by reviewing alignment profiles. When issues were identified such as missing invert information or inconsistent slope information, the City was asked to check as-built drawings and/or field check information.

Diversion

There is one weir diversion in the system located in manhole 67 at the intersection of Smith and Water Streets. This diversion allows flow to be diverted to the pipe along Smith Street when flow in the pipe along Water Street is deep enough to over-top the weir. Unfortunately specific information on this weir is unavailable. Field crews estimate the weir height to be ±0.5 ft.

Lift Stations

The City provided lift station and wet well data. Lift station and wet well data included pump curves, set points and wet well dimensions and elevations. Pump curves were available for the Alder Avenue, Florida Drive, Grant Street and Hobart Road lift stations. The pump design point was used for the remaining lift stations (West Main Street, Monson Road, Jefferson Street, and Oregon Gardens lift stations).

Industrial Flows

Flows representing the two largest industrial contributors, Bruce Pac and Quest were assigned as point loads in the model. These flows were assigned to manholes closest to their actual physical location in the City. Table 6-3 shows the flow loads assumed for each industry and model location. Diurnal patterns were not applied to these flows.

For future planning purposes the City has identified an industrial reserve to be included when analyzing the system for future flow conditions. This industrial reserve is assumed to be of similar size as the current Bruce Pac usage. Future model runs included this industrial reserve to identify capacity issues that may arise from this additional future load. This load was applied in one of two locations – in the industrial area near the current Bruce Pac location (Site A) or in a possible future industrial area in the northwest portion of the City (Site B). Information on the industrial reserve is also shown in Table 6-3. Figure 8-3 shows the locations of where these industrial loads were applied.

Table 8-2: Industrial Flows

Industry	Maximum Day Flow Assumed (gpd)	Location	Model Location
Bruce Pac	171,000	1st & D Streets	Manhole 93
Quest	2,300	Eska St. south of Hobart St.	Manhole 858
Industrial Reserve	300,000	1st St. south of Schlador St. (Site A)	Manhole 92
		James St. north of Western (Site B)	Manhole JM202

Oregon Gardens Hotel Future Flow

A future resort hotel to be located near the Oregon Gardens has been identified. The hotel development is currently in the planning stages. For future planning purposes this facility has been included in the model runs evaluating future flow conditions. The developer anticipates opening this facility in 2008.

The maximum estimated flow contribution from the hotel is expected to be about 12,500 gallons per day. This determination is based on the following assumptions:

- 121 room resort hotel and conference center
- Maximum occupancy of 250 people
- Flow loading of 50 gallons per day per person

The location where this flow was loaded into the system is also shown on Figure 8-3.

Model Calibration

The model was calibrated for both wet and dry weather using flow monitoring data collected between April 7th and May 31st 2006. Two flow monitors were placed in the system to collect data. Site 1 was located on Water Street and High Street. Site 2 was located on Water Street between Jersey Street and Lane Street. These two monitoring locations are shown on Figure 8-3. Treatment plant influent data were also used for wet weather model calibration. The model is calibrated for peak flow value and hydrograph shape.

Dry Weather Flows

A four-step process was used to calculate the dry weather flows. The steps are:

1. Calculate the average daily flow for each loading manhole based on current land use and initial unit flow factors as described above.

2. Assign the identified diurnal flow pattern to each flow load location.

3. Run the model and compare results to the flow monitoring locations.

4. Re-compute the average daily flows by modifying the flow factors.

The steps were repeated until the model results represented observed monitored data. The calibrated model dry weather flow at the Site 2 monitor is shown in Figure 8-4.

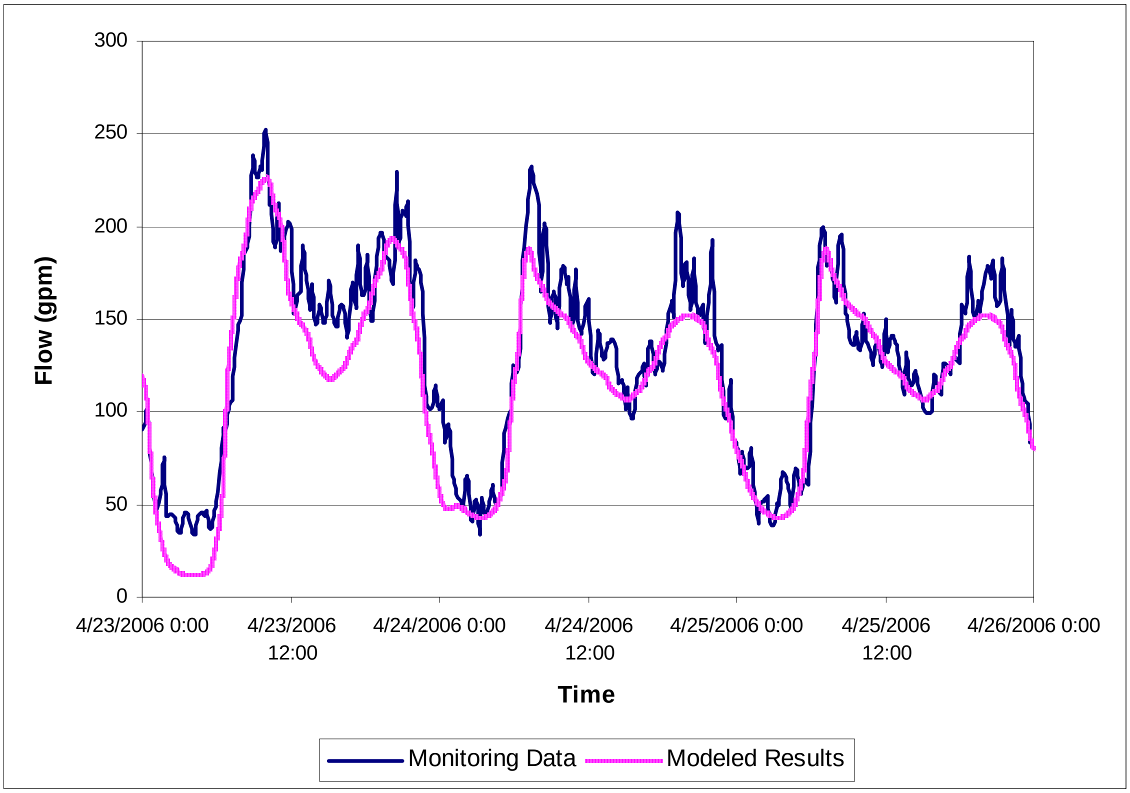


Figure 8-4: Dry Weather Flow Model Calibration

Inflow and Infiltration

The model was calibrated for wet weather flows using rainfall events that occurred during the monitoring period and monitoring data from the treatment plant and monitors. Unfortunately during the largest rainfall event during the monitoring period (beginning on April 14, 2006 and lasting 50 hours with a total rainfall of 1.4 inches) the influent meter at the treatment plant was being serviced. So an alternative storm event beginning on April 8, 2006 and lasting about 67 hours with at total rainfall of 0.9 inches was used for wet weather calibration.

The April 8, 2006 storm was used to determine the appropriate R, T and K values to be used for the model. The values used in the model are shown in Table 8-3

Table 8-3: Wet Weather Calibration Factors

Storm Type	R	T	K
Short Duration	0.001	3	1
Medium Duration	0.003	8	1
Long Duration	0.015	24	1

Figure 8-5 shows the results of the calibration results for the storm beginning on April 8, 2006 for the treatment plant location.

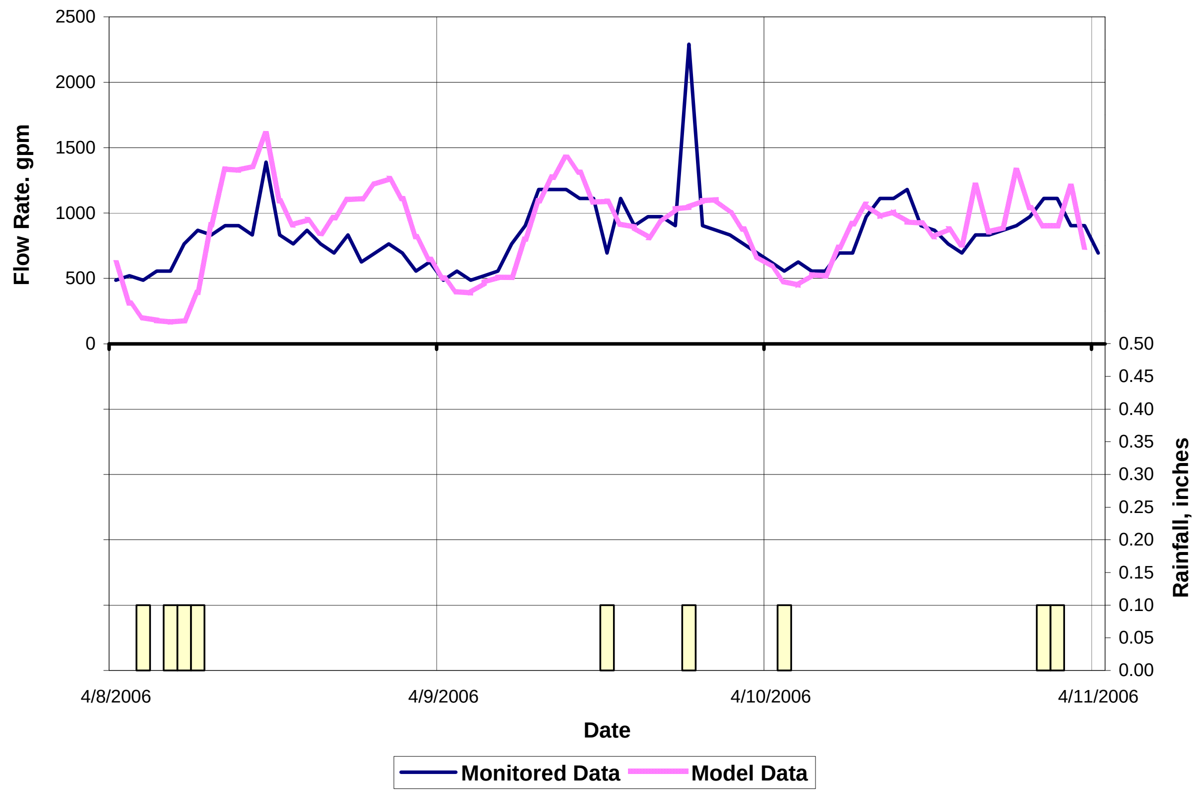


Figure 8-5: Wet Weather Flow Model Calibration April 8, 2006 Event – Treatment Plant

Figure 8-6 shows the results of the calibration for the storm beginning on April 8, 2006 for the Site 2 monitoring location.

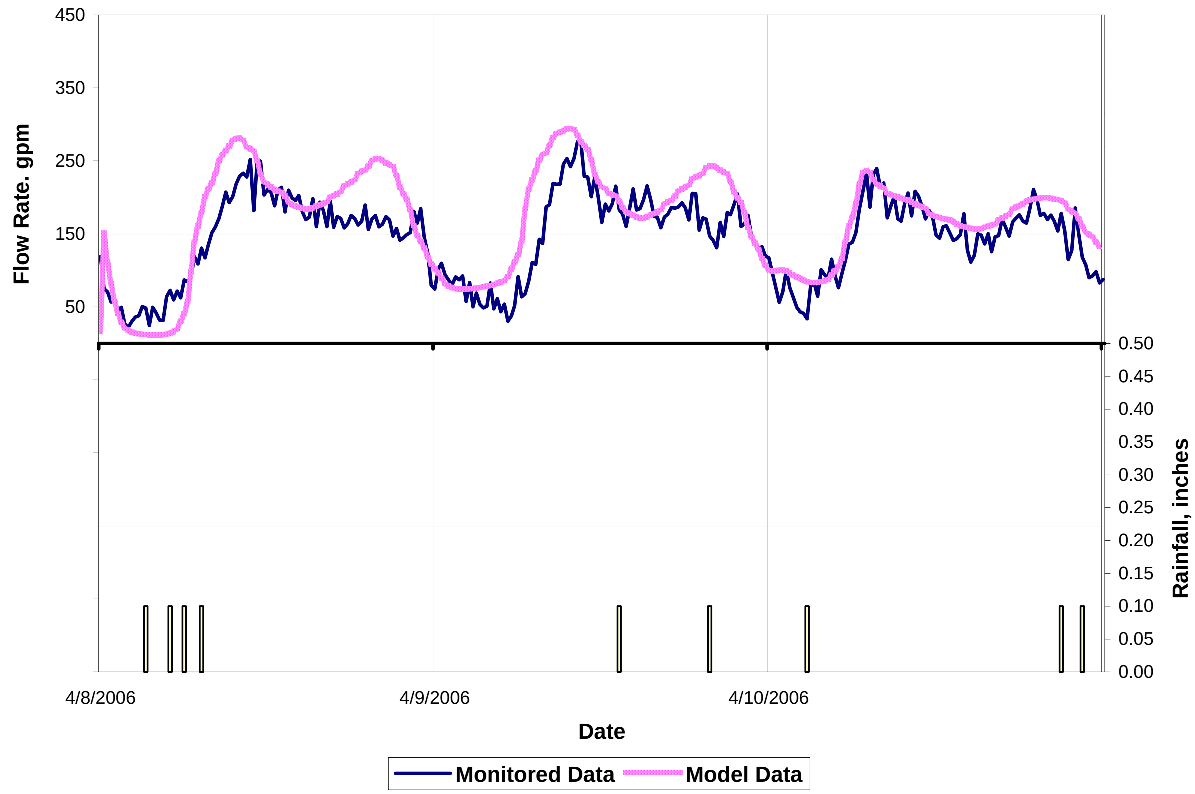


Figure 8-6: Wet Weather Flow Model Calibration April 8, 2006 Event – Site 2 Monitor

Hydraulic Criteria

The first task in the conveyance system analysis is to use the calibrated model to determine the location of problems. To characterize the hydraulics of the collection system, a set of hydraulic criteria were developed. The purpose of the hydraulic criteria is to provide a method to objectively evaluate model results and determine where improvements are needed. This section describes the criteria and their application.

Criteria

The primary goal of this study is to evaluate the ability of the sewer collection system to handle current and future flows by identifying areas where pipe capacity is exceeded. Throughout the system a pipe surcharge condition, where the hydraulic grade line of the water exceeds the pipe crown, is defined as an undesirable condition that could result in an unacceptable risk to property and health. As a result, pipes segments in the model where flow depths were at or above 75% of the pipe capacity for a particular flow condition were identified as critical locations.

The capacity for each lift station was also evaluated. If the modeling shows that the firm capacity of a lift station is exceeded then an increase in pumping ability is needed. Firm capacity is defined as the flow the lift station can pump with one pump out of service.

Criteria Application

After identifying problem locations, the problem cause was determined. For the sewer collection system, there are two main issues that cause the system to flood or cause risk to properties and health:

1. Upstream inflows exceed the conveyance capacity of the sewer.

2. Downstream constraints cause sewage to back up and impact upstream conduits. This includes pump station capacity issues.

Problem identification and potential causes are documented in the following section.

Conveyance System Analysis

This section describes the methods and results of the existing collection system analysis. Results of the analysis were used as a basis for development of the capital improvement program described in Chapter 10.

Method

Using the calibrated model, the hydraulic capacity of the existing collection system was analyzed based on year 2006, 2030 and ultimate build-out flow conditions. For all future model conditions model runs, the I/I rates and sanitary flows were increased as described above.

The model results were mapped using the GIS tools available in the model which enabled easy mapping of model results based on hydraulic criteria.

Capacity Results – Current Conditions

Figure 8-7 presents the model results with current development, Bruce Pac and Quest discharges, and estimated I/I using the 5-year rainfall and SCS Type IA storm pattern.

As shown in Figure 8-7, there are three locations where utilized pipe capacity exceeds 75%. One location is along Alder Street and crosses Silver Creek (Pipe 538-121). It is an inverted siphon and is expected to have the full pipe utilized. Since the inverted siphon does not negatively affect the capacities of surrounding pipes, the capacity of this pipe is sufficient for current condition flows.

The other two locations where pipe capacity was determined to be critical are summarized in Table 8-4.

Table 8-4: Insufficient Capacity Locations for Current Conditions

ID	Location	Upstream Manhole	Downstream Manhole	Diameter (in)	Length (ft)	Maximum Utilized Capacity (%)
CP-1	Westfield Street	MH-510	MH-507	6	910	80
CP-2	S. James Avenue	MH-116	MH-40	8	280	82

The capacity limitation shown in Table 8-4 for pipe CP-1 is due to pipe size constraints. The capacity limitation in pipe CP-2 is due to the water depth in the larger downstream pipe along Schlador Street.

The City has identified additional areas not identified by the model where I/I is a significant issue. These areas include along Schlador Street west of First Street and upstream of the diversion structure (MH-67) along Water Street. While the model did not identify Schlador Street as a critical location for capacity limitations, the capacities in this segment are approaching the critical stage of 75% utilized.

Conditions that may affect pipe capacity other than hydraulic limitations include direct connections from stormwater facilities and/or poor pipe conditions. A pipe condition assessment is discussed later in this chapter.

Capacity Results – Future Conditions

Two future flow conditions were run - 2030 flow loading and ultimate build-out flow loading. For both of these conditions two additional flow contribution were included in addition to the flow generated by the general population:

- An industrial reserve was included at one of two locations described above.
- Flow contributions from the proposed Oregon Gardens hotel.

In addition to the additional flow contribution locations additional system piping was added to the model. This new piping will be in place during the planning period. Flow loading in the model was adjusted to route future flows to these new pipelines. The areas include new pipes installed along Olsen Road in the south east part of the City, and new pipes along James Street in the northwestern part of the City. Associated with the James Street improvements are the addition of a new James Street lift station and the removal of the Jefferson Street and Florida Street lift stations.

2030 Conditions

Figure 8-8 and Figure 8-9 show the results of the analysis for projected 2030 population and include the estimated Oregon Gardens hotel load and industrial reserve. Figure 8-8 shows the system capacity results with the industrial reserve applied at Site A, near the existing Bruce Pac facility. While Figure 8-9 shows the results with the industrial reserve applied at Site B in the northwestern portion of the City. The analysis identified four areas where utilized pipe capacity exceeds 75%. These locations are summarized in Table 8-5.

Table 8-5: Insufficient Capacity Locations for 2030 Conditions

ID	Location	Upstream Manhole	Downstream Manhole	Diameter (in)	Length (ft)	Maximum Utilized Capacity (%)
CP-1*	Westfield Street	MH-510	MH-507	6	910	83
CP-3	S. James Avenue	MH-503	MH-502	12	576	82
CP-4	Sherman Street	MH-120	MH-09	12	175	86
CP-5	Adams Street	MH-285	MH-28	8	407	79

* Same locations as in Table 8-4

Item CP-1 is the same as shown in Table 8-4. Item CP-2 from Table 8-4 has been eliminated with the system modifications associated with the new pipeline and lift station along James Street. The capacity limitation in pipe CP-3, shown in Table 8-5, is due to a lower slope in the next downstream pipe. Capacity limitations in pipes CP-4 and CP-5 are due to the water depth in the larger downstream pipes.

In addition to the pipe capacities, pump station capacity was also evaluated. Currently the at each pump station only one pump is active at a time. Modeling showed that with the addition of the Oregon Gardens hotel both pumps were needed to convey the additional flow to the rest of the system. Thus, the pumps at the Oregon Gardens Pump Station will need to be upgraded. With this upgrade the Oregon Gardens force main, currently 4-inches, will also need to be upgraded. Velocities in this forcemain including the projected hotel flows are about 10 ft/s.

Ultimate Build-Out Conditions

Figure 8-10 and Figure 8-11 show the system capacity results for the system analysis using the ultimate build-out population, and includes the Oregon Gardens hotel and industrial reserve. Figure 8-10 shows the results for the industrial reserve located at Site A, near the existing Bruce Pac facility. Figure 8-11 shows the results for industrial reserve located at Site B in the northwestern portion of the City. In addition to the areas identified in the 2030 analysis, the analysis identified additional locations where utilized pipe capacity exceeds 75%. These locations are summarized in Table 8-6.

Table 8-6: Insufficient Capacity Locations for Ultimate Build-Out Conditions

ID	Location	Upstream Manhole	Downstream Manhole	Diameter (in)	Length (ft)	Maximum Utilized Capacity (%)
CP-1*	Westfield Street	MH-510	MH-507	6	910	87
CP-3*	S. James Avenue & McClaine Street	MH-504	MH-502	12	1,126	92
CP-4*	Sherman Street & Maple Street	MH-121	MH-09	12	342	85
CP-5*	Adams Street	MH-824	MH-28	8	850	92

* Same locations as in

All items in Table 8-6 are the same as those listed in Table 8-5. The reasons for the capacity limitations remain the same for each. For items CP-3, CP-4 and CP-5 capacity issues have expanded to include the next upstream pipe segment.

Other than the Oregon Gardens Pump Station, no other pump station capacity improvements are anticipated for ultimate build-out conditions.

Conclusions

Based on model results, addressing the capacity issues identified in and Table 8-6 will prevent most problems seen under future flow conditions. The following conclusions were drawn from the system analysis.

- Upgrade to the Oregon Garden Pump Station will be necessary to accommodate flows from the new Oregon Gardens hotel.
- Capacity improvements are needed at various locations in the system. Some capacity improvements may be combined with improvements identified in the condition assessment.
- Additional capacity issues may arise due to poor pipe condition and/or direct connections to stormwater facilities. These locations should be identified through a conditions assessment program as discussed later in this chapter.

Solutions to the problems identified in the conveyance system analysis are described below.

Recommendations

Capacity Improvements

Table 8-7 lists pipeline improvement projects that are recommended to address capacity issues identified in the hydraulic modeling analysis. The table also lists estimated construction costs. Costs are in 2006 dollars (based on an ENR multiplier of 8655) and include contingency and engineering, administrative and legal costs. Detailed information on the construction costs can be found in Appendix E.

Table 8-7: Recommended Capacity Related Pipeline Improvements for 2030

Improvement ID	Capacity Issue ID	Improvement Location	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-1	CP-1	Westfield Street	Upsize 6-inch to 8-inch	910	$229,800	2008
IMP-2	n/a	Oregon Gardens Pump Station and force main	Increase pump station firm capacity from 200 gpm to 400 gpm.	2 new 400 gpm pumps (1 stand-by)	$18,600	2007 - 2008 (completed before hotel opening)
			Upsize force main from 4-inches to 6-inches	909	$182,500	
IMP-3	CP-3	S. James Street	Upsize 12-inch to 18-inch	576	$214,600	2020-2030
IMP-4	CP-4	Sherman Street	Upsize 12-inch to 18-inch	175	$70,000	2020-2030
IMP-5	CP-5	Adams Street	Upsize 8-inch to 12-inch	850	$283,900	2020-2030

Additional Pump Stations

In addition to the pipeline improvements identified in Table 8-7, the City has identified the locations for three new pump stations to serve future growth areas within the Urban Growth Boundary. These pump stations are described in Table 1-5.

Table 8-8: Additional Pump Stations

Improvement ID	Pump Station	Description	Estimated Cost*	Project Timing
PMP-1	James Street	New pump station & 8-inch forcemain. Including 18-inch and 12-inch trunk lines on James and Jefferson to connect to existing system. Decommission James & Florida Drive & Second & Jefferson Street Pump Stations	$928,400	2008
PMP-2	Pine Street	New pump station & forcemain	$162,100	2009
PMP-3	Setness Lane	New pump station & 6-inch forcemain and associated 8-inch collector pipes.	$1,038,000	2020

* Estimated cost for the James Street and Pine Street Pump Stations provided by the City.

Estimated costs for the James Street and Pine Street pump stations were supplied by the City and adjusted, where necessary, to 2006 dollars. The cost for the Setness Lane Pump Station was estimated based on ultimate condition flows and estimated pipe lengths. All estimated costs should be revisited detailed pump station design begins.

Improvements Based on Known Present Condition

A limited condition assessment was completed as described in Chapter 6. Table 8-9 lists high priority pipeline improvement projects that are recommended to address condition issues identified in the Electroscan condition assessment that was performed in August, 2006. The table also includes estimated total construction costs.

Table 8-9: Recommended Condition Assessment Related Pipeline Improvements

Improvement ID	Improvement Location	Existing Diameter (in)	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-6	Schlador Street	18	Slipline/pipeburst	572	$70,000	2007
IMP-7	Lone Oaks Street	15	Slipline/pipeburst	355	$40,000	2007
IMP-8	Third St.	15	Slipline/pipeburst	770	$85,000	2008
IMP-9	Meat Packers/High School Area	18	Slipline/pipeburst	385	$46,000	2008

A unit cost of $110 per linear foot was used to determine the estimated cost for rehabilitation on the Loan Oaks and Third Street rehab projects (IMP-7 and IMP-8). A unit cost of $120 per linear foot was used to determine the estimated cost for rehabilitation on the Schlador Street and Meat Packers pipe segments (IMP-6 and IMP-9).

Timing of Improvements

Recommended improvements have been prioritized based on the deficiency analysis described above. For each capacity and condition assessment project identified a recommended project initiation date has been estimated. These dates are listed in Table 8-7 and Table 8-9 and described further below.

- Work on IMP-1 should prior to 2008 to accommodate existing flows as well as future flows associated with the planned Oregon Gardens resort hotel.
- Upgrading the Oregon Gardens pump station and force main (IMP-2) should be completed prior to the opening of the planned hotel. The developer plans to have this hotel open in 2008.
- IMP-3, 4 and 5 are needed to address critical capacity issues identified in the system for the 2030 conditions. These improvements should be in place prior to the end of the planning period in 2030.
- IMP-6, 7, 8 and 9 are listed according to priority based on Electroscan results obtained in August, 2006. The grouping of CA-1 with CA-2 and CA-3 with CA-4 assumes an approximate capital expenditure of approximately $120,000 per year during FY 2007 and 2008 respectively.

Condition Assessment Expansion

As stated previously, a limited condition assessment was completed as described in Chapter 6. However, to better understand the system condition and help refine the prioritization of sewer CIP projects, it is recommended that condition assessment continue as part of the City's routine maintenance program. In order to develop a systematic condition assessment approach, a complete analysis was performed on the collection system that utilized all known physical and historical information available. The primary source of information was the City's GIS database with supplementary information provided by the City's 1986 Sanitary Sewer Inventory. The purpose of this effort was to determine a prioritized schedule for expansion of the sanitary sewer condition assessment program.

The following criteria (in order of importance) were used in order to rank the numerous sewer segments for prioritized condition assessment:

2. Pipe Material

    a. Clay

b. Concrete (primarily along Water Street)

c. Unknown material

d. Remaining concrete

e. Ductile iron

f. PVC

Within each of the pipe material classes listed above, suggested priority was given to larger diameter pipes over smaller diameter pipes. For example, a 15-inch diameter concrete pipe would have been given suggested priority over a 10-inch diameter concrete pipe. Also, within each diameter classification, high priority was given to long reaches of pipe over short reaches.

The resulting recommended list of prioritized pipe segments is presented in Appendix F. A summary of the types of pipe materials and corresponding lengths required for condition assessment is presented in Table 8-10 below.

Table 8-10: Prioritized Program for Future Condition Assessment

Pipe Material	Total Length Required for Assessment (ft)	PW Cost	Year(s) to be Performed
Clay	6,080	$6,080	2007
Unknown	63,530	$51,163	2008-2019
Concrete (excluding Water St.)	24,830	$16,662	2019-2020
Ductile Iron	1,780	$1,177	2020
PVC	52,080	$29,830	2020-2030
Total	**148,300**	**$104,913**	

Present worth costs were calculated for a discount rate of 3% over a 25 year period. A unit cost of $1.00 per linear foot inspected was assumed.

An average assumed inspection footage of approximately 6,000 linear feet per year was used to determine the length of time that would be required to inspect each class of pipe. This is shown in the "Year(s) to be Performed" column of Table 8-10.

Chapter 9 - Wastewater Treatment and Disposal Master Planning

Liquid Treatment

Headworks and Primary Treatment

Based on facility plan projections for updated flows and loads, it was determined that projected flow would not increase beyond the existing design flows (see Chapter 4). Both the influent screening and primary clarifier are adequate for the existing design flow; therefore, they do not require expansion at this time.

The existing influent screen was commissioned in 1996 and is in very good shape. It is expected that with adequate maintenance, the screen's useful life would extend to 2030.

The primary clarifiers are also rated for the current and future design flows and no additional clarifiers are required prior to 2030. The existing mechanism is over 20 years old, but is still working well; however, due to its age, cost for replacement should be anticipated between 2020 and 2030. However, it may remain in service while repair and maintenance efforts are within acceptable levels. The costs for the mechanism replacement are estimated in the range of $80,000 – $100,000.

Secondary Treatment

Present flows and loads are equivalent to 45 percent of the existing secondary treatment design capacity. Based on the existing flows and loads secondary treatment has a capacity of about 2.6 MGD, which would be sufficient until 2030.

However the updated flows and loads change the design capacity of the existing plant as the new design waste load is stronger. Table 9-1 shows a comparison of existing design loading and future design loading. The reason for the different projections for flow and loads are outlined in Chapter 4. Based on design BOD and TSS loadings, the secondary treatment capacity will be reached when maximum month dry weather flows reach 2.2 MGD, meaning additional capacity would be required in 2020 and planning would have to begin in 2015.

Due to limited historic performance data of stable operation in its design mode it is difficult to evaluate the potential secondary treatment system performance and related facility improvements needs. The effluent ammonia data show occasional effluent ammonia excursions in excess of the most stringent future maximum daily ammonia limit (1.9 mg/L). On a monthly average basis the plant has been consistently meeting the future effluent ammonia requirement of 0.84 mg/L maximum monthly average.

Table 9-1: Comparison of current and future design flows and loads

	Current	Future (2030)
DWMM Flow	2.65 MGD	2.65 MGD
INF TSS Load	7900 lb/d	8525 lb/d
INF BOD Load	7900 lb/d	9158 lb/d
INF TSS concentration	357 mg/L	385 mg/L
INF BOD concentration	357 mg/L	417 mg/L
PE TSS concentration *	161 mg/L	173 mg/L
PE BOD concentration *	232 mg/L:	269 mg/L

*based on typical PCL removal rates

Because the excursions are single day events, they indicate that for a short period of time, the nitrification capacity of the aeration system was exceeded resulting in substantial ammonia bleed through. One operational parameter that increases the impact of such peak loads is the air on/off mode; even when the on/off cycle is alternated between trains (which were not the case during the first years after startup with only one train running) half the flow is only partially nitrified during the air off cycle. With current online monitoring technology, control feedback is available to adjust the on/off cycle according to the nitrification requirements for permit compliance.

The ammonia bleed through disadvantage of the air on/off cycle will be magnified with increasing flows and loads.

This problem can be overcome by eliminating the air on/off cycle and instead relying on the racetrack DO profile to provide local anoxic conditions and subsequently simultaneous nitrification/denitrification. Furthermore, online ammonia analyzers could be installed to monitor the nitrification performance and provide a second control signal in addition to DO control signal. This would allow operators the run the aeration system for maximum nitrogen removal. The control loop would be designed such that the effluent ammonia concentration defines the DO set points, which control the air supply.

Even with improved DO control, expansion of the secondary treatment system will be required during the planning horizon. Due to the previously mentioned lack of data representing normal secondary treatment operating conditions, the future capacity and treatment performance was estimated based on conservative design criteria and typical treatment performance.

The approach for the secondary treatment upgrade consists of two phases; (1) process control upgrade and optimization, which would include a rerating of the plant to its true capacity, and (2) capacity expansion.

The alternatives considered for the secondary treatment capacity expansion include:

- Capacity Expansion by addition of a third treatment train
- Capacity expansion with Membrane Bioreactor Technology
- Capacity expansion with Hybrid Technology

Some future solids treatment scenarios may produce a recycle stream with high ammonia concentrations. It is therefore assumed that future dewatering recycle would be flow equalized, which minimizes the impact on secondary nitrogen removal. Under this scenario the recycle load would account for 5% of the total capacity.

Phase 1 – Secondary Treatment Improvements - Process control upgrades and optimization

Based on the process review conducted as part of this Facility Plan, it appears likely that the treatment process can be optimized to gain additional treatment capacity. In order to optimize the process for improved performance and increased capacity, some process control improvements are necessary.

Currently the secondary treatment system is equipped with basic process control and monitoring equipment. While this level of control was and is adequate under current flows and loads, once influent flow and loading begin to approach design values, the lack of better control will be limiting to both effluent quality and treatment capacity. The recommended process upgrades (which include the necessary SCADA upgrades) are:

- Online alkalinity control
- Aeration control based on multi-point aeration basin DO measurement and online effluent ammonia analyzer
- Automated SRT with Online MLSS meter

These process control improvements would increase capacity and treatment performance. They could also eliminate certain routine tasks such as sampling and analysis.

Online Alkalinity control

Presently, alkalinity is added at a constant dosage rate, which is set based on periodic alkalinity analysis. To account for day to day and diurnal variability, the dosage rate is set conservatively to maintain a minimum alkalinity at all time. This results in overdosing of alkalinity supplement at times, but also insufficient dosage other times. While this does not result in direct pH related permit violations, it does potentially result in higher chemical and cost and higher cost associated with the handling of additional chemical sludge. More importantly, it results in fluctuations in alkalinity and pH in the aeration system, which is less favorable for nitrification and overall stable treatment performance.

The online alkalinity control system would include an online alkalinity meter located just upstream of the aeration basin but downstream of the alkalinity feed point and downstream of the RAS return. Based on the online alkalinity reading, the alkalinity supplement feed would be adjusted to the target alkalinity within a range of +/- 10 mg/L of the target. This design approach assumes that the current practice of air on/off nitrification would be replaced by simultaneous nitrification denitrification.

Aeration Control Upgrade

Presently, the air supply is controlled by a single DO probe per basin. The DO reading is used to control a modulating air control valve while a pressure sensor upstream of the modulating valve controls the blower output.

To increase denitrification, the air is turned off altogether every 4.5 hours for 1.5 hours. While the trains are alternated, this does still allow a certain amount of ammonia bleed through during the air off time and could be partially responsible for the occasion effluent ammonia excursions. To eliminate these air-off related ammonia excursions, the air on/off denitrification would be replaced by simultaneous nitrification/denitrification.

Simultaneous nitrification/denitrification is accomplished by varying the air supply such that the roughly 20 – 30% of the basin maintains anoxic conditions. To achieve the appropriate level of control, two more DO probes are required and would be placed at a distance equivalent to 30% and 40% of the basin rotation time counter clockwise from the beginning of the diffuser grid.

Under this operating mode, the DO is controlled to maintain a target DO residual at the new probe locations. The resulting DO will still be monitored at the current location as additional control feedback.

To insure maximum nitrification, an online effluent ammonia analyzer would be installed. If the effluent ammonia increases above the target value, the size of the anoxic basin fraction would be reduced or completely eliminated. This will allow maximum nitrification capacity at peak loading time at the expense of nitrogen removal, which is not a permit requirement. Conversely this control system as a whole would allow maximizing denitrification and with that oxygen and alkalinity recovery as well as SVI control through more consistent selector operation.

The existing performance evaluation has shown a strong correlation between low effluent phosphate concentrations and low SVI values. Theoretically one would expect low SVI values with a functioning anoxic selector that the above described control upgrade would provide. If effluent phosphorus remains the indicator parameter for SVI, online metering of phosphate and/or nitrate may be justified to control the size of an anaerobic selector. However it is assumed that this would not be necessary after the aeration control upgrades have been implemented.

Finally, the automation of SRT control offers the advantage of operating the plant much more consistently while reducing day to day operator activities related to SRT control. The automated SRT control can be accomplished two ways; (1) installation of an online MLSS meter, and (2) or changing the SRT control strategy to hydraulic wasting.

The online MLSS probe simply replaces the manual effort of taking MLSS samples, running the TSS analysis, and adjusting the wasting rate based on the measured SRT and the target SRT. To close the control loop, the WAS concentration is estimated based on measured MLSS and RAS rate, both of which are then used to determine the wasting rate and automatically adjust the WAS pump speed. This would require a variable speed drive for the WAS pump.

Hydraulic wasting is a different activated sludge control approach. Instead of maintaining a target MLSS that is based on a target SRT, the hydraulic wasting method maintains a constant SRT by wasting a constant fraction of the RAS; via the flow paced RAS rate the WAS is also pumped flow paced. The result is a constant SRT but varying MLSS due to varying loadings. Further discussion is based on online MLSS monitoring, however either strategy is suitable for Silverton and the ultimate decision should be strongly influenced by staff preference.

Process Optimization and Capacity Rerating

The control upgrades described in the previous section are an important element in the process optimization as they provide the necessary tools for the operator to fine tune the activated sludge process. This would entail:

- adjusting control loops and SCADA programming
- controlled variance of key control parameters such as SRT, target DO, anoxic zone size, effluent ammonia concentration
- expanded water quality parameter and process monitoring program

Once the process and its operation is optimized, full scale stress testing would be conducted. During the stress testing, the load to one train will be gradually increased while its performance is closely monitored. The second train will treat the residual and be available as backup to the stress testing train. The objective of the stress testing is to determine the maximum capacity of the secondary system, including secondary clarifier capacity. The stress testing should be conducted while there is sufficient excess capacity and during months of spring and early summer, and would ideally span a four- to six-month period. Further process optimization may become necessary during the stress testing. The stress testing should also include peak load events.

It is very helpful to conduct secondary process simulation in parallel to the stress testing. With a calibrated process model, anticipated process performance under the next stress testing level can be simulated ahead of time. It also allows testing of scenarios that cannot be tested full scale due to the potential for causing permit violations. The results of the stress testing and process simulations can ultimately be used to rerate the secondary treatment facility to its true capacity in order to refine the implementation timeframe for the secondary process expansion.

Table 9-2 shows a comparison of the existing capacity based on the updated flows and loads and the capacity of the optimized system. The two design parameters that stand out are the mixed liquor concentration of 4500 mg/L, which is higher than typical conservative design values but is certainly within the typical range for actual conventional activated sludge facility operation (including operation at Silverton, where the plant has been operated MLSS above 4000 mg/L in the past years). In addition the two large secondary clarifiers provide sufficient surface area that the resulting solids loading rated remain below typical design loads for modern clarifiers (25 lb/sf/d). The other design parameter is the oxygen uptake rate (OUR). Typically, 55 mg/L/hr is considered the maximum for conventional activated sludge. Thus the plant would be operated near the maximum oxygen uptake rate, which is acceptable for an optimized system with an advanced process control system.

Table 9-2 shows that the combination of process control upgrades and process optimization can increase the treatment capacity substantially. This equates to adding one additional aeration basin, and would provide sufficient capacity beyond the 2030 planning horizon. Since the oxygen demand increases beyond what the existing blowers can deliver, an additional blower would have to be installed or existing blowers replaced with larger blowers to increase the aeration capacity to approximately 5000 scfm. It is assumed that aerating system upgrade would also include additional diffusers and some modification and expansion of the existing air piping.

Table 9-2: Design parameters for Optimized Secondary Treatment

Parameter	Unit	Existing*	Optimized
Aerobic Volume**	MG	1.3	1.3
SRT	Day	10	10
Yield	lb TSS/lb BOD	0.65	0.65
MLSS	mg/L	3,000	4,500
SCL Loading***	lb/sf/d	8.4	18.8
OUR	mg/L/hr	35.2	52.8
Oxygen Demand	lb/d	9160	13740

Capacity		2.2	3.3

*based on updated flows and loads

** 100% of aeration volume aerated under maximum month conditions

*** with two clarifiers in service

Table 9-3 summarizes the estimated cost (in 2006 dollars) for the process control upgrades and process optimization. The process control upgrade costs include any necessary upgrades to the SCADA system and programming. Phase 1 includes upgrades recommended as soon as they can be accommodated in the City's budget. Phase 1b should be implemented between 2010 and 2015, and Phase 1c improvements should be implemented when influent maximum month dry weather flows approach 2.2 MGD.

Table 9-3: Estimated Cost for Process Control Upgrades and Process optimization

Alkalinity Feed Control	$75,000
Aeration control	$150,000
Online Ammonia Analysis	$50,000
Total – Phase 1a	**$275,000**
Aeration system upgrade*	$250,000
Total – Phase 1b	$250,000
Automated SRT control	$75,000
Stress Testing**	$40,000
Rerating	$10,000
Total – Phase 1c	**$125,000**

* additional diffusers, headers, and blowers

** does not include operator time, assumes are sample analysis would be done in-house or with online metering equipment

Phase 2 – Secondary Treatment Improvements - Capacity Expansion

Capacity Expansion with conventional treatment

This expansion alternative would simply add an additional aeration basin as planned in the previous expansion, which increases the secondary treatment capacity by 50%. With the existing secondary clarifier surface area, no additional clarifier would be required. The two existing clarifiers provide sufficient capacity for the 2030 design condition. Table 9-4 shows a summary of design parameters expanded treatment system with three aeration trains and two clarifiers.

Table 9-4: Design parameters for Secondary Treatment third Conventional Train

Parameter	Unit	Optimized Existing	With 3rd Train
Aerobic Volume*	MG	1.3	1.95
SRT	Day	10	10
Yield	lb TSS/lb BOD	0.65	0.65
MLSS	mg/L	4,500	4,500
SCL Loading**	lb/sf/d	18.8	12.5
OUR	mg/L/hr	52.8	35.2
Oxygen Demand	lb/d	13,740	20,610
Capacity	MGD	3.3	5.0

*100% of aeration volume aerated under maximum month conditions, ** with two clarifiers in service

Table 9-5 summarizes the cost estimate for this alternative, which includes a new aeration basin, replacing two existing blower with larger models, and yard piping.

Table 9-5: Estimated Cost for Secondary Treatment third Conventional Train (order of magnitude estimate)

3rd Aeration Basin	$1,500,000
Yard Piping	$200,000
Blower Expansion	$200,000

Capacity Expansion with MBR

In recent years membrane bioreactors (MBRs) have become the technology of choice for many new facilities and for facility expansion. The main benefit of the MBR is that the secondary clarifier is replaced with a physical membrane barrier. This allows the operator to raise the maximum mixed liquor concentration and subsequently, the SRT. Typically, MBRs operate with MLSS between 7,000 and 10,000 mg/L while producing effluent quality with no suspended solids and turbidity of les the 0.1 ntu.

However, the advantages of the smaller foot print and reuse quality effluent come at the price of potentially significantly higher capital cost.

The factors that would increase the economic feasibility of MBR technology are:

- Requirement for reuse quality effluent
- Need for additional capacity
- Lack of space for conventional expansion

Presently, neither capacity nor the lack of space provides strong incentive for MBR technology. Cost of a full upgrade to 2.6 MGD MBR capacity would be in the range of $8 – $14 million. Effluent filtration for TSS and turbidity compliance can be installed at a fraction of the cost.

One option to significantly reduce the cost for MBR is to only provide MBR capacity for a fraction of the design flow. In the case of a single train, MBR design would lend itself as a solution to reduce the total cost. A separate membrane holding tank would be constructed which would operate in combination of either train. How much capacity would be installed can then be depended on the demand for reuse quality effluent or the demand for low TSS and low turbidity effluent for final effluent blending. For this alternative it is assumed that initially MBR capacity would be equal to 50% of the summer permit season maximum month flow.

Table 9-6 summarizes the key design parameters for a single train MBR upgrade at Silverton WWTP. The design includes

- Membrane module tank
- Mixed liquor recycle pumps
- Piping sized to handle 4 Q to and from either aeration basin
- MBR effluent piping
- MBR support building
- MBR peripherals (Clean in place, scour air blowers, chemical feed etc)

The flux, which is the amount of water that can be pushed through a square foot of membrane surface, improves with the solids retention time. Therefore, it is advantageous to run a higher SRT, even though a much lower SRT would be sufficient for nitrification. Fifteen days were selected as the design SRT. The other advantage of higher SRT is that it lowers the WAS yield to approximately 0.55, which reduces the total solids load to the digester.

Municipal MBR system capacity is typically limited by how much oxygen can be transferred. Using a maximum oxygen uptake rate of 75 mg/L/hr, a single MBR train at Silverton would provide 2.3 MGD capacity. The second existing (optimized) train would add another 1.65 MGD to bring the total also to 3.9 MGD. The second train would not only provide additional treatment capacity but also allow a more cost efficient MBR design by absorbing the peak hydraulic loads and reducing the maximum flux requirements for the MBR.

One key concern with a single train MBR at Silverton is redundancy. To accomplish full redundancy for the MBR train, the upgrade would be designed that either train would be used as the MBR train.

Table 9-6: Design summary for the single train MBR

Parameter	Unit	MBR Train	Optimized Conventional Train	Total
Aerobic SRT	day	15	10	-
Yield	lb TSS / lb BOD	0.55	0.65	-
MLSS	mg/L	8,000	4500	-
Total Volume	MG	0.65	0.65	1.3
OUR*	mg/L/hr	73.9	52.8	-
Oxygen Demand*	lb/d	9,620	6,870	16,490
Capacity	MGD	2.3	1.65	3.9

* 100% BOD removal, 90% TKN nitrification,

The advantages of an MBR-based upgrade at Silverton are:

- Reuse quality effluent from MBR train
- Reduced energy demand on UV disinfection (lower effluent suspended solids)
- Modular upgrade possible

The disadvantages are:

- Expensive upgrade
- Increased operation cost through membrane replacement and pumping cost
- High internal recycle flow (3- 4Q)
- Fine screening required

Table 9-7 summarizes the main cost items for the MBR upgrade. The MBR system includes equipment provided by the MBR manufacturer, most of which would be housed in the MBR building (i.e. scour air blower, clean-in-place system, electrical, controls). It is assumed that fine screens would be installed upstream of the aeration basin and only screen the MBR influent. Because of the much higher oxygen demand in the MBR train, the aeration system would have to be modified to roughly double in capacity.

Table 9-7: Estimated Cost for the single train MBR (order of magnitude estimate)

MBR upgrade	$6,000,000
Blower Expansion	$200,000
Yard Piping	$200,000

Capacity expansion with Hybrid Technology

Hybrid systems refer to an activated sludge system with some kind of integrated attached growth. This attached biomass can be growing on fixed media as well as suspended media. Hybrid systems are further subdivided into two groups:

- Integrated fixed film activated sludge systems (IFAS)
- Moving bed biofilm reactor (MBBR)

The media used in the IFAS system can be physically fixed in place and suspended within the aeration basin. The purpose is to allow growth of additional biomass without increasing the concentration of suspended biomass (and thus loadings on secondary clarifiers). Even more significant for nitrification, the biomass age attached to the media is independent from the suspended biomass SRT. This makes it possible to grow nitrifying bacteria at much lower suspended biomass SRTs than conventional activated sludge systems.

IFAS Hybrid systems can reach stable nitrification at a suspended SRT[4] of 3 – 4 days. This makes the IFAS system an attractive option to increase the secondary treatment capacity at Silverton WWTP.

The volume displaced by the media is less then 8%, with media fill of less then 50%. Because the nitrifying bacteria grow on the media, the treatment capacity is dependent on the total media surface. This makes it possible to modulize the upgrade by only installing enough media to meet current permit requirements. If additional capacity is required, media can be added in the future.

Figure 9-1 shows some examples of IFAS systems. For this alternative the IFAS technology with suspended media was selected. For the use of suspended media requires some basin modifications would be required including the installation of media retention measures and coarse bubble aeration. Suspended media has a number of

[4] Accounts only for suspended biomass and biomass attached to media

advantages, such as ease of installation and the ability to pump media into another basin or storage tank during basin maintenance.

Figure 9-1: Examples of IFAS Hybrid media system

The design criteria listed in Table 9-8 shows key design parameters for this alternative. They indicate that upgrading the existing aeration basin with IFAS would increase the treatment capacity by only 6% percent. The reason for the small increase is that the hybrid system was limited to a maximum oxygen uptake rate of 55 mg/L/hr. Since this is still an emerging technology little experience is available about the practicality operating at higher OURs than 55 mg/L/hr. It would be reasonable to assume, however, that by the time planning for the next expansion begins, these specific design questions will have been addressed through years of operating experience from numerous facilities.

Table 9-8: Design parameters for Secondary Treatment third Conventional Train

Parameter	Unit	Optimized Existing	With Hybrid Media
Aerobic Volume*	MG	1.3	1.3
SRT	Day	10	5
Yield	lb TSS/lb BOD	0.65	0.8
MLSS	mg/L	4500	3,000**
Media content	%	-	40
SCL Loading***	lb/sf/d	18.8	12.7
OUR	mg/L/hr	52.8	55.0
Oxygen Demand	lb/d	13,740	14,390
Capacity		3.3	3.5

* 100% of aeration volume aerated under maximum month conditions
** suspended solids only
*** with two clarifiers in service

The advantages of upgrading Silverton WWTP with IFAS technology are:

- Relatively simple retrofit
- No additional aeration basin volume required
- No increase in solids loading to the secondary clarifiers
- Modular and staged upgrade possible
- Nitrifying bacteria on media cannot wash out during peak flow event, thus allowing more stable nitrification
- Simultaneous denitrification can occur in the media’s fixed film biomass, thereby facilitating nitrogen removal
- Hybrid course bubble aeration system requires less energy and less maintenance, while maintaining fine bubble oxygen transfer rates due to the extended contact time

The disadvantages are:

- Increased aeration basin headloss through media
- Requires retrofit of existing aeration diffuser grid
- Newer process

- Emerging technology

Because the IFAS technology is still emerging, the type and amount of media required would be determined by performing a pilot test with the actual media. The required treatment capacity and total media surface can be calculated based on the measured performance and known pilot filter media surface. The setup for a Hybrid media pilot is shown in Figure 9-2.

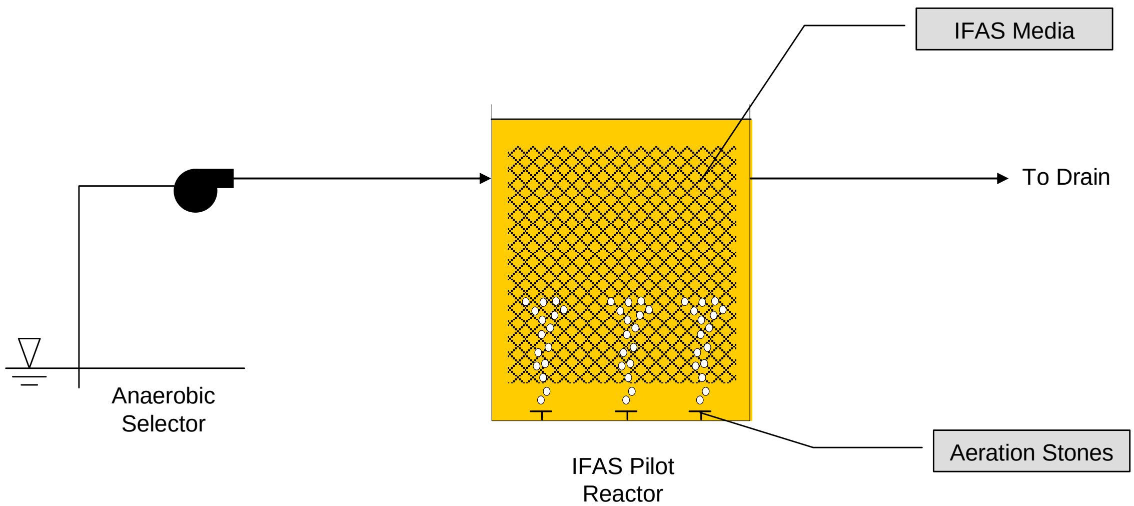


Figure 9-2: Schematic of IFAS media pilot test

Table 9-9 lists the costs for the IFA improvements, which include:

- Installation of course bubble diffusers
- Installation of media retention sleeves for suspended media
- Installation of media support system for fixed media
- Media removal, replacement, and storage infrastructure
- Odor control (required for media storage during maintenance)

Other design considerations for upgrading Silverton WWTP to an IFAS system are:

- Some media types require installation of fine screens to protect media from clogging with debris
- Modification of aeration system to provide role pattern aeration (for most media

types)

- Odor control and storage room for removed media (during maintenance)
- Access to media and aeration system

Table 9-9: Required improvements for the IFAS nitrification alternative

	Cost
Hybrid System – 3.5 MGD	$1,100,000

Alternative Analysis

The previous section established two distinct steps in the secondary treatment capacity expansion. The first step includes process control upgrades followed by process optimization, stress testing and a rerating of the secondary treatment capacity. These improvements should be complete before the planning of the next secondary expansion occurs. Presently, the facility is projected to provide sufficient capacity until 2020, and planning for the next expansion should begin by 2015. However, with completion of the Phase 2 Optimization Improvements by the year 2015, the City will have a solid understanding of the true plant capacity and required timeframe for secondary treatment expansion. It is expected that the process control upgrades and process optimization will increase the plant capacity to be sufficient for the 2030 flows and loads. Thus the decision regarding future secondary treatment expansion technologies (conventional treatment, MBR, or IFAS) can likely be deferred until the next facility plan update.

Given that the optimized facility will provide sufficient capacity well beyond the 2030 planning horizon, a recommendation of either technology is difficult at this time since both MBR and Hybrid technology are still emerging. Based on today's state of the technology and current economic parameters, the hybrid technology appears most attractive, followed by the addition of a third conventional treatment train.

The need for reuse quality water alone does not provide sufficient justification for the capital cost of an MBR system. Simple effluent filtration can produce the required effluent quality for reuse at a fraction of the cost.

To provide Silverton WWTP with all options in the future, the site master plan reserves room for a third conventional treatment train, and an MBR system (the hybrid technology would not require additional space).

Effluent Filtration

Currently, Silverton WWTP does not have effluent filtration. The new effluent turbidity requirements; however, may require some form of backup to secondary treatment. In addition, implementation of an effluent reuse program will likely be required to comply with the City's thermal load limit. For planning purposes, this chapter evaluates options for providing effluent filtration to provide 1 MGD of reuse quality water.

Historically, the plant had the occasional effluent turbidity excursion beyond the estimated potential new daily maximum limit (in one instance, even the monthly limit). During the summer permit season excursions beyond the daily limit of 7 ntu are very rare – they have not occurred at all in the last two years (Figure 9-3).

In the past, the facility has not had these effluent requirements. Consequently, there was no reason for operators to react to the increase in effluent turbidity if other effluent parameters were in compliance. Operators do have options to influence the secondary clarifier solids removal and, subsequently, effluent turbidity. They could divert flow to the equalization basin, reduce the RAS rate to reduce clarifier loading, increase effluent DO to minimize floatation due to denitrification, or add a coagulant or polymer to the secondary clarifier to improve the solids retention.

As mentioned in the previous section, consistently lower SVIs would greatly improve the effluent turbidity Figure 9-4 shows that at times of low SVI, the effluent turbidity is consistently very low. Thus, before installing effluent filtration, to meet a future turbidity limit, other measures can be taken to ensure permit compliance. This could also include providing a small on-demand chemical feed system for a coagulant or polymer that could be used to mitigate secondary treatment problems resulting in high effluent turbidity. Therefore this analysis will review effluent filtration options to provide 1 MGD of filtered effluent to both meet potential reuse needs and a potential effluent turbidity limit.

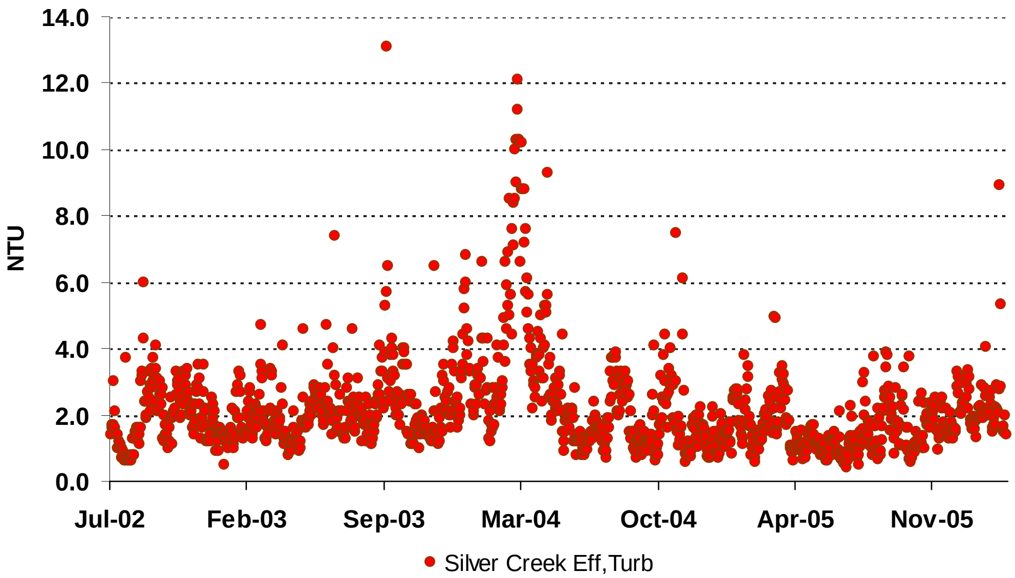


Figure 9-3: Historical Effluent Turbidity

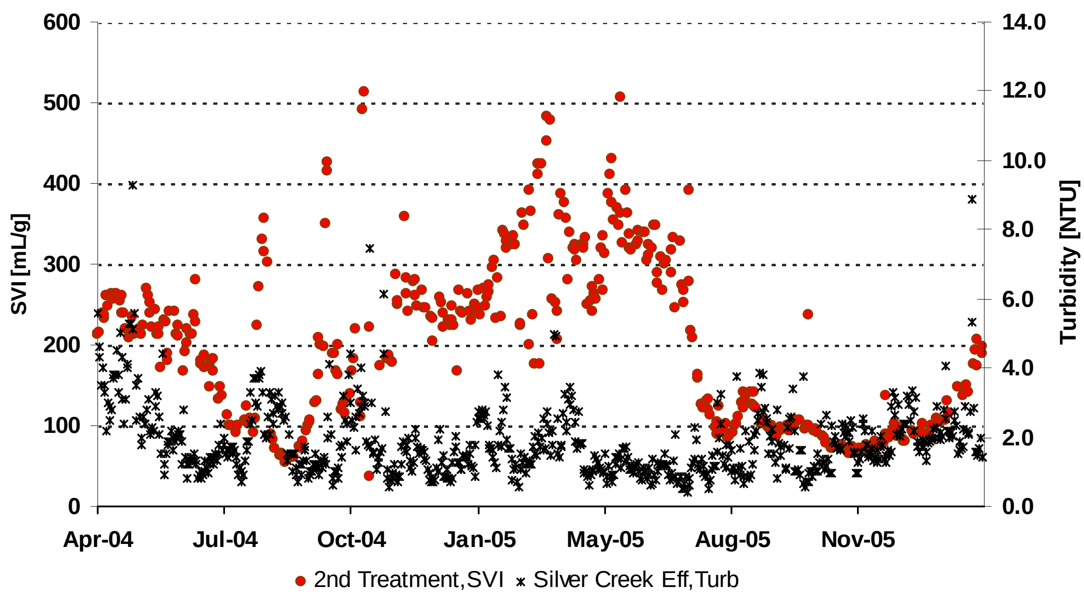


Figure 9-4: Relationship of Effluent Turbidity and SVI at Silverton WWTP

Because of the desire to produce Level IV reuse water for maximum reuse applications, only technologies currently approved by the Department of Health Protection Services for Title 22 applications would be considered for this project. Therefore the following filtration alternatives were reviewed:

- Continuous Backwash Filters:

  DynaSand® filter (Figure 9-5) consists of individual 50 square foot modules which contain approximately 40 inches of sand media. A total of two modules makeup a single filter cell. It is estimated that a total of two filter cells would be required. In the DynaSand® filter, secondary effluent is conveyed upward through the media at a maximum loading rate of 5 gpm/sf. Each of the modules are continuously backwashed by an airlift pump and sand washer and separator. Typically, continuous backwash flows are about 10 percent of the feed flow based on discussions with operators that utilize this technology.

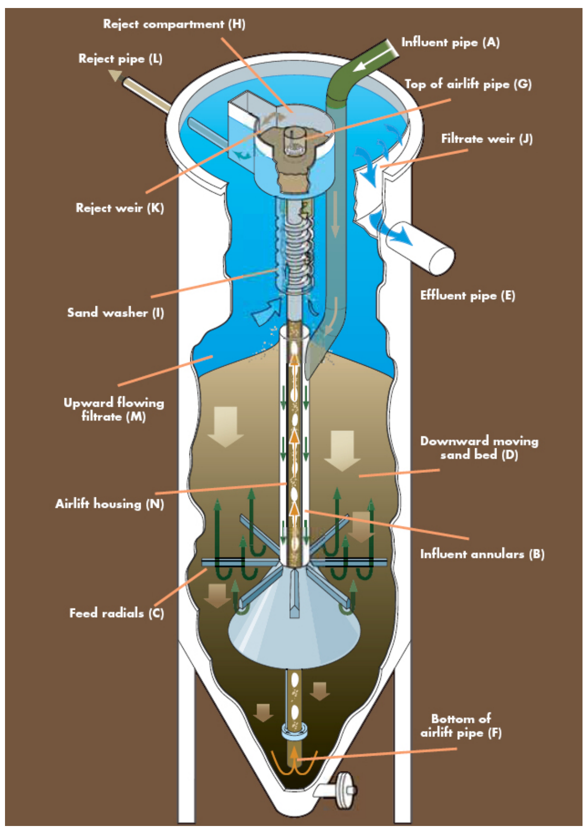


Figure 9-5: Schematic of DynaSand™ continuous backwash filter (Image by Parkson Inc)

- Pulsed Bed Filters:

Hydro-Clear® filters (Figure 9-6) are similar in plan geometry to conventional dual and monomedia filters. However, Hydro-Clear® filters have a shallow bed depth (10 to 12 inches) and use low-pressure air to produce air pulses. Air pulses are used to convey solids from the media surface down into the media thereby regenerating the media surface. Typically, 6 to 10 air pulses occur prior to filter backwashing. Instantaneous backwash demand for this technology is estimated to be 1,320 gpm. The maximum hydraulic loading rates do not exceed 5 gpm/sf.

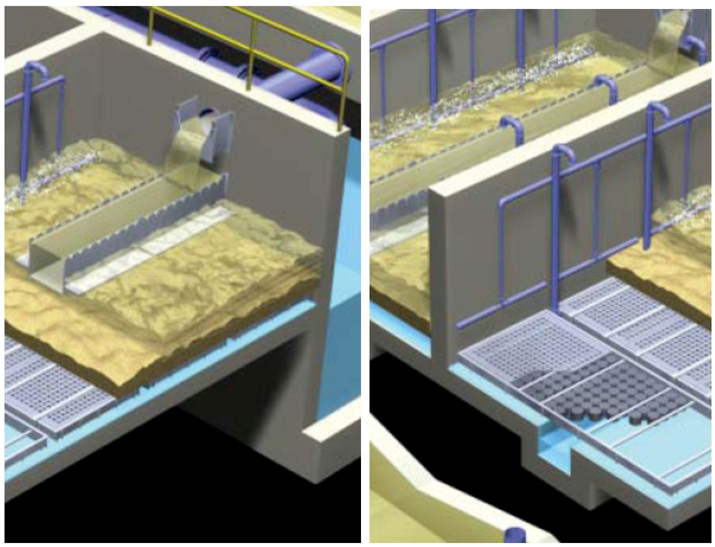


Figure 9-6: Rendering of Zimpro Hydro-Clear™ pulse bed filter (Source: US Filter Product Brochure)

- Cloth Media Disk Filters:

The AquaDISK™ filter (Figure 9-7) utilizes random weave cloth media disks to remove suspended solids and fine particulate matter. The cloth media is approximately 3.8 mm thick and has a nominal pore size of 10 μm, which is considerably less than other Title 22 approved filter technologies. Each disk is comprised of six pie-shaped sections mounted vertically to a common hollow filtrate header tube. The disks are oriented vertically, to provide a relatively large amount of filter surface area within a small footprint area. The filter requires relatively low headloss and is designed to backwash automatically based on water level differential. Besides low backwash demand and production, this technology also offers the benefit of maintaining an almost constant effluent flow rate while backwashing.

Recently, this technology was approved by the Department of Health Services for Title 22 applications provided the hydraulic loading rate does not exceed 6 gpm/sf. Studies conducted by the University of California, Davis found the AquaDISK™ filter to produce effluent turbidities values that were consistently less than 2 NTU for influent turbidities values up to 25 NTU, at filtration rates between 2.5 and 6.5 gpm/sf.

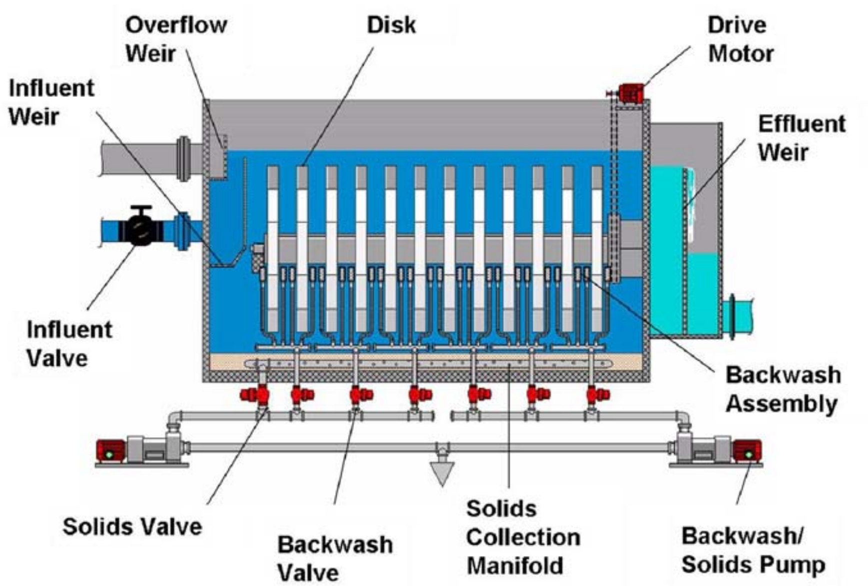


Figure 9-7: Schematic of Dynasand™ continuous backwash filter (image by Aqua-Aerobics)

Figure 9-8 shows a comparison of Title 22 approved filtration technologies. The Aquarist cloth filter appears to perform not only better but also more consistently even with high influent turbidity.

The estimated costs for the different treatment technologies are shown in Table 9-10. The cloth media filter is the least expensive. The O&M costs for all three technologies are very similar ($10,000 - $13,000), and would not change the ranking based on cost. These are capital costs, and do not include engineering or administration fees.

While the cloth media filter appears to be the most cost-competitive, the capital costs are comparable enough that the City could refrain from choosing a desired technology and instead allow the various filter vendors to bid head-to-head. With this approach, it is recommended that the City budget around the median capital cost ($400,000) to provide flexibility in selecting the best filtration equipment.

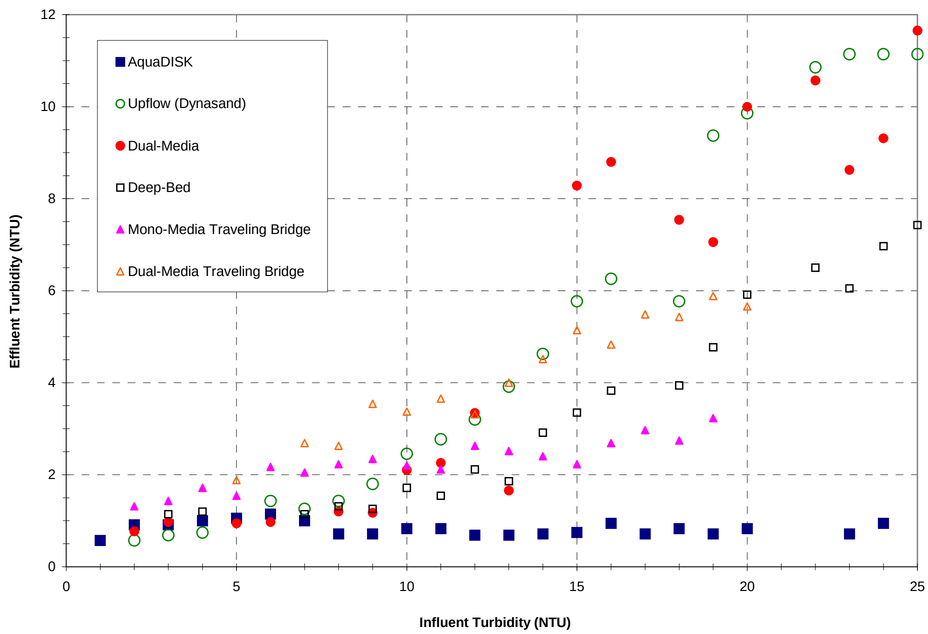


Figure 9-8: Comparison of Influent and Effluent Turbidities for the Various Title 22 Approved Filter Technologies. (Riess, J. et al.) Evaluation of the Aqua-Aerobic Systems Cloth-Media Disk Filter (CMDF) for Wastewater Recycling Applications in California, 4/01

Table 9-10: Estimated Capital cost for effluent filtration

Continuous Backwash (Dynasand™)	$475,000
Pulsed Bed Filter (Hydro-Clear™)	$400,000
Cloth Media Disk Filter (AquaDISK™)	$325,000
Recommended CIP Budget	**$400,000**

Effluent Pump Station

The flood level pumps and equalization basin washdown pumps have sufficient capacity and do not require improvements. The high service pumps that are used to convey effluent to the Oregon garden have a maximum capacity of 600 gpm with one pump running. While modification s could be made to allow both pumps to run at the same time, for redundancy reasons it is recommended to add a third pump and increase the firm capacity to 1200 gpm (1.7 mgd). The cost of this improvement is estimated at approximately $20,000. Additional information regarding the effluent conveyance to the Oregon garden can be found in the Effluent Management section of this chapter.

Solids Treatment

Primary Sludge Pumping

Primary sludge is pumped from the primary clarifiers by a single recessed impeller pump. This pump is located next to the administration building in a wood frame shelter with aluminum siding, as shown in Figure 9-9 and Figure 9-10. The suction line is too long (approximately 80 feet) and the elevation of the pump is too high, potentially causing plugging and cavitation problems. This pump and shelter should be demolished and replaced with a new primary sludge pump station with multiple pumps and should be located closer to the primary clarifiers in an underground vault.

Figure 9-9: Primary Sludge Pump Shelter (Tan Walls with Blue Roof)

Figure 9-10: Primary Sludge Pump

Primary Sludge Grit Removal

Currently, primary sludge is degritted using a single cyclone and classifier, which is shown in Figure 9-11. The equipment is not enclosed and is located adjacent to the anaerobic digesters and gravity thickener. Classified grit is collected in a haul-off container and periodically taken to a local landfill for disposal.

The cyclone was replaced in 1998, but the classifier is corroded and beyond its service life and should be replaced. Consideration should be given to enclosing the process for odor control purposes.

Figure 9-11: Silverton Primary Sludge Degritting Equipment

Sludge Thickening

Currently, primary sludge is thickened by gravity and waste-activated sludge (WAS) is thickened by dissolved air flotation.

Primary Sludge Thickening

A single 20-foot-diameter gravity thickener (Figure 9-12), constructed in 1982, thickens primary sludge to approximately 3 to 4, percent depending on loading and influent solids concentrations. The thickener receives degritted sludge at approximately 0.5 percent solids concentration, and is located adjacent to the anaerobic digesters and primary sludge degritting equipment. The gravity thickener skimmer/sludge collector drive has been recently replaced, and the structure and weir are in adequate condition.

Figure 9-12: Silverton Gravity Thickener

Table 9-11 shows the current and future solids and hydraulic loadings to the gravity thickener. Assuming a primary sludge concentration of 0.5 percent, the gravity thickener is adequately sized for current and future loadings; however, there is currently no backup for primary sludge thickening. A second gravity thickener should be constructed in the future to provide redundancy for primary sludge thickening. For the interim, a spare drive for the primary thickener mechanism should be kept onsite.

Table 9-11: Estimated Loading Rates for Silverton Gravity Thickener*

	Units	Design Criteria	MMWWF 2005	MMWWF Projected 2030
Solids loading rate	lb/sf/d	24	6.4	17
Hydraulic loading rate	gpd/sf	700	152	401

* Assumes primary sludge solids concentration is 0.5%.

WAS Thickening

A single 20-foot-diameter dissolved air flotation thickener (DAFT), constructed in 1998, thickens WAS to approximately 3 to 4 percent, depending on loading and influent solids concentrations. The thickener receives WAS at approximately 5,000-8,000 mg/L solids concentration, and is located near the aeration basins and secondary clarifiers. The DAFT is shown in Figure 9-13.

Figure 9-13: Silverton DAFT

Table 9-12 shows the current and 2030 solids and hydraulic loadings to the DAFT. The DAFT has adequate capacity to handle current and 2030 flows and loads assuming no changes in WAS solids concentration. However, there is currently no backup for WAS thickening. A second backup DAFT is recommended in the future to provide adequate redundancy for WAS thickening.

Table 9-12: Estimated DAFT Loadings for Current and 2030 Projections

	Units	Design Criteria	MMWWF 2005	MMWWF Projected 2030
Solids loading rate	lb/sf/d	24	4.6–4.9[1]	11.1–11.9[1]
Hydraulic loading rate	gpm/sf	2.5	0.075[2]	0.077[2]

1 Range covers various stabilization and dewatering options.

2 Value does not account for recycle.

Recycle of Sidestream Flows

Currently, a single 6-foot-diameter manhole with two submersible pumps returns the following flows to the headworks:

- Gravity thickener overflow
- DAFT underflow
- Drain from grit classifier

- Drains from anaerobic digestion facilities

Plant staff stated that both pumps are running on a relatively continuous basis to match flows into the manhole. Concrete inside the manhole is badly corroded and spalled, as can be seen in Figure 9-14. If a dewatering process is constructed, a larger recycle flow manhole and pumping station would be required to accommodate the increased flows.

Figure 9-14: Manhole and Pump Station for Sidestream Recycle Flows

Sludge Stabilization

Currently, two 30-foot-diameter (81,000 gallons each) anaerobic digesters stabilize thickened primary sludge (TPS) and thickened WAS (TWAS) to Class B biosolids standards. The volatile solids destruction in the digesters averages 60 percent, which is very good performance and is adequate to meet vector attraction reduction requirements. Figure 9-15 shows a photo of the two digesters and the digester control building.

Table 9-13 shows the estimated detention times at current and 2030 design flows and loads. The SRT in the aeration basins is currently being operated as high as 80 days to minimize WAS production to avoid overloading the digesters. The table shows that even if thickening were improved to 4 percent, digesters would still be overloaded, especially if a digester was taken down for cleaning.

Figure 9-15. Silverton Anaerobic Digesters and Digester Control Building

Table 9-13: Anaerobic Digester Analysis

	Units	Design Criteria	2005	Projected 2030
Detention time, MMWWF, all units in service	days	20	13.6-13.8	5.4
Detention time, ADWF, largest unit out of service	days	15	10	3.9-4.0
Volatile solids loading rate, MMWWF, all units in service	Lb VS/d/cf	0.16	0.11	0.27
Volatile solids loading rate, ADWF, largest unit out of service	Lb VS/d/cf	0.24	0.14	0.36-0.38

NOTES:

1. Range covers various dewatering options analyzed.
2. Assumes thickened primary sludge and thickened WAS are both 3% solids.
3. Loadings for current digesters only, not including future digesters.

- Digester Structure

Modern anaerobic digestion facilities are designed with a separate control building to address the current fire code (NFPA 820, 2003). Control buildings house heating equipment and can house gas handling equipment if properly designed. Due to fire code issues, the existing building could not easily house new mixing, heating, and gas handling equipment without a variance from the local fire marshal or appropriate fire code enforcement official.

- Cover

  The existing digesters have floating steel covers that are in fair shape. Plant staff indicate the covers travel up and down with no difficulties.

- Mixing

  Gas mixing systems were popular up until the 1990's when hydraulic/pumped and mechanical mixing technology improved. Gas mixing is not as effective as pumped or mechanical mixing and often exacerbates digester foaming issues. If the existing digesters continue to be operated, it is recommended the gas mixing system be replaced.

- Foaming Issues

  The existing anaerobic digesters have experienced foaming problems in the past. Foaming is typically caused by filamentous bacteria from the secondary treatment system and is difficult to control for plants that nitrify because of low ammonia limits. Some advanced digestion processes such as acid-phase digestion, thermophilic digestion, and temperature-phased anaerobic digestion (TPAD) can mitigate foaming issues.

- Heating

  There is one existing combination boiler and heat exchanger unit for heating both digesters. The unit is sufficient to heat both digesters to 95°F at current loading conditions during winter.

- Recirculation Pumping

  Currently, temporary piping is used for recirculation as the original piping had a long vertical run and the recirculation pumps had air binding problems. This piping should be replaced with a permanent system.

- Gas Handling System

  The existing digester gas flare and gas piping is beyond its service life (installed in 1982) and should be replaced.

Storage

The two solids storage lagoons have a combined capacity of 640,000 gallons and individual surface areas of approximately 8,000 square feet each. An abandoned 100-foot-diameter trickling filter basin is used for additional storage. For average 2005 solids production, the two lagoons provide approximately 230 days at average dry weather flows and approximately 110 days at maximum month wet weather flows. This storage volume would be adequate if Silverton was able to apply biosolids for approximately 5 to 6 months out of the year. It is currently inadequate, however, because biosolids application is limited to a two week period during late summer.

Biosolids Management

Sludge leaving the digesters is stored in one of two sludge ponds or an out-of-service trickling filter. Biosolids are removed on an annual basis (typically during August) for land application on private agricultural land near Silverton. The biosolids land application program is based on having a willing farmer (or farmers) to accept the biosolids; the City does not own the property on which biosolids are applied, nor does the City have formal agreements with the land owners ensuring that sites will be available for future land application. The current biosolids management program is not sustainable and a combination of management and treatment plant upgrades will be recommended in the sections to follow.

Alternatives for Solids Handling and Biosolids Management

Solids Dewatering

The existing plant does not have a solids dewatering process other than the solids lagoons, which do not have adequate storage for seasonal limitations on biosolids land application as discussed in preceding sections. Dewatering will provide the greatest flexibility for on-site solids storage and is recommended due to the currently overloaded and under capacity solids storage lagoons. Several proven solids dewatering technologies are available and are presented below.

Centrifuge

Centrifuge dewatering is based on the application of centrifugal force to digested solids in order to separate as much liquid from the cake as possible. The digester effluent is spun at 1000-4000 rpm in a cylindrical/conical shaped bowl, utilizing the rotational force to pull solids from a liquid centrate. The central bowl contains a conveyor shaft that rotates counter to the centrifugal force, pushing solids toward one end of the unit while centrate is decanted at the opposite end. Polymer is added prior to the dewatering feed to increase the efficiency of the removal process by conglomerating smaller solid particles into larger units. A well designed polymer feed system is essential to dewatering quality. Inadequate mixing, aging, or polymer feed strategies will result in increased costs and lower centrifuge efficiency.

Centrate quality will vary from plant to plant, depending on the degree and type of solids processing prior to dewatering. In the case of Silverton, the post-digestion centrate will likely be high in nitrogen content, particularly ammonia produced during anaerobic digestion. This will have a significant impact on upstream liquid processes to which the centrate stream is recycled and must be considered when implementing such a dewatering program. If raw sludge is dewatered (either primary, WAS, or combined sludges), ammonia recycle is not significant because most of the nitrogen in the sludges is in the form of organic nitrogen.

The primary design criteria for sizing a centrifuge unit are the solids feed rate and concentration. The unit is continuous flow, but is often designed to process a full week of peak month solids production during 1-2 shifts, five days a week. A redundant unit with the same capacity is also assumed to be included in the general design. Thus, accounting for approximately an hour to bring the machine up to speed and slow it down for clean out, the unit can be used over 5-7 days in set shifts, or run continuously for fewer days depending on the upstream sludge storage capacity and the operations staff available.

Centrifuge units can often produce cake solids in excess of 25% for mixtures of anaerobically digested primary sludge and WAS, making them ideal for solids processing prior to composting, lime stabilization, or thermal drying. Advantages of centrifuges include:

- They typically provide higher solids concentrations than belt filter press and screw press units.
- A solid bowl unit typically requires minimal operator attention when running smoothly.
- They typically have a smaller footprint than a belt filter press or screw press.
- The device is easy to clean and can often maintain high solids cake content.
- Multiple commercial vendors are avalable.

Disadvantages of centrifuges:

- Higher polymer dosing may be required.
- They require specialized maintenance.
- They are subject to excessive wear due to grit.
- They are more difficult to monitor; the operator’s view of the centrate and solids is blocked.
- Higher power consumption.

- More noise generation than a belt filter press or screw press.
- They require operating experience to optimize.
- High solids centrifuges have been shown to produce relative high levels of volatile sulfur compounds, which can be a potential source of odor problems during cake storage (WERF, 2003).

Commercially available centrifuge designs include disk nozzle, imperforate basket, and solid bowl. Only the latter can perform acceptably with digested solids and will be considered in this facility plan. Humboldt, Sharples, Alfa Laval, and Andritz are leading manufacturers of solid bowl units.

Figure 9-16 shows a section cut of a typical solid bowl centrifuge. Feed is introduced through the central shaft, centrate exists on the left of the bowl, and solids exit on the right (picture used with the permission of Alfa-Laval).

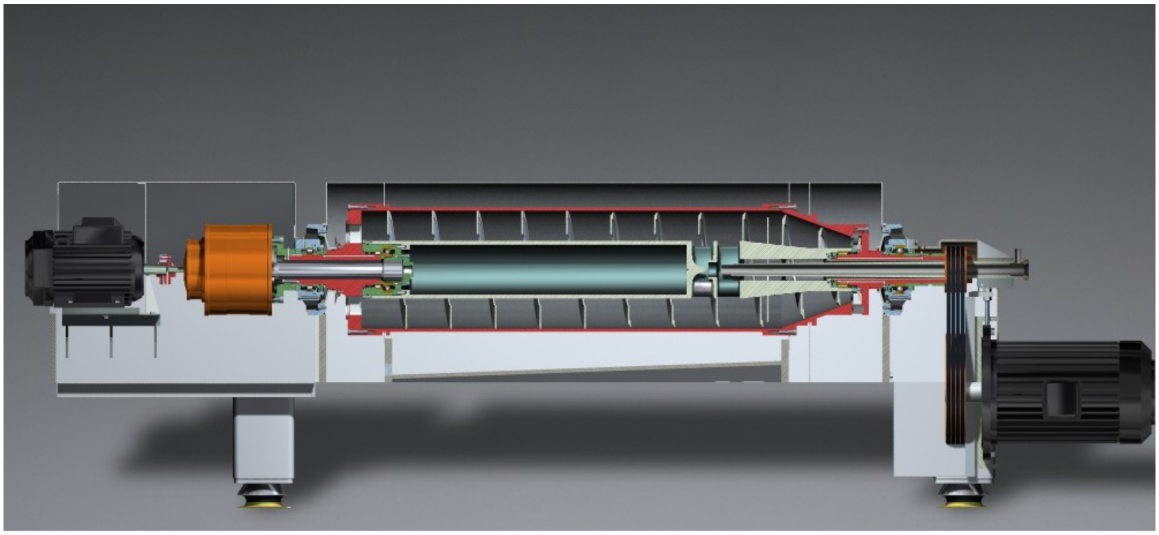


Figure 9-16: Example of a Solid Bowl Decanter Centrifuge

Belt Filter Press

Belt filter press (BFP) dewatering technology operates by applying pressure to solids squeezed between two porous belts. The sandwiched solids are passed between various rollers while maintaining tension on the belt. As with a centrifuge, the sludge is initially conditioned with polymer before passing through a gravity drainage zone, which is essentially a single belt conveyor that allows the initial free water to drain from the solids, producing a 5-10% solids cake. Next, the solids proceed through a low pressure, or “wedge” zone, in which upper and lower belts begin to squeeze the solids as they are passed between various rollers. The pressure is typically 5-15 psi and can be adjusted by regulating the belt tension. Finally, a high pressure zone with multiple rollers completes the dewatering, which in the case of Silverton could likely reach 20% cake solids.

BFPs require continuous wash water during operation (unlike a centrifuge, which does not require continuous wash water) at high pressure to clear solids from the belt as it is recycled through the rollers. Of the three dewatering technologies discussed in this section, BFPs will have the highest amount of centrate/filtrate to recycle to the front of the plant. Wastewater with high levels of oil and grease can blind the belt filter, despite washing, and raw influent must be adequately screened to avoid sharp objects that can damage the belt fabric. Odors can also be a concern as the belts are open, and a BFP facility requires adequate ventilation.

Belt width can range from approximately 0.5-3.5 meters (2 meters is the suggested design for Silverton) and the unit is typically sized according to the solids and/or hydraulic loading. Presses can be operated in a similar manner to a centrifuge (1-2 shifts per day over multiple days to process the solids production for the week), with a single unit sized to handle peak month conditions along with a redundant unit as a backup. Advantages of BFPs:

- Startup and shut-down are more rapid than a centrifuge.
- Less noise generation than centrifuges.
- Maintenance is not as specialized, allowing for plant staff to adequately service the unit and replace belts as needed.
- Multiple commercial vendors are avalable.

Disadvantages of BFPs:

- Highest amount of recycle stream produced.
- Lowest performance for solids concentration.

For these reasons, BFPs will not be considered further for Silverton.

Screw Press

The screw press is a mechanical device used for liquid/solid separation. A cross section of the press is shown in Figure 9-17. 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, as shown in Figure 9-17. The shaft is surrounded by a screen system that contains small (less than 1/8-inch diameter) punched holes. A typical screen is shown in Figure 9-18. The screen support housing includes adjustment nuts to adjust the screen to achieve the proper clearance from the screw flygts. Steam can be provided to the screw in conjunction with lime stabilization system for producing Class A biosolids.

The leading screw press manufacturer in the U.S. can provide screw presses from 4 inch to 53 inches in diameter, with wetted lengths up to 30 feet. The machines are manufactured from stainless steel and are all welded. The base is typically manufactured from carbon steel, but is available in stainless steel by request.

There are several drive systems available, but the most typical has a VFD-driven motor, cyclogear to reduce speed, and a chain drive. This combination provides for a final rotational speed in the range of 0.05 to 1.5 rpm. A typical rotational speed for digested solids is approximately 0.07 rpm.

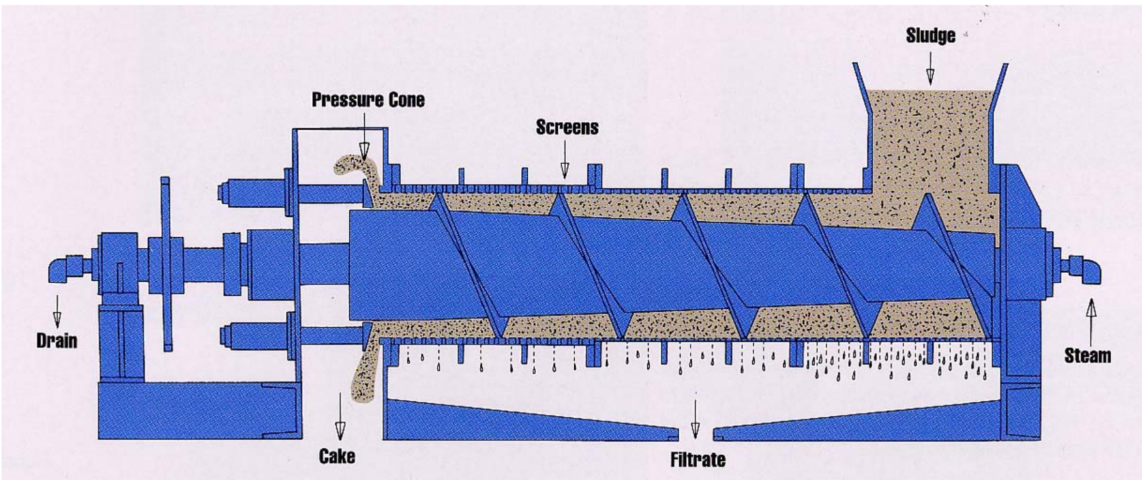


Figure 9-17: Cross-Section Schematic of Screw Press

Figure 9-18: Typical Screw Press Screen

The inlet to the screw press can be piped directly into the press or introduced through the inlet box. In either case, the inlet box is required to allow waste solids to back up into the box, which places a hydraulic head on the material to force it into the screw area.

On-site testing of a screw press design showed that a cake solids concentration of approximately 26-27 percent could be achieved on a combination of primary sludge and WAS, and a concentration of approximately 16-18 percent for WAS only. Testing was not conducted on anaerobically digested solids.

Dewatering Summary

Table 9-14 provides a capital cost comparison of centrifuge and screw press technologies. The centrifuge estimate assumes two units, each capable of handling 2030 design flows, operating 40 hours per week. The screw press assumes two units[5] capable of handling 2030 design flows operating 60 hours per week, as the screw press can be operated unattended. Costs assume that anaerobically digested biosolids will be the influent. The backup plan when the dewatering equipment is out of service is assumed to be pumping liquid biosolids to the storage lagoons.

Table 9-14: Cost Comparison of Dewatering Alternatives — Centrifuge (2), Screw Press (2)

Item	Centrifuge Estimate ($1,000)	Screw Press Estimate ($1,000)
Dewatering Equipment (2 each)	$756	$593
Building	$240	$405
Total Comparative Capital Cost[1]	**$996**	**$998**

1. Costs are comparative and are not representative of a complete dewatering facility.

Table 9-15 provides an annual O&M cost estimation for each system.

Table 9-15: O&M Comparison of Dewatering Alternatives (2006 Dollars)

Item	Centrifuge Estimate ($1,000)	Screw Press Estimate ($1,000)
Power ($0.07/kwh)	$7	$2.2
Labor ($30/hour, 1 FTE)	$78	$62.4
Materials	$10	$1
Chemicals ($2.60/active lb polymer)	$44.1	$44.1

[5] This differs from the quote provided by FKC on June 23, 2005, which was for a single screw press and therefore did not provide redundant equipment.

Total Current Year O&M	**$139.1**	**$109.7**

Table 9-16 and Table 9-17 summarize the qualitative and cost characteristics of the three dewatering options. From a feasibility standpoint, centrifuges and screw presses provide greater flexibility because they can consistently produce cake solids concentrations sufficient to make composting, lime stabilization, or thermal drying viable. A centrifuge requires the smallest footprint and produces the lowest volume of centrate requiring storage and pumping to the head of the plant, an energy savings which partially offsets its relatively higher operating power consumption.

Table 9-16: Comparison of Dewatering Alternatives (1 = Poor, 5 = Excellent)

	Centrifuge	Screw Press
Compatibility with Class A alternatives	5	4
Ease of operation	3	4
Ease of maintenance	2	5
Potential odors	3	4
Power consumption	2	5
Total	**15**	**22**

Centrifuges have higher maintenance requirements than a screw press and the risk of potential odors is high; however, they are similar in cost and performance. For these reasons, a screw press is recommended for dewatering solids at the Silverton plant.

Table 9-17: Cost Benefit Analysis for Dewatering Options[1]

Option	Centrifuge ($1,000(	Screw Press ($1,000)
Present worth[2]	$2,592	$2,256
Benefit score	15	22
Cost benefit ratio	$173	$103

1 Represents 2006 dollars.

2 Based on a 6% discount rate. Based on comparative costs from Table 9-14 and Table 9-15 only.

Depending on the solids stabilization process, the sidestream from the dewatering process may present some unique challenges. Sidestreams from the dewatering of anaerobically digested solids typically contain high levels of ammonia nitrogen. As described earlier, recycling of this dewatering sidestream adds a significant load to the secondary treatment system, which can require additional capacity. In addition, daytime dewatering operations can cause a spike in ammonia load (as much as twice the loads normally encountered during diurnal fluctuations). Because of this, storage is typically provided to recycle the sidestream during the night or other off hours to minimize this peak. Costs for sidestream storage and recycle pumping are included in the evaluation of solids stabilization alternatives.

Solids Stabilization, Storage, and Management Alternatives

The most critical element of the solids handling process that requires improvements is the solids stabilization system. Three alternatives will be considered. Other solids stabilization options are available, such as composting. However, based on experience, they are less cost-effective and desirable for Silverton than the three alternatives presented below. The three selected alternatives are summarized below and described in the following sections.

Alternative 1:

- Continue and expand anaerobic digestion process.
- Construct a liquid biosolids storage tank.
- Construct a solids dewatering process.
- Construct additional solids storage facilities.
- Expand biosolids customer base so that biosolids can be applied for 5-6 months of the year.
- Construct a Class A pasteurization or thermal drying system in the future.

Alternative 2:

- Abandon the existing anaerobic digestion process.
- Provide a thickened sludge blend tank.
- Construct a solids dewatering process.
- Make provisions for hauling to the local solid waste incinerator.
- Construct a Class A lime stabilization system and limed biosolids storage area.
- Expand biosolids customer base (include the public) so that biosolids can be applied for 6 months of the year or more.

Alternative 3:

- Continue and expand anaerobic digestion process.
- Construct a liquid biosolids storage tank.

- Construct a solids dewatering process.
- Construct additional solids storage facilities.
- Expand biosolids customer base so that biosolids can be applied for 5-6 months of the year.
- Construct a Class A thermal drying system.

Alternative 1: Anaerobic Digestion, Dewatering, Cake Storage, Land Application

Because the two existing digester structures are in good condition, it is assumed they will continue in operation. For this alternative (Figure 9-19) the gas handling equipment, mixing equipment, and covers should be replaced if anaerobic digestion is continued. An additional digester is required to meet the design criteria for 2005 flows. A second digester/storage tank should be constructed as flows are predicted to rise in the immediate future. It is recommended that two 40-foot diameter digesters be constructed and one of the existing digesters be converted to a liquid biosolids storage tank. Storage prior to dewatering is required to equalize the fluctuations caused by shift dewatering operations and provide a buffer for unscheduled maintenance events.

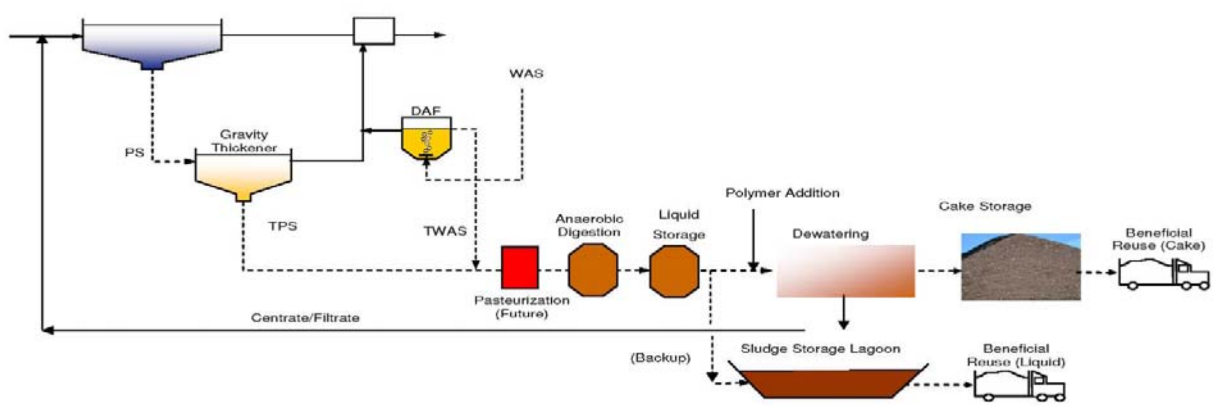


Figure 9-19: Process Schematic, Solids Processing – Alternative 1

A new digester control building would be required to house digester heating, gas handling, and pumping equipment. The current fire code (NFPA 820) specifies that equipment and facilities within 10 feet of the wall of an anaerobic digester have explosion-proof motors and electrical equipment; therefore, it is more cost effective to construct a separate building to house this equipment. Gas storage would be provided by gas holder covers on the existing digesters, including the one converted to a liquid biosolids storage tank.

Dewatering

A separate dewatering facility would be constructed to reduce the volume of solids to be stored and land applied. Pressate equalization tanks and recycle pumping is required to minimize the impact on secondary treatment.

Cake Storage

Oregon DEQ requires 180 days of storage unless an adequate means of managing biosolids is in place for winter application. A dewatered solids cake storage facility is recommended unless Silverton enters into a long term contract for biosolids management with a private company or farmer for land application during the wet season. A minimum of seven days storage is still recommended if road conditions do not allow hauling to land application sites. This could be provided in two 20-cubic-yard-capacity trucks at the plant for 2005 maximum month wet weather flows.

Biosolids Management

Silverton's current biosolids customer should be informed of the change in biosolids characteristics due to dewatering. Additional customers should be developed to allow biosolids land application from spring to fall. A long term contract with a private biosolids management company or farmer should be seriously considered to provide alternative means of managing biosolids during the wet season. This will minimize storage requirements and provide flexibility to the biosolids management program.

Dewatered cake application equipment will need to be purchased or a contractor with this type of equipment will need to be procured. Table 9-18 shows the opinion of probable costs for Alternative 1.

Table 9-18: Opinion of Probable Cost for Solids Processing Alternative 1

Item	Capital Cost ($1,000)
Phase I	
One 40-foot-diameter anaerobic digester and digester control buildings	$2,035
Engineering, Administrative, and Legal (30% of Construction)	$610
Total Phase I Project Costs	**$2,645**
Phase II	
One 40-foot-diameter anaerobic digester and rehabilitation of two existing digesters	$1,801
Dewatering facility	$1,527
Biosolids storage facility	$1,082
Odor control	$624
Total Phase II Construction Costs	$5,034
Engineering, Administrative, and Legal (30% of Construction)	$1,510
Total Phase II Project Costs	**$6,544**

Alternative 2: Thickened Sludge Blending, Lime Stabilization, Dewatering, and Storage

Alternative 2 involves construction of a lime stabilization system of sufficient capacity to process the design maximum 2030 sludge flow and load. Only systems capable of producing Class A biosolids will be considered. Compliance with Part 503 Class A biosolids requirements means the pH of the solids must be greater than 12 for 72 hours and the temperature must be above 52°C (126°F) for at least 12 of the 72 hours (EPA, 1999). The solids must then be air dried, typically in windrows, to more than 50 percent solids. Alternatively, the requirements of Part 503 Class A Alternative 1 (time-temperature) can be met for compliance. Typically, this means that the solids are held for at least 30 minutes at temperatures no lower than 70°C (158°F).

When added to sludge, lime reacts with the water and releases a tremendous amount of heat (exothermic). Lime could be added to either liquid sludge or dewatered cake. Addition to dewatered cake (sometimes referred to as "postlime stabilization") is preferred due to reduced lime requirements. A schematic of Alternative 2 is shown in Figure 9-20.

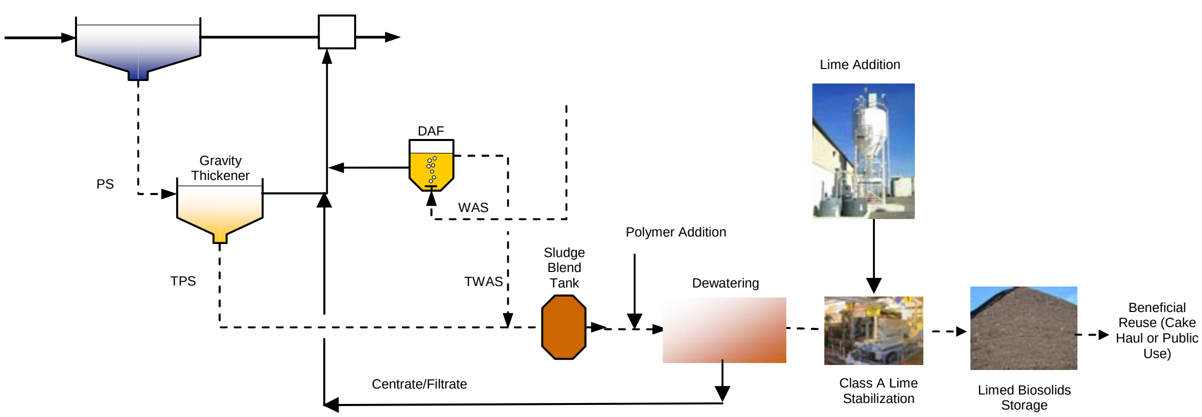


Figure 9-20: Process Schematic, Solids Processing Alternative 2

One train of lime stabilization/pasteurization is sufficient, as long as spare parts and an adequate service agreement are also provided. The odor control system servicing the lime stabilization system must be capable of handling the odors produced during the processing of raw sludge.

There are several commercially-available systems capable of producing Class A biosolids. Several leading systems are listed and described in Table 9-19. As shown in Table 9-19, RDP and FKC are the only manufacturers with Class A lime stabilization systems installed in the Pacific Northwest. Figure 9-21 shows a lime stabilization facility in Newport, Oregon.

The facilities would include the following:

- Lime stabilization building.
- Lime stabilization equipment.
- Lime pumps.
- Biosolids loadout.
- Odor control equipment.

Table 9-19: Leading Class A Lime Stabilization/Pasteurization Systems

Manufacturer	System	Advantages/ Disadvantages	Installations in Pacific Northwest
RDP Technologies, Inc.	EnVessel Pasteurization™	Time-temperature used to meet Class A requirements, windrow drying not required Heat added to reduce lime requirement, lower operating costs	Centralia, Washington (construction) Chehalis, Washington Newport, Oregon Victoria, Australia Kelso, Washington (design)
N-Viro	N-Viro Soil Process	High quality product Requires windrow drying to 50% solids	None
BIOSET®	BIOSET Process	Requires acid addition	None
FKC		Requires screw press dewatering (proprietary system) Uses anhydrous lime, which is currently used at the plant Requires lime mixing with sludge prior to dewatering	Sedium, Washington

It is assumed a lime stabilization building would be constructed directly south of the existing anaerobic digesters. The building would include the selected dewatering equipment to minimize conveyance of dewatered biosolids (cake) and centrally locate both the dewatering and lime stabilization equipment in one building. The approximate footprint for this building would be 3500 square feet.

The market for Class A lime stabilized cake is still developing in the Pacific Northwest. Based on recent experience in Centralia, Washington and Newport, Oregon, cake could be marketed to topsoil manufacturers, farmers, or local fertilizer brokers contingent on low odors and consistent aesthetic characteristics.

Figure 9-21: Lime Stabilization Facility in Newport, Oregon

Lime-stabilized cake is also well suited to reclamation sites as it provides the ability to raise the pH of acidic soils. If Silverton selects this biosolids management alternative, a detailed marketing study should be performed to identify potential customers and revenue that could be generated from the final product. Preliminary indications from Centralia, Washington show there are opportunities to market the lime-stabilized product locally to topsoil manufacturers and fertilizer brokers, especially during summer months.

Limed Biosolids Storage

Although Class A lime-stabilized biosolids is a valuable fertilizer in western Oregon, adequate storage for limed biosolids should be provided for the wet season. A minimum of 120 days of storage should be provided in a covered facility with adequate access and drainage.

Biosolids Reuse

Given the acidic soils in the Willamette Valley, limed stabilized biosolids can provide both pH adjustment and organic fertilizer. As such, it would be a valuable commodity. A significant amount of education and marketing effort would be required; however, before adequate demand for the product developed. Silverton's existing customer, as well as other local farmers, should be surveyed to confirm the value of limed stabilized biosolids.

If the City desired, a public give-away or revenue generation program could be developed as the product would have unrestricted use. Even more effort in marketing and education would be required for this type of program. To spread the costs of improvements out, a two-phased approach should be taken. Table 9-20 shows the opinion of probable costs for Alternative 2.

Table 9-20: Opinion of Probable Cost for Solids Processing Alternative 2

Item	Capital Cost ($1,000)
Phase I	
Thickened sludge blend tank (conversion)	$374
Dewatering and lime stabilization facility (Dewatering facility only)	$2,386 ($1,527)
Odor control (assume 3-stage chemical scrubber)	$624
Recycle Manhole and Pumping	$305
Total Construction	$3,690
Engineering, Administrative, and Legal (30%)	$1,107
Total Phase I Project Costs	**$4,796**
Phase II	
Limed biosolids storage facility	$1,096
Primary sludge pump station	$420
Grit classifier replacement	$159
Total Construction	$1,675
Engineering, Administrative, and Legal (30%)	$503
Total Phase II Project Costs	**$2,178**

Notes: A dewatering facility without lime stabilization would require another method of meeting Part 503 requirements for solids stabilization. Additional costs for this option are not shown.

Alternative 3: Anaerobic Digestion, Dewatering, Drying

Alternative 3 is the same as Alternative 1 except for storage requirements and the construction of a thermal drying system.

Thermal Drying

A schematic of the Silverton solids handling process with a thermal dryer is shown in Figure 9-22. There are several commercially-available systems capable of producing Class A biosolids. Fenton Environmental Technologies, Komline-Sanderson, and Andritz are the leading manufacturers with thermal drying equipment for small-to-medium sized WWTPs. They are also the most economical units on the market at this time for facilities the size of Silverton.

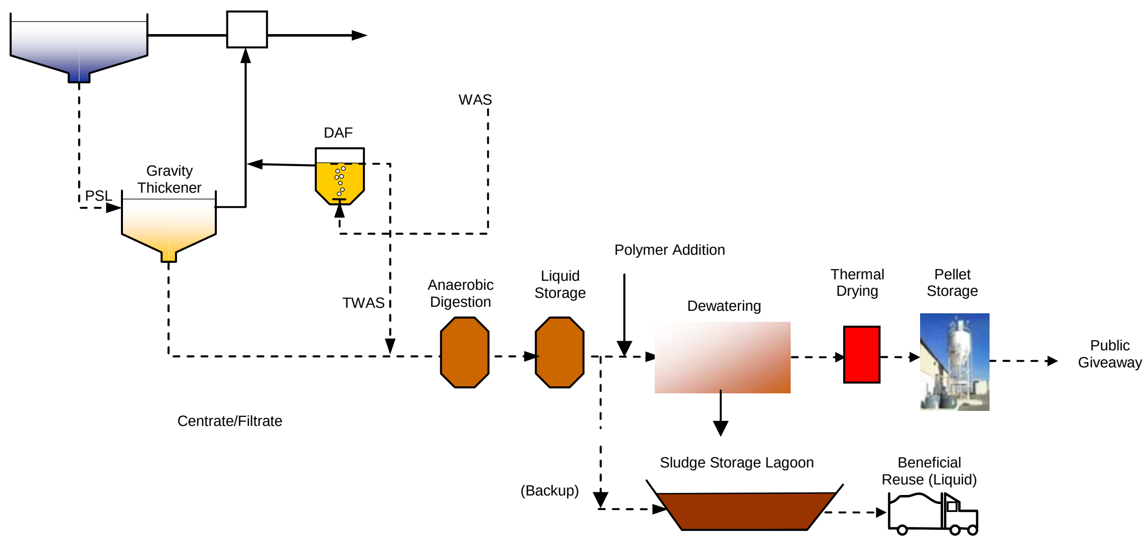


Figure 9-22: Schematic of Solids Processing Alternative 3

The best location for a thermal dryer is just west of the digester complex adjacent to a new dewatering facility. A building would be required to protect the equipment and attenuate the noise generated. Dewatered cake would discharge into a dryer feed hopper via a screw conveyor. Table 9-21 shows the opinion of probable costs for Alternative 3.

Table 9-21: Opinion of Probable Cost for Solids Processing Alternative 3

Item	Capital Cost ($1,000)
Two 40-foot-diameter anaerobic digesters and digester control buildings	$2,909
Rehabilitation of existing digesters	$717
Dewatering facility	$1,527
Biosolids drying and storage facility	$3,305
Odor control (assume 3-stage chemical scrubber)	$624
Total Construction	$9,083
Engineering, Administrative, and Legal (30%)	$2,725
Total Project	$11,807

Dried Pellet Storage

Storage of dried biosolids would be provided in a hopper. Careful attention to hopper design and dryer operations are required to minimize the risk of combustion of the dried product.

Reuse

Dried biosolids are a valuable fertilizer for farmers, golf courses, and other fertilizer users. The market for dried biosolids is not well established in the Pacific Northwest, but effort in education and marketing could develop demand for the product. Biosolids drying concentrates metals as some of the organic material is combusted during the drying process. A backup plan must be in place in the event of dryer downtime. Class B dewatered cake land application would be an appropriate backup as long as customers and permitted land application sites are maintained.

Alternatives Analysis

The discussion below outlines key assumptions related to each alternative, summarizes biosolids quantities produced under each alternative, and compares cost and non-cost factors related to each alternative. Detailed cost estimates for the alternatives are included in Appendix E

1. **Alternative 1:**

- Two 40-foot diameter, 25-foot side water depth digesters would be constructed immediately. New digesters would be cast-in-place concrete and geotechnical conditions would not require pilings.

- The digester control building would be approximately 900 square feet.
- Gas handling and mixing equipment, and covers on the existing digesters would be replaced. Pumped mixing and gas holder covers would be provided on both existing digesters. Fixed steel covers with pumped mixing would be provided on the new digesters.
- One screw press would be provided in the first phase of construction. Liquid biosolids would be stored in the existing lagoons in the event the screw press was out of service.
- A covered cake storage facility would be approximately 1,750 square feet, which would provide 120 days of storage at 2005 maximum month wet weather flows.

2. Alternative 2:

- Existing digesters would be converted to thickened sludge blend tanks. Pumped mixing systems would be added to the tank, which would be partitioned to minimize the detention time in the tank.
- Gas handling and mixing equipment on the existing digesters would be removed.
- One screw presses would be provided in the dewatering and lime stabilization facility.
- A covered limed biosolids storage facility would be approximately 2,800 square feet, which would provide 120 days of storage at 2005 maximum month wet weather flows, and about 48 days at 2030 maximum month wet weather flows. Depending on how the market develops, additional storage may be necessary as flows and loads increase.

3. Alternative 3:

- Two 40-foot diameter, 25-foot side water depth digesters would be constructed immediately. New digesters would be cast-in-place concrete and geotechnical conditions would not require pilings.
- The digester control building would be approximately 900 square feet.
- Gas handling and mixing equipment, and covers on the existing digesters would be replaced. Pumped mixing and gas holder covers would be provided on both existing digesters. Fixed steel covers with pumped mixing would be provided on the new digesters.

- One screw press would be provided in the first phase of construction. Liquid biosolids would be stored in the existing lagoons in the event the screw press was out of service.
- One thermal dryer with a capacity for 2030 MMWWF would be provided.
- The dewatering and drying facility would be approximately 2,200 square feet.
- A dried biosolids storage silo would provide 90 days of storage at 2005 maximum month wet weather flows.
-

4. Operations and Maintenance Costs:

- Electricity: $0.07/kW-hr
- Natural gas: $0.74/therm
- Lime: $80/ton
- Lime Dose: 400 lb/dry ton
- Polymer: $2.60/lb active
- Polymer Dose: 20 lb active/dry ton
- Labor Rate (Loaded): $30/hr
- Materials: 1% of equipment costs
- Approximate Management Costs:
- Dewatered cake biosolids (Alternative 1): $80/dry ton.
- Lime stabilized biosolids (Alternative 2) and the dried biosolids (Alternative 3): $20/dry ton.

Comparisons of all three alternatives are shown in the tables below. Projected biosolids quantities produced at 2005 and 2030 are shown in Table 9-22. Alternative 2 will produce substantially more solids due to the solids not being digested and the precipitation of the added lime. However, the volumes produced by the lime stabilization process will not be directly proportional to the additional solids produced due to increased dryness of the solids. Alternative 3 would produce substantially less volume of solids due volume of liquid removed during the drying process.

Operations and maintenance cost estimates are presented in Table 9-23 Table 9-24 presents a life cycle cost comparison; Table 9-25 presents a non-cost evaluation of the alternatives; and Table 9-26 shows a cost benefit analysis.

Table 9-22: Biosolids Quantities Produced for Three Alternatives

Design Condition	Alternative 1	Alternative 2	Alternative 3
2005 ADWF (lb/d)	1,137	2,578	1,137
2030 ADWF (lb/d)	2,881	6,511	2,881
2005 ADWF (cf/d)	79.2	125.2	19.8
2030 ADWF (cf/d)	200.7	316.1	50.2

1. Assumes no combustion of solids during drying.
2. Assumes dried biosolids have a solids concentration of 92%.

Table 9-23: Operations and Maintenance Cost Estimates for Biosolids Management Alternatives

	Alternative 1 ($1,000)	Alternative 2 ($1,000)	Alternative 3 ($1,000)
Power	$26.5	$13.9	$32.3
Fuel	$1.7	$3.4	$28.1
Chemicals	$47.6	$71.3	$47.6
Labor	$53	$84.2	$87.4
Materials	$23.6	$14.2	$36.1
Biosolids Management	$29.3	$16.6	$7.3
Sum	**$182**	**$204**	**$245**

Table 9-24: Life Cycle Cost Comparison of the Three Alternatives

	Alternative 1 ($1,000)	Alternative 2 ($1,000)	Alternative 3 ($1,000)
Total Capital cost of all phases	$9,402	$6,397	$12,020
Annual O&M cost	$182	$204	$245
Life cycle cost	$11,273	$8,655	$14,615

Table 9-25: Non-Cost Analysis of the Three Alternatives

	Alternative 1	Alternative 2	Alternative 3
Impact on liquid stream	1	3	1
Biosolids Marketability	1	2	3
Ease of O&M	3	2	1

Reliability	3	2	1
Regulatory Sensitivity	1	2	3
Potential Odors	2	1	3
Sum	**11**	**12**	**12**

Table 9-26: Cost Benefit Analysis of the Three Alternatives

	Alternative 1	Alternative 2	Alternative 3
Life cycle cost	$11,273	$8,665	$14,615
Benefit score	11	12	12
Cost benefit ratio	$1,025	$722	$1,218

Recommended Solids Handling Improvements

Based on the analysis above, Alternative 2 (screw press dewatering with lime stabilization) provides the greatest benefits to the City at the lowest capital and life cycle cost. The capital cost is still significantly higher than the budget currently allocated in the City's capital improvement plan, so two modifications to this alternative were included as part of the Recommended Plan. These modifications include constructing a covered, open-air limed biosolids storage facility using the City's existing sludge storage lagoons, and identifying other potential disposal sites to avoid short-term capacity limitations while funding is obtained.

It is recommended that the City proceed immediately with design and construction of improvements related to increasing the biosolids treatment capacity. Additional improvements related to equipment condition and reliability (e.g., primary sludge pumping and grit classifier improvements) can be deferred to a later phase if funding is not available to include these elements in the initial solids handling expansion.

Initial Biosolids Storage Improvements

Temporary limed biosolids storage will be provided by retrofitting one existing biosolids storage lagoons to be capable of storing limed and dewatered biosolids. A lightweight, open-frame building (such as that made by Cover-all, Inc.) should be constructed over one of the existing sludge storage lagoons. Roof drainage needs to be directed away from the storage lagoon, preferably to a nearby storm drain. The lagoons are concrete lined and have slopes at an angle that allows driving a vehicle in and out. A front-end loader or dump truck would collect biosolids from the discharge screw conveyor of the Dewatering and Lime Stabilization Building and move it to the storage area. A front end loader or dump trucks could also be used to unload and remove biosolids from the storage area.

The total project cost associated with this storage facility (including contingency, administration, and engineering) is approximately $400,000, representing a significant savings over the $1.1 million required to construct an enclosed biosolids storage facility. The open-frame facility does have greater potential to create offsite odors than an enclosed facility, with odor potential increasing in relationship to the length of storage. If the City is able to implement a limed biosolids reuse program that results in minimal storage time, it may be possible to avoid the need for enclosed biosolids storage in the future.

Temporary Biosolids Disposal Options

The Marion County waste-to-energy (incineration) facility was contacted regarding the potential to accept Silverton's dewatered biosolids. After internal discussions, staff indicated that biosolids were not accepted at this facility due to operations and maintenance issues (see Appendix B for correspondence). Discussions with local landfills were held regarding disposal of Silverton's sludge/biosolids. Only Coffin Butte landfill (near Corvallis, OR) indicated that biosolids would be accepted pending analytical data from dewatered cake solids (see Appendix C for data required). Tipping fees were not obtained as the landfill requires analytical data before quoting a price. Coffin Butte officials would not state whether or not raw sludge could be accepted without analytical data and a sample of the material. A dewatering process is required for landfilling sludge/biosolids, but it is possible that other plant improvements could be avoided pending further analytical evaluation and subsequent discussions with Coffin Butte.

Discussions with Silverton's current biosolids customer regarding the change in product from liquid to dewatered cake should be started immediately. Additional customers should be sought to allow the City to apply biosolids for a longer time period during the dry season and provide more flexibility for the biosolids management program. The recommended capital improvements are presented in Table 9-27.

Table 9-27: Recommended Capital Improvements for Silverton Solids Processing ($1,000)[1]

Improvement	Cost
Phase 1a	
Thickened Sludge Blend Tanks	$374
Dewatering and lime stabilization facility[2]	$2,386
Covered Limed Biosolids Storage	$342
Odor control (assume 3-stage chemical scrubber)	$624
Recycle Pump Station	$305
Total Phase 1a Construction Costs	**$4,032**
Engineering, Administrative, and Legal (30%)	$1,175
Total Phase 1a Project Costs	**$5,027**
Phase 1b	

Primary Sludge Pump Station	$420
Grit Classifier Replacement	$159
Storage	$1,096
Total Phase 1b Construction Costs	**$579**
Engineering, Administrative, and Legal (30%)	$174
Total Phase 1b Project Costs	**$753**

1. Costs include engineering, administrative, and legal costs (estimated at 20% of construction cost for biosolids storage facility; 30% of construction costs for other improvements)
2. $1,527 of this cost is attributable to the dewatering facility only.

Laboratory and Administrative Facilities

As noted in Chapter 7, improvements to the lab and administrative building are required to support the staff functions required for efficient long-term operation and maintenance of the treatment plant. Recommended improvements include:

- Adding a new laboratory space with a dedicated HVAC system
- Remodeling the existing laboratory to provide office space for operations and records storage
- Providing new male and female locker room facilities

The cost of these improvements was estimated assuming the total area for new and renovated facilities would be approximately 1,000 square feet. The estimated project cost is $300,000, including contingency, engineering, and administrative fees. The City should initiate this project with a Schematic Design effort to determine specific facility needs, adjacencies, and layout.

Effluent Management

As described in Chapter 5, the City's NPDES permit establishes excess thermal load limits based on the allowable temperature impacts outlined in the Oregon Temperature Standard (OAR 340-041-0028). These limits become effective upon expiration of the permit or completion of the Molalla-Pudding TMDL.

The City has been monitoring effluent temperature, and recognizes that compliance with the current excess thermal load limit will be challenging. Daily maximum effluent temperatures have been recorded since mid-September 2005. They indicate that during the warmest parts of the summer, the effluent temperature significantly exceeds the stream criteria. Table 9-28 below shows the excess thermal load discharges to Silver Creek based on daily maximum temperature data through June 8, 2006.

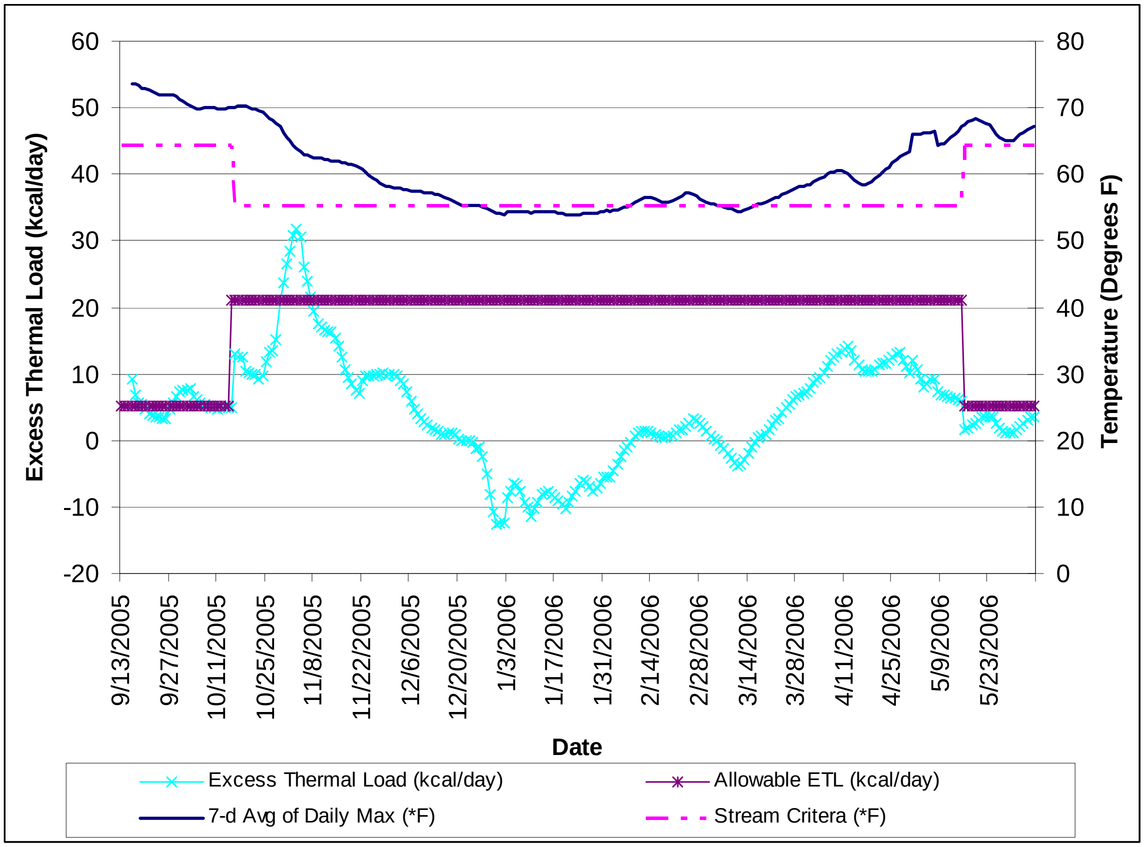


Figure 9-23: Excess Thermal Load Discharge to Silver Creek

Future Thermal Load Evaluation

Analysis of projected future thermal discharges is based on:

- Projected maximum month WWTP flows at the end of the planning period
- Projected flows to the Oregon Garden
- Assumed effluent temperatures (7-Dday Average of Daily Maximum, 7DADM)

Projected Maximum Month Flows

Projected thermal loads are based on a maximum month flow of 2.3 MGD for the period of May through October, and 4.2 MGD for the period of November through April.

Projected Flows to the Oregon Garden

In recent years, the City has operated the treatment system to maximize flow to the Oregon Garden. As shown in Figure 9-24 below, flow to the garden has averaged 0.4 – 0.8 MGD since spring 2004.

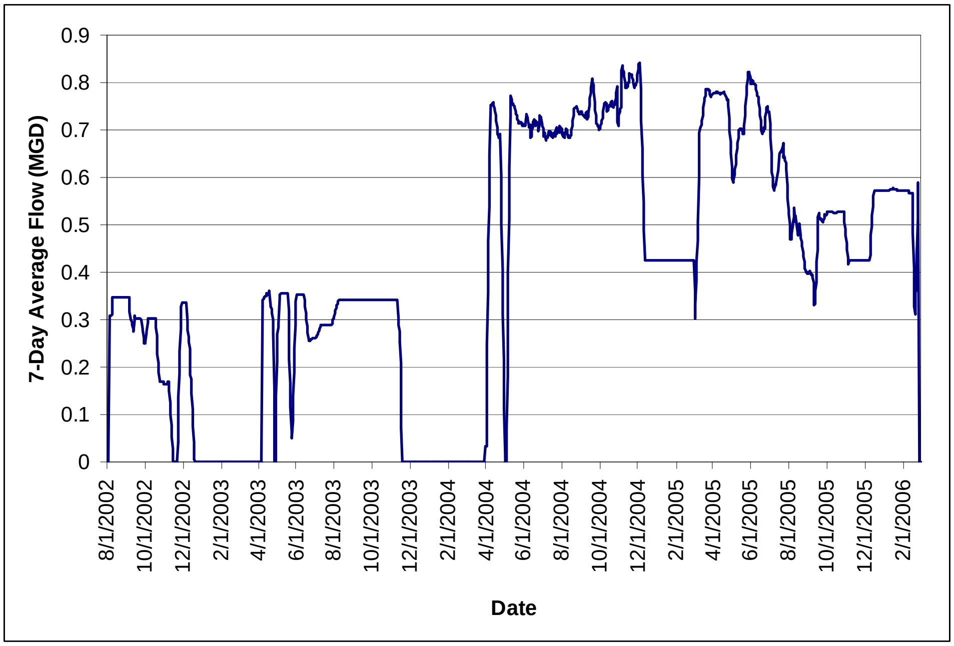


Figure 9-24: Historical Flow to Oregon Garden

The City and the Oregon Garden jointly developed a plan describing how treated effluent will be provided from the City to the Garden (Oregon Garden Foundation/City of Silverton Oregon Garden Water Management Plan, HDR Engineering, November 2000). This plan outlines the anticipated month-by-month effluent quantities to be accepted by the Garden. The monthly flows were based on the City's previous NPDES permit, which limited the discharge volume to Silver Creek to a three-month average (July through September) of 1.0 MGD. The document indicates the Oregon Garden will accept up to 120 percent of the monthly flows outlined in the plan. Table 9-28 below shows allowable discharges based on the Water Management Plan, average historical discharges, and recommended discharge rates based on agronomic irrigation of current and planned plantings at the Garden and related facilities. Future discharge rates are based on the following assumptions:

- The Oregon Garden will continue to develop under new ownership, with additional plantings on the current 80-acre site resulting in additional irrigation demand. This demand is assumed to grow to projected 2015 demands detailed in the Water Management Plan.

- The development of a destination hotel on an 11-acre site overlooking the Garden will be irrigated from existing effluent-supplied ponds at the Garden. It is assumed that fifty percent of the site will require irrigation at typical landscape irrigation rates of 4-9 inches/month during the period of May through September.

-

- Based on these assumptions, monthly flows to the Oregon Garden are anticipated to be as follows (Table 9-28):

Table 9-28: Assumed Flow to the Oregon Garden

Month	Projected 2015 Demand at Garden (MGD)	Garden Hotel Irrigation Demand (MGD)	Total Flow to Garden (MGD)
January[1]	0.50		
February[1]	0.50		
March[1]	0.50		0.40
April[1]	0.50		0.40
May[1]	0.50	0.02	0.40
June[2]	0.67	0.03	0.40
July[2]	1.01	0.04	0.52
August[2]	1.00	0.03	0.70
September[2]	0.92	0.02	1.05
October[2]	0.43		1.04
November[1]	0.50		0.94
December[1]	0.50		0.43

[1] Based on historical discharge

[2] Based on Oregon Garden Water Management Plan

The maximum pumping rate to the Oregon Garden is 600 gpm (0.87 MGD); therefore, a third pump must be added to the effluent pump station to provide the anticipated total flow to the Garden.

Monthly Effluent Temperature

Monthly effluent temperatures for use in future excess thermal load calculations were selected as the maximum values from the running 7DADM temperatures at the treatment plant from September 13, 2005 through June 10, 2006. Monthly temperatures were used rather than seasonal temperatures so that allowable discharges could be correlated with the Oregon Garden Plan. In addition, monthly temperatures allow consistency with DEQ's current approach of establishing monthly thermal load limits in the Willamette Basin. Unfortunately, the available data did not include the months of July, August, and early September – times when effluent temperatures are typically high. Therefore, the highest maximum 7DADM from the dataset (observed during September) was used for the months of July, August, and September.

Table 9-29: Future 7DADM Temperatures

	Maximum 7DADM Temperature (°C)	Maximum 7DADM Temperature (°F)	Stream Criteria (°F)
September	23.1	73.6	64.4
October 1-15	21.6	70.9	64.4
October 16-31	21.2	70.2	55.4
November	18.3	64.9	55.4
December	14.5	58.0	55.4
January	12.5	54.5	55.4
February	14.0	57.2	55.4
March	14.6	58.3	55.4
April	17.4	63.3	55.4
May 1-15	19.5	67.1	55.4
May 16-31	20.2	68.3	64.4
June	19.9	67.8	64.4
July	23.1	73.6	64.4
August	23.1	73.6	64.4

Projected Future Excess Thermal Loads

Based on the data and assumptions described above, projected future excess thermal loads were calculated on a monthly basis. Excess thermal loads are based on the projected 2030 flows to the treatment facility, which as described in Chapter 3, are lower than the current design flows. Table 9-30 identifies projected excess thermal loads, and also shows the additional flow that must be diverted from Silver Creek to meet the excess thermal load limits in the City's current NPDES permit.

Table 9-30 highlights two important conclusions:

- At projected 7DADM temperatures, some additional flow diversion or cooling will be required to meet the existing thermal load limits during all but the coolest months of the year (December through February).

- The required flow diversion is approximately 1 MGD, with the exception of the months of November and April, when the required diversion is estimated at 2.5 – 2.7 MGD.

Table 9-30: Projected Future Excess Thermal Load and Additional Flow Diversions

	Maximum 7DADM Temp. (°F)	WWTP Flow (MGD)	Flow to Oregon Garden (MGD)	Excess Thermal Load (kcal/day)	Allowable ETL (kcal/day)	Additional Diversion (MGD)	Revised ETL (kcal/day)
September	73.6	1.62	0.94	13	5.2	1.7	-20
October 1-15	70.9	1.79	0.43	19	5.2	1.7	-5
October 16-31	70.2	1.79	0.43	42	21.0	1.7	-11
November	64.9	2.56	0.40	43	21.0	0.0	43
December	58.0	4.55	0.40	23	21.0	0.0	23
January	54.5	4.32	0.40	-8	21.0	0.0	-8
February	57.2	3.37	0.40	11	21.0	0.0	11
March	58.3	3.23	0.40	17	21.0	0.0	17
April	63.3	3.1	0.40	45	21.0	0.0	45
May 1-15	67.1	2.26	0.42	45	21.0	0.0	45
May 16-31	68.3	2.26	0.02	18	5.2	1.7	4
June	67.8	1.68	0.70	7	5.2	1.7	-5
July	73.6	1.41	1.05	7	5.2	0.50	-3
August	73.6	1.54	1.03	10	5.2	0.50	0
September	73.6	1.62	0.94	13	5.2	1.7	-20

Future Effluent Management Strategies

Winter Discharge

The existing year-round limits on thermal load to Silver Creek are based on statewide criteria and not on specific conditions or natural thermal potential in the stream. It is extremely difficult to achieve reductions in winter excess thermal load discharges in Western Oregon, as there are no consumptive uses for treated effluent. The Oregon Administrative Rules allow for variances from the winter criteria, described in OAR 340-041-0028(11)(b):

"*A point source that discharges into or above salmon and steelhead spawning waters that are colder than the spawning criterion, may not cause the water temperature in the spawning reach where the physical habitat for spawning exists during the time spawning through emergence use occurs, to increase more than the following amounts after complete mixing of the effluent with the river:*

(A) If the rolling 60 day average maximum ambient water temperature, between the dates of spawning use as designated under subsection (4)(a) of this rule, is 10 to 12.8 degrees Celsius, the allowable increase is 0.5 Celsius above the 60 day average; or

(B) If the rolling 60 day average maximum ambient water temperature, between the dates of spawning use as designated under subsection (4)(a) of this rule, is less than 10 degrees Celsius, the allowable increase is 1.0 Celsius above the 60 day average, unless the source provides analysis showing that a greater increase will not significantly impact the survival of salmon or steelhead eggs or the timing of salmon or steelhead fry emergence from the gravels in downstream spawning reach."

If the final Molalla-Pudding TMDL includes a winter thermal load limit that appears unattainable based on existing data, it is recommended that the City conduct a biological evaluation to determine actual impacts on salmonids and assess whether a variance can be granted.

A previous review by Fishman Environmental determined that habitat and other physical conditions (unaffected by the wastewater treatment plant discharge) limit salmonid production in Silver Creek, and that the removal of the treatment plant effluent from the stream would not impact the likelihood of salmonid spawning or rearing downstream of the outfall. Therefore, if the final Molalla-Pudding TMDL includes a winter thermal load limit that appears unattainable based on existing data, it is recommended that the City conduct an updated biological evaluation to determine actual impacts on salmonids and assess whether a variance can be granted.

If a variance cannot be granted and if diurnal low stream temperatures are lower than the designated stream criteria, it may be possible to store effluent during the day in the existing equalization basin and discharge at night. There is limited diurnal temperature data available for Silver Creek upstream of the treatment plant discharge, so it was not possible to evaluate the potential impacts of diurnal storage and release as part of this Facility Plan.

Summer Discharge

As shown above, the City's discharge exceeds the allowable excess thermal load limits under future conditions, and could exceed limits under current conditions. Many communities in Oregon are facing similar issues, and are exploring or implementing a range of options to address these issues. The options that hold the most promise for Silverton include:

- Optimization of the Oregon Garden Wetland.

  Monitoring data shows that the Oregon Garden wetlands provide limited temperature reduction during portions of the summer. As flows to the Garden are increased it will be important to ensure that the wetland function is optimized to provide the maximum water quality benefit and thermal reduction.

- Subsurface Discharge and Rapid Infiltration.

  A number of communities in Oregon and Washington are studying or implementing subsurface discharge for cooling. The City conducted an Effluent Management Study in 1998 , including examination of the potential to infiltrate effluent on approximately 3 acres of property immediately west of the treatment plant. The study showed that the adjacent property contains a gravel layer suitable for infiltration, and that significant temperature reduction can be achieved, especially during the early summer and late fall months. At the maximum hydraulic rate of 1.7 MGD determined through field testing, estimated effluent temperatures following subsurface infiltration were as shown in Table 9-31 below.

Table 9-31: Projected Cooling from Subsurface Infiltration*

	Silver Creek Temperature	Average Effluent Temperature (°F)	Effluent Temperature After Subsurface Discharge (°F)
May	54.5	63.5	62.3
June	58.8	64.4	63.2
July	63.2	71.6	69.5
August	66.2	73.4	71.0
September	57.0	64.4	63.1
October	50.5	57.2	56.6

*From Effluent Management Study, HDR Engineering, December 1998

This table shows that subsurface infiltration on the adjacent property has the potential to cool effluent to below the stream criteria. As shown in Table 9-30, diversion of 1.7 MGD from the Silver Creek outfall would significantly reduce the effluent thermal load discharge.

Perhaps more importantly, the regulatory climate in Oregon has changed such that subsurface infiltration is becoming a more accepted practice than it was during the previous plant expansion. Subsurface discharge was recently permitted in Lebanon, Oregon, and is being considered as an allowed reuse practices in the current revisions to Division 55 of the Oregon Administrative Rules. Therefore, implementing subsurface discharge on property adjacent to the treatment plant appears to be a viable option to reduce thermal load to Silver Creek.

- Effluent Reuse.

  Even with increased discharge to the Garden and development of subsurface infiltration, additional effluent must be diverted from Silver Creek during July and August. Since this is the period of peak irrigation demand, implementing an effluent reuse program is a logical component of the City's temperature management strategy. Production of Level IV effluent would allow relatively unrestricted use of treated effluent for irrigation. While actual irrigation rates are use-specific, typical rates in Western Oregon are 5-10 inches per month during the summer irrigation season (May – September). Since irrigation requirements in July are typically close to 10 inches, providing reuse opportunities for 0.5 MGD of flow requires approximately 75 acres of irrigated area. A 29-acres school property located roughly between the treatment plant and the Oregon Garden would be a likely candidate for reclaimed water irrigation. Additional reclaimed water customers could be identified through one-on-one contacts with property owners near the treatment plant.

- Additional Wetland Construction.

  If the waste load allocations in the TMDL cannot be met with the methods described above, the City could construct another wetland for temperature reduction. The City of Salem operates a wetland system designed to provide a full two days of detention, with a design loading rate of 3.65 inches/day. Intermittent discharge is typically required, as flow through open water wetlands undergoes warming during the day and cooling at night. At a loading rate of 3.65 inches/day, an additional 10 acres of wetland would be required per MGD of effluent discharged. It would be preferable for wetlands to be constructed in the vicinity of Silver Creek upstream of the existing outfall, resulting in potential temperature credits for wetland cooling.

- Mechanical Cooling.

  Many communities are considering mechanical cooling of effluent to meet thermal load limits; however, the high energy demands associated with this type of treatment result in high life-cycle costs over a 20 year planning period. Mechanical cooling would be a viable option if no other alternatives are available; however, it is not the most cost-effective option.

- Thermal Credit Trading.

  A group of stakeholders has formed an organization called the Willamette Partnership. Their goal is to develop a market-based program for meeting regulatory goals in the Willamette Basin as cost effectively as possible. The Partnership is focusing its initial efforts on developing a thermal credit trading program that allows regulated parties to purchase credits and use them toward implementing high-value temperature reduction measures in the Willamette Basin.

While the Partnership is in the early stages of program development, it is feasible that a thermal credit trading plan will be in place before the end of the compliance period related to the Molalla-Pudding TMDL.

Effluent Management Recommendations

Modeling and data analysis conducted through the Molalla-Pudding TMDL may result in a modified thermal waste load allocation for the City, raising or lower the City's allowable thermal discharge. Therefore, it is recommended that Silverton initiate activities to facilitate compliance with a waste load allocation similar to the excess thermal load in the current NPDES permit, but refrain from making significant capital investments until the TMDL is completed. As described in Chapter 4, the TMDL completion in late 2006 or early 2007 will trigger a compliance schedule within which the City can complete additional capital improvement.

Recommended near-term activities include the following:

- Budget for installation of a third pump in the effluent pump station to allow increased flow to the Oregon Garden
- Conduct a study to optimize performance of the Oregon Garden Wetland for increased temperature reduction and water quality improvement.
- Update the 1998 thermodynamic model of subsurface discharge on the property adjacent to the wastewater treatment plant to evaluate potential temperature reduction based on current effluent and stream temperatures.
- Initiate discussions with the Silver Falls School District regarding irrigation of school property with reclaimed water.
- Initiate a public outreach program to identify additional potential users of reclaimed water.
- Continue to monitor activities of the Willamette Partnership to identify opportunities to buy or sell temperature credits.

Costs related to these activities are shown in Table 9-32.

Table 9-32: Costs of Effluent Management Recommendations

	Cost
Install Third Effluent Pump	$20,000
Oregon Garden Wetland Optimization Study	$25,000
Updated Thermodynamic Model	$35,000

TOTAL	**$80,000**

Chapter 10 - Recommended Plan

Overview

This chapter summarizes the recommended improvements to provide adequate conveyance, treatment, and discharge capacity to serve the City of Silverton's needs through 2030. The project descriptions, costs, and timing are intended to serve as the basis for a Capital Improvement Plan (CIP) for implementing the necessary improvements. All costs are presented in 2006 dollars.

Recommended Future Projections

The future projected flows and loadings are presented in Table 4- below. The table also includes the current facility design flow for comparative purposes.

As Table 4- clearly illustrates, the projected 2030 flows are lower than the current design flow capacity for all conditions other than peak day and peak instantaneous flows. Design BOD, TSS, and ammonia loadings are not shown in Table 4-, as they are lower than the projected 2030 loadings.

Table 10-1: Projected 2030 Total Flow and Loading

	Projected Flow (MGD)	Current Design Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day)	NH3 (lb/day)
Dry Weather						
ADWF	1.71*	2.5	7,765	5,788	1,313	821
MMDWF	2.65	4.3	9,158	8,525	1,504	940
MWDWF	3.06	N/A				
MDDWF		6.0				
Wet Weather						
AWWF	2.54	4.6				
MMWWF	4.17	6.6	9,158	8,525	1,504	940
MWWWF	6.62	N/A				
PDAF	10.87	10.0				
PIF	15.73	12.0				

* The average dry weather flow was also adjusted by adding 0.2 MGD to account for baseline infiltration in the measured plant effluent (on average, measured plant effluent exceeds influent by approximately 0.2 MGD).

Because the design capacity has already been provided as part of the previous facility upgrades, the current design capacity will be used as the future planning basis for all flow conditions other than PDAF and PIF. For those two flow conditions (as well as CBOD, TSS, and nutrient loadings) the projected 2030 values will serve as the future planning basis. This approach results in the recommended facility plan flow and loading values shown in Table 10-2.

Table 10-2: Recommended Facility Plan Flow and Loading

	Flow (MGD)	CBOD (lb/day)	TSS (lb/day)	TKN (lb/day	NH3 (lb/day)
Dry Weather					
ADWF	2.5	7,800	5,800	1,300	820
MMDWF	4.3	9,200	8,500	1,500	940
MWDWF					
MDDWF	6.0				
Wet Weather					
AWWF	4.6				
MMWWF	6.6	9,200	8,500	1,500	940
MWWWF					
PDAF	11				
PIF	16				

Wastewater Collection System Recommendations

Collection system recommendations address three specific system needs, which are described individually below.

- Improvements to Increase Hydraulic Capacity
- Improvements to Address Condition Deficiencies
- Implementation of a Comprehensive Condition Assessment Program

Hydraulic analysis of the collection system has identified several locations where improvements need to be made to the collection system to handle current and future flow loading. The overall goal of the collection system improvements is to eliminate surcharging in the system; therefore, improvements were identified in pipe segments where flow depths under future conditions were at or above 75%of the pipe capacity.

Table 8-7 shows the recommended pipeline improvements to address hydraulic capacity issues in the collection system. The hydraulic analysis revealed that several pipe segments will exceed design capacity in the near future, and additional capacity will be required in the Oregon Gardens Lift Station and force main to serve the new hotel located near the Oregon Garden. Otherwise, the system is generally adequately sized to meet the City's needs through the planning horizon. Several additional pipe segments exceeded the 75% criteria near the end of the planning horizon; however, no surcharging was predicted under design conditions.

Table 10-3: Recommended Capacity Related Pipeline Improvements

Improvement ID	Capacity Issue ID	Improvement Location	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-1	CP-1	Westfield Street	Upsize 6-inch to 8-inch	910	$229,800	2008
IMP-2	n/a	Oregon Gardens Lift Station and force main	Increase lift station firm capacity from 200 gpm to 400 gpm.	2 new 400 gpm pumps (1 stand-by)	$18,600	2007 - 2008 (complete before hotel opening)
			Upsize force main from 4-inches to 6-inches	909	$182,500	
IMP-3	CP-3	S. James Street	Upsize 12-inch to 18-inch	576	$214,600	2020-2030
IMP-4	CP-4	Sherman Street	Upsize 12-inch to 18-inch	175	$70,000	2020-2030
IMP-5	CP-5	Adams Street	Upsize 8-inch to 12-inch	850	$283,900	2020-2030

In addition to the pipeline improvements identified in Table 8-7, the City has identified the locations for three new pump stations to serve future growth areas within the Urban Growth Boundary. These pump stations are described in Table 1-5.

Table 10-4: Additional Pump Stations

Improvement ID	Pump Station	Description	Estimated Cost	Project Timing
PMP-1	James Street	New pump station & 8-inch forcemain. Including 18-inch and 12-inch trunk lines on James and Jefferson to connect to existing system. Decommission James & Florida Drive & Second & Jefferson Street Pump Stations	$928,400	2008
PMP-2	Pine Street	New pump station & forcemain	$162,100	2009
PMP-3	Setness Lane	New pump station & 6-inch forcemain and associated 8-inch collector pipes.	$1,038,000	2020

Figure 10-1 shows the locations of these recommendations in the collection system.

Table 10-5 lists high priority pipeline improvement projects that are recommended to address condition issues identified in the Electroscan pipe condition assessment that was performed in August, 2006.

Table 10-5: Recommended Condition Assessment Related Pipeline Improvements

Improvement ID	Improvement Location	Existing Diameter (in)	Recommended Improvement	Total Length (ft)	Estimated Cost	Project Timing
IMP-6	Schlador Street	18	Slipline/pipeburst	572	$70,000	2007
IMP-7	Lone Oaks Street	15	Slipline/pipeburst	355	$40,000	2007
IMP-8	Third St.	15	Slipline/pipeburst	770	$85,000	2008
IMP-9	Meat Packers/High School Area	18	Slipline/pipeburst	385	$46,000	2008

It is recommended that CCTV collection system condition assessment continue as part of the City's routine maintenance program. A summary of the types of pipe materials and corresponding lengths required for condition assessment is presented in Table 10-6 below. In general, the pipe condition or rate of pipe deterioration is often related to the pipe material. For example, clay pipe is typically found in older collection systems and therefore has higher defect rates whereas PVC is associated with newer construction and generally has lower defect rates. The condition assessment priorities were established based on HDR's experience and observations of pipe condition in other similar systems.

Table 10-6: Prioritized Program for Future Condition Assessment

Pipe Material	Total Length Required for Assessment (ft)	PW Cost	Year(s) to be Performed
Clay	6,080	$6,080	2007
Unknown	63,530	$51,163	2008-2019
Concrete (excluding Water St.)	24,830	$16,662	2019-2020
Ductile Iron	1,780	$1,177	2020
PVC	52,080	$29,830	2020-2030
Total	**148,300**	**$104,913**	

Figure 10-1
Recommended Collection System Improvements

HDR ONE COMPANY *Many Solutions*[SM]

Wastewater Treatment Facility Recommendations

The major focus of improvements at the wastewater treatment plant is on the biosolids treatment and storage processes. Additional improvements were identified to enhance biological treatment, address operational and/or condition deficiencies, provide the capability to produce Level IV reclaimed water, and enhance lab and administrative facilities. Recommended improvements are shown on the site plan in Figure 10-1.

Liquid Stream Treatment Improvements

Because the flows to the treatment plant are generally not projected to increase beyond the current design capacity, few improvements are required in the liquid stream treatment process. Furthermore, the facility was initially designed based on conservative assumptions regarding treatment performance, and process modeling conducted as part of this Facility Plan indicates that the actual treatment capacity is greater than the current design capacity.

Based on the evaluation of the secondary treatment, it is recommended that initial improvements (Phase 1) focus on process control upgrades and optimization, including a rerating of the plant to its true capacity. Treatment expansion options (Phase 2) were examined, but these are not expected to be required during the planning horizon.

Process control upgrades and optimization during Phase 1 include the following

- Online alkalinity control
- Aeration control based on multi-point aeration basin DO measurement and online effluent ammonia analyzer
- Automated SRT with Online MLSS meter
- Process Optimization, stress testing and capacity rerating

The optimized process is expected to provide 50% more treatment capacity than the current 2.2 MGD capacity; and therefore, only improvements to the existing aeration system are required during the planning horizon.

For the Phase 2 expansion phase, three alternatives were reviewed:

- Additional capacity using existing activated sludge process configuration
- MBR technology
- Hybrid technology

Given that the optimized facility will provide sufficient capacity well beyond the 2030 planning horizon, and that both MBR and Hybrid technologies are still emerging, a specific technology is not appropriate. Based on today's state of the technology and current economic parameters, the hybrid technology appears most attractive, followed by the addition of a third conventional treatment train.

The cost of the secondary treatment improvements is estimated at $650,000. The aeration system upgrade does not have to occur until approximately 2020 when the plant loading exceeds the aeration capacity of the existing system. Once aeration system improvements are in place, the City can proceed with stress testing and system rerating. Table 10-7 shows the cost breakdown of the improvements, which are broken up into three phases. Phase 1a includes recommendations that should be implemented immediately, and Phase 1b includes upgrades recommended as soon as they can be accommodated in the City's budget. Phase 2 should be implemented between 2010 and 2015. Phase 3 improvements should be implemented when influent maximum month dry weather flows approach 2.2 MGD, which is expected to occur in 2020.

Table 10-7: Estimated Cost for Process Control Upgrades and Process optimization

Improvement	Estimated Cost
Phase 1b	
Alkalinity Feed Control	$75,000
Aeration control	$150,000
Online Ammonia Analysis	$50,000
Total – Phase 1b	**$275,000**
Phase 2	
Automated SRT control	$75,000
Aeration System upgrade**	$250,000
Total – Phase 2	**$325,000**
Phase 3	
Stress Testing*	$40,000
Rerating	$10,000
Total – Phase 3	**$50,000**

* does not include operator time, assumes are sample analysis would be done in-house or with online metering equipment

** additional diffusers, headers, and blowers

Liquid stream improvements are also required to provide additional effluent pumping capacity. This project is described under the Effluent Management Recommendations below.

Effluent Filtration

The existing treatment plant does not include effluent filtration; however, filtration may be required to comply with future effluent turbidity limits described in Chapter 5. In addition, implementation of an effluent reuse program will likely be required to comply with the City's thermal load limit. For planning purposes, this Plan evaluated options for providing effluent filtration to provide 1 MGD of reuse quality water.

The cloth filter technology appears to be the most compact and least expensive option that also delivers the lowest effluent turbidity. However, filter technologies are improving rapidly, and costs amongst the various technologies evaluated in this Plan were very competitive. Therefore it may be beneficial for the City to allow other filtration technologies in the bidding process to produce the most favorable bids possible. The estimated cost for a cloth media effluent filtration system is $380,000. This cost only includes yard piping within the treatment plant property lines. Construction cost for a reuse discharge line and potential reuse pump station are not included.

Solids Stream

Major improvements to the plant's solids handling facilities are required to provide adequate treatment capacity, reduce the volume of solids stored onsite, and address equipment condition and operational deficiencies. Recommended solids handling improvements include:

- Replacing the primary sludge pump station and grit classifier
- Implementing sludge dewatering and lime stabilization, including:
- Conversion of the existing digesters to thickened sludge blend tanks
- Construction of a new building to house new screw press dewatering and lime stabilization equipment
- Conversion of an existing sludge storage lagoon to an open-air dewatered biosolds storage facility

It is recommended that the City proceed immediately with design and construction of improvements related to increasing the biosolids treatment capacity. The primary sludge pumping and grit classifier improvements can be deferred to a later phase if funding is not available to include these elements in the initial solids handling expansion. As it is not feasible to provide odor control for the open-air biosolids storage facility, it may ultimately be necessary for the City to construct an enclosed storage building with appropriate odor control facilities. The need for this building will be determined in large part by the success of the City's limed biosolids reuse program and the length of time for which biosolids must be stored.

The recommended solids handling capital improvements are presented in Table 10-8.

Table 10-8: Recommended Capital Improvements for Silverton Solids Processing[1]

Improvement	Cost
Phase 1a	
Thickened Sludge Blend Tanks	$374
Dewatering and lime stabilization facility[2]	$2,386
Covered Limed Biosolids Storage	$342
Odor control (assume 3-stage chemical scrubber)	$624
Recycle Pump Station	$305
Total Phase 1a Construction Costs	**$4,032**
Engineering, Administrative, and Legal (30%)	$1,175
Total Phase 1a Project Costs	**$5,207**
Phase 1b	
Primary Sludge Pump Station	$420
Grit Classifier Replacement	$159
Total Phase 1b Construction Costs	**$579**
Engineering, Administrative, and Legal (30%)	$174
Total Phase 1b Project Costs	**$753**

1. Costs include engineering, administrative, and legal costs (estimated at 20% of construction cost for biosolids storage facility; 30% of construction cost for other improvements)

2. $1,527 of this cost is attributable to the dewatering facility only

Laboratory and Administrative Facilities

Improvements to the lab and administrative building are required to support the staff functions required for efficient long-term operation and maintenance of the treatment plant. Recommended improvements include:

- Adding a new laboratory space with a dedicated HVAC system
- Remodeling the existing laboratory to provide office space for operations and records storage
- Providing new male and female locker room facilities

The cost of these improvements was estimated assuming the total area for new and renovated facilities would be approximately 1,000 square feet. The estimated project cost is $300,000, including contingency, engineering, and administrative fees. The City should initiate this project with a Schematic Design effort to determine specific facility needs, adjacencies, and layout.

Effluent Management Recommendations

Future Effluent Management Strategies

The recommended effluent management strategy is driven by the need to meet an excess thermal load limit during the summer season. Recommendations are based on the calculated thermal load limits that will become effective upon expiration of the City's permit, but may be modified through implementation of the Molalla-Pudding TMDL. The City has been actively following the development of the TMDL, and should continue to monitor its progress and potential impacts on the City's program. It is recommended that Silverton initiate activities to facilitate compliance with a waste load allocation similar to the excess thermal load in the current NPDES permit, but refrain from making significant capital investments until the TMDL is completed.

Winter Discharge

The existing year-round limits on thermal load to Silver Creek are based on statewide criteria and not on specific conditions or natural thermal potential in Silver Creek. It is extremely difficult to achieve reductions in winter excess thermal load discharges, since there are no consumptive uses for treated effluent. A prior study by Fishman Environmental suggested that removal of the treatment plant effluent from the stream would not impact the likelihood of salmonid spawning or rearing downstream of the outfall. Therefore, if the final Molalla-Pudding TMDL includes a winter thermal load limit that appears unattainable based on existing data, it is recommended that the City conduct a biological evaluation to determine actual impacts on salmonids and assess whether a variance can be granted.

Summer Discharge

- A number of options were evaluated for compliance with the anticipated summer excess thermal load limits. Recommended near term activities include the following:
- Budget for installation of a third pump in the effluent pump station to allow increased flow to the Oregon Garden
- Conduct a study to optimize performance of the Oregon Garden Wetland for increased temperature reduction and water quality improvement.
- Update the 1998 thermodynamic model of subsurface discharge on the property adjacent to the wastewater treatment plant to evaluate potential temperature reduction based on current effluent and stream temperatures.

- Initiate discussions with the Silver Falls School District regarding irrigation of school property with reclaimed water.
- Initiate a public outreach program to identify additional potential users of reclaimed water.
- Continue to monitor activities of the Willamette Partnership to identify opportunities to buy or sell temperature credits.

Costs related to these activities are shown in Table 10-9.

Table 10-9: Costs of Effluent Management Recommendations (all Phase 1)

	Cost
Install Third Effluent Pump	$20,000
Oregon Garden Wetland Optimization Study	$25,000
Updated Thermodynamic Model	$35,000
TOTAL	**$80,000**

Summary of Project Costs and Implementation Schedule

Table 1-9 summarizes recommended collection system and treatment plant improvement projects, costs, and timing. Five discrete wastewater treatment plant projects were identified, incorporating various elements of the overall treatment improvement recommendations. The projects are described below.

- **Project 1: Phase 1 Biosolids Expansion, Phase 1a Process Optimization, Effluent Pumping.** This project includes the Phase 1 capacity-related biosolids improvements (blend tank, dewatering/lime stabilization facility, odor control, recycle pump station improvements, sludge storage), addition of the third effluent pump, and installation of alkalinity feed control, aeration control, and online ammonia analyzers associated with Phase 1a of the secondary treatment improvements. Ongoing process optimization will begin at the completion of Project 1.
- **Project 2: Phase 2 Biosolids Handling, Lab & Admin Facilities.** This project includes upgrading the primary sludge pump station and replacing the grit classifier, as well as expansion of the lab and administrative facilities.
- **Project 3: Aeration System Upgrade.** This project provides additional blower and aeration capacity to support treating higher loads in the secondary treatment process. This project will be required when maximum month influent flows approach 2.2 mgd, which is anticipated to occur after 2015.

- **Project 4: Secondary Treatment Stress Testing/Rerating.** The secondary treatment system stress testing and rerating will be completed following the aeration system upgrade.

- **Project 5: Effluent Filtration/Subsurface Discharge/Reuse.** This project includes capital improvements required to meet temperature TMDL requirements or support development of an effluent reuse program. The timing and cost of this project will depend on the final thermal load allocation in the Molalla-Pudding TMDL, and/or opportunities to use effluent for beneficial reuse applications.

Table 10-10: Recommended Capital Improvements for Silverton Collection System and Treatment Plant Improvements ($1,000s

	2007	2008	2009	2010	2011	2015	2020-2030	Cost ($1,000s)
COLLECTION SYSTEM IMPROVEMENTS								**$ 1,345**
IMP-1 (Westfield Street Capacity)								$ 230
IMP-2 (Oregon Garden Lift Station Capacity)								$ 201
IMP-3 (S. James Street Capacity)								$ 215
IMP-4 (Sherman Street Capacity)								$ 70
IMP-5 (Adams Street Capacity								$ 284
IMP-6 (Schlador Street Condition)								$ 70
IMP-7 (Lone Oaks Street Condition)								$ 40
IMP-8 (Third Street Condition)								$ 85
IMP-9 (Meat Packers/High School Condition)								$ 46
Condition Assessment Program								$ 105
ADDITIONAL PUMP STATIONS								**$ 2,128**
PMP-1 James Street Pump Station								$ 928
PMP-2 Pine Street Pump Station								$ 162
PMP-3 Setness Lane Pump Station								$ 1,038
WASTEWATER TREATMENT PLANT IMPROVEMENTS								**$ 7,018**
Studies								*$ 85*
Thermodynamic Model Update								$ 35
Wetland Optimization Study								$ 25
Laboratory/Admin Facility Schematic Design								$ 30
Project 1 - Phase 1 Biosolids Expansion; Phase 1a Process Optimization; Effluent Pumping								***$ 5,507***
Solids/Effluent Pumping Expansion								$ 5,232
Predesign								
Design								
Construction								
Phase 1a Process Optimization								$ 275
Ongoing Process Optimization								
Project 2 - Phase 2 Biosolids Handling/Lab & Admin								***$ 1,023***
Design								
Construction								
Project 3 - Aeration System Upgrade								***$ 325***
Project 4 - Secondary Treatment Stress Testing/Rerating								***$ 163***
Project 5 - Effluent Filtration/Subsurface Discharge/Other Reuse								

Chapter 11 - References

City of Silverton

2001 Comprehensive Plan

Department of Environmental Quality (DEQ)

2005a Climatography of the United States No. 20, Climatic Summaries for Selected Sites, 1951-1980.

2005b National Pollutant Discharge Elimination System Permit Evaluation and Fact Sheet, January 28, 2005.

2005c Reasonable Potential Analysis for Toxic Pollutants, Oregon Department of Environmental Quality Internal Management Directive, September 2005

2005d Draft Implementation Guidance for Turbidity Standard, Oregon Department of Environmental Quality, October 14, 2005.

2005e Implementing Oregon's Biosolids Program, Oregon Department of Environmental Protection, December 2005.

Environmental Protective Agency (EPA)

1999 Biosolids generation, use and disposal in the United States, Environmental Protection Agency, Office of Solid Waste EPA530-R-99-009

2005 Proposed EPA Policy on Permit Requirements for Peak Wet Weather Discharges from Wastewater Treatment Plants Serving Sanitary Sewer Collection Systems, December 2005

FCS Group

2005 City of Silverton System Development Charge Study for the Transportation, Water, and Sewer Services. August 2005

HDR Engineering, Inc. (HDR)

2000 Oregon Garden Foundation/City of Silverton Oregon Garden Water Management Plan, HDR Engineering, November 2000.

WERF

2003 Identifying and Controlling Odor in the Municipal Wastewater Environment Phase II: Impacts of in-Plant Parameters on Biosolids Odor Quality

City of Silverton

Wastewater System Facility Master Plan

Appendix A

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Western Region – Salem Office
750 Front Street NE, Suite 120, Salem, OR 97301-1039
Telephone: (503) 378-8240

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:

City of Silverton
306 S. Water Street
Silverton, OR 97381

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 2.35
Oregon Gardens Wetland	002	Oregon Gardens Wetland
Emergency Overflow		
Surge Basin Overflow	003	R.M. 2.35

FACILITY TYPE AND LOCATION:

Activated Sludge
Silverton STP
400 Schemmel Lane
Silverton
Treatment System Class: Level IV
Collection System Class: Level III

RECEIVING STREAM INFORMATION:

Basin: Willamette
Sub-Basin: Molalla-Pudding
Receiving Stream: Silver Creek
LLID: 1228414450001 - 2.35 - D
County: Marion

EPA REFERENCE NO: OR002065-6

Issued in response to Application No. 983753 received January 29, 2004. This permit is issued based on the land use findings in the permit record.

Michael H. Kortenhof, Western Region Water Quality Manager　　Date　August 2, 2005

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

Unless specifically authorized by this permit, by another NPDES or WPCF permit, or by Oregon Administrative Rule, any other direct or indirect discharge of waste is prohibited, including discharge to waters of the state or an underground injection control system.

SCHEDULE A

1. **Waste Discharge Limitations not to be exceeded after permit issuance.**

a. Treated Effluent Outfall 001 (to Silver Creek)

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations		Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
	Monthly	Weekly			
$CBOD_5$ (see Note 1)	10 mg/l	15 mg/l	300	330	420
TSS	10 mg/l	15 mg/l	300	330	420

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations		Monthly** Average lb/day	Weekly** Average lb/day	Daily** Maximum lbs
	Monthly	Weekly			
$CBOD_5$ (see Note 1)	25 mg/l	40 mg/l	830	1100	1500
TSS	30 mg/l	45 mg/l	1300	1700	2200

* Average dry weather design flow to the facility is 2.5 MGD. Effluent loadings are based on the capability of the treatment works at 3.6 MGD monthly average, 4.0 MGD weekly average and 5.0 MGD daily maximum (two year recurrence flows).

** Average wet weather design flow to the facility is 4.6 MGD. Effluent loadings are based on the capability of the treatment works at 5.0 MGD monthly average, 6.6 MGD weekly average and 8.8 MGD daily maximum (two year recurrence flows).

(3) Other parameters

Year-round (except as noted)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (see Note 2)
pH	Shall be within the range of 6.5 - 9.0
$CBOD_5$ and TSS Removal Efficiency	Shall not be less than 85% monthly average for $CBOD_5$ and TSS.
Dissolved Oxygen	Shall not be less than 6.5 mg/l as a daily average (May 1 - October 31).
Ammonia-N (May 1 through October 31)	Shall not exceed a monthly average concentration of 0.88 mg/L and a daily maximum concentration of 2.0 mg/L (see Note 3)
Excess Thermal Load (May 16 through October 14)	Shall not exceed a weekly average of 5.2 million Kcals/day (see Note 4)
Excess Thermal Load (October 15 through May 15)	Shall not exceed a weekly average of 21 million Kcals/day (see Note 4)

(4) No chlorine or chlorine compounds shall be used for disinfection purposes and no chlorine residual shall be allowed in the effluent due to chlorine used for maintenance purposes.

(5) Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0445 except in the following defined mixing zone:

The allowable mixing zone is that portion of the Silver Creek where the effluent mixes with 25 percent of the stream flow but in no case shall it extend farther than seventeen (17) feet from the north bank of the river and extending from a point ten (10) feet upstream of the outfall to a point one hundred sixty (160) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) is defined as that portion of the allowable mixing zone that is within sixteen (16) feet of the point of discharge.

b. Treated Effluent Outfall Number 002 (to Oregon Gardens)

(1)

Parameter	Average Effluent Concentrations		Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
	Monthly	Weekly			
$CBOD_5$ (see Note 1)	10 mg/l	15 mg/l	300	330	420
TSS	10 mg/l	15 mg/l	300	330	420

* Effluent loadings are based on the capability of the treatment works at 3.6 MGD monthly average, 4.0 MGD weekly average and 5.0 MGD daily maximum (two year recurrence flows).

(2) Other parameters

Year-round	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (see Note 2)
pH	Shall be within the range of 6.5 - 8.5
$CBOD_5$ and TSS Removal Efficiency	Shall not be less than 85% monthly average for $CBOD_5$ and 85 TSS.
Dissolved Oxygen	Shall not be less than 5.5 mg/l as a daily average.
Ammonia-N(see Note 3)	Temperature dependent (see below)
Monthly average effluent temperature ≤ 12°C	Shall not exceed a monthly average concentration of 1.3 mg/L and a daily maximum concentration of 3.0 mg/L
Monthly average effluent temperature >12°C but ≤ 14°C)	Shall not exceed a monthly average concentration of 1.3 mg/L and a daily maximum concentration of 3.0 mg/L
Monthly average effluent temperature >14°C but ≤ 16°C)	Shall not exceed a monthly average concentration of 1.3 mg/L and a daily maximum concentration of 2.9 mg/L
Monthly average effluent temperature >16°C but ≤ 18°C)	Shall not exceed a monthly average concentration of 1.3 mg/L and a daily maximum concentration of 2.9 mg/L
Monthly average effluent temperature >18°C but ≤ 20°C)	Shall not exceed a monthly average concentration of 1.3 mg/L and a daily maximum concentration of 2.9 mg/L
Monthly average effluent temperature >20°C but ≤ 22°C)	Shall not exceed a monthly average concentration of 1.1 mg/L and a daily maximum concentration of 2.5 mg/L
Monthly average effluent temperature >22°C but ≤ 24°C)	Shall not exceed a monthly average concentration of 0.96 mg/L and a daily maximum concentration of 2.2 mg/L
Monthly average effluent temperature >24°C)	Shall not exceed a monthly average concentration of 0.84 mg/L and a daily maximum concentration of 1.9 mg/L

(3) No chlorine or chlorine compounds shall be used for disinfection purposes and no chlorine residual shall be allowed in the effluent due to chlorine used for maintenance purposes.

(4) Not withstanding the effluent limitations established by this permit, except as provided

for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0445. No acute or chronic toxicity due to ammonia or other compounds as measured by whole effluent toxicity testing shall be allowed in the effluent.

c. Combined Mass Load Discharge from Outfall 001 and 002:

The mass load of $CBOD_5$ and TSS in the combined discharge from Outfalls 001 and 002 shall not exceed the seasonally appropriate $CBOD_5$ and TSS mass load limits for Outfall 001.

d. Surge Basin Overflow Outfall Number 003

No waste shall be discharged from this outfall and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0445, unless the cause of the discharge is due to storm events as allowed under OAR 340-041-0120(13) and (14) as follows:

Raw sewage discharges are prohibited to waters of the State from November 1 through May 21, except during a storm event greater than the one-in-five-year, 24-hour duration storm, and from May 22 through October 31, except during a storm event greater than the one-in-ten-year, 24-hour duration storm. If an overflow occurs between May 22 and June 1, and if the permittee demonstrates to the Department's satisfaction that no increase in risk to beneficial uses occurred because of the overflow, no violation shall be triggered if the storm associated with the overflow was greater than the one-in-five-year, 24-hour duration storm.

e. No activities shall be conducted that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be managed and disposed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

NOTES:

1. The CBOD5 concentration limits are considered equivalent to the minimum design criteria for BOD5 specified in Oregon Administrative Rules (OAR) 340-041. These limits and CBOD5 mass limits may be adjusted (up or down) by permit action if more accurate information regarding CBOD5/BOD5 becomes available.

2. If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.

3. The ammonia limits in Schedule A, Condition 1.a (3) and Condition 1.b (2) shall become effective upon completion of the compliance schedule contained in Schedule C, Condition 3 or by the expiration date of this permit, whichever is sooner. The ammonia limits are based on the estimated background concentration, estimated dilution in the mixing zone and the 1986 EPA Gold Book Criteria. The ammonia limits are considered interim. The State of Oregon has adopted the EPA 1999 ammonia criteria. Upon approval of the new standard by the EPA, the following limits will automatically be applied to the discharge without a permit modification:

Outfall 001

Parameter	Limitations
Ammonia-N (May 1 to October 31)	Shall not exceed a monthly average concentration of 3.0 mg/L and a daily maximum concentration of 7.8 mg/L

Outfall 002

Monthly Average Effluent Temperature	Limitations

≤ 12°C	No limit
>12°C but ≤ 14°C	Shall not exceed a monthly average concentration of 4.4 mg/L and a daily maximum concentration of 10.0 mg/L
>14°C but ≤ 16°C	Shall not exceed a monthly average concentration of 3.9 mg/L and a daily maximum concentration of 8.8 mg/L
>16°C but ≤ 18°C	Shall not exceed a monthly average concentration of 3.4 mg/L and a daily maximum concentration of 7.7 mg/L
>18°C but ≤ 20°C	Shall not exceed a monthly average concentration of 3.0 mg/L and a daily maximum concentration of 6.8 mg/L
>20°C but ≤ 22°C	Shall not exceed a monthly average concentration of 2.6 mg/L and a daily maximum concentration of 6.0 mg/L
>22°C but ≤ 24°C	Shall not exceed a monthly average concentration of 2.3 mg/L and a daily maximum concentration of 5.2 mg/L
>24°C	Shall not exceed a monthly average concentration of 2.0 mg/L and a daily maximum concentration of 4.6 mg/L

The revised limits shall apply to Schedule A, Condition 1.a (3) from May 1 through October 31. The revised limit shall apply to Schedule A, Condition 1.b (2) year-round.

4. The Permittee shall comply with the Excess Thermal Load limits upon completion of Schedule C, Condition 3 or by the expiration date of this permit, whichever is sooner. The Excess Thermal Load limit is considered interim and may be adjusted up or down or eliminated when more accurate effluent temperature data becomes available. In addition, upon approval of a Total Maximum Daily Load for temperature for this sub-basin, this permit may be re-opened to include new or revised limits or other conditions or requirements regarding temperature and/or thermal loads.

SCHEDULE B

1. **Minimum Monitoring and Reporting Requirements** (unless otherwise approved in writing by the Department). The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by this permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

a. Influent

The facility influent grab and composite samples and all measurements samples are taken just after the barscreen. The composite sampler is located on the catwalk over influent channel.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
$CBOD_5$	2/Week	24-hour Composite
TSS	2/Week	24-hour Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent composite samples and measurements are taken just before the effluent meter in the effluent pumping station. The composite sampler is located on the concrete walkway beside the Parshall flume. Grab samples are taken from the post aeration basin.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
$CBOD_5$	2/Week	24-hour Composite
Ammonia-N	2/Week	24-hour Composite
TSS	2/Week	24-hour Composite
pH	3/Week	Grab
E. coli	2/Week	Grab (see Note 1)
UV Radiation Dose	Daily	Reading (see Note 2)
Pounds Discharged ($CBOD_5$ and TSS)	2/Week	Calculation
Average Percent Removed ($CBOD_5$ and TSS)	Monthly	Calculation
Temperature:		
Effluent Temperature, Daily Max	Daily	Continuous (see Note 3)
Effluent Temperature, Average of Daily Maximums	Weekly	Calculation
Excess Thermal Load (May 16 through October 14)	Weekly	Calculation (see Note 4)
Excess Thermal Load (October 15 through May 15)	Weekly	Calculation (see Note 5)

b. Treated Effluent Outfall 001 (Continued)

Item or Parameter	Minimum Frequency	Type of Sample
Nutrients:		
TKN, NO2+NO3-N, Total Phosphorus	1/Week (May-Oct)	24-hour Composite
Toxics:		
Metals (Ag, Cd, Cu, Hg, Pb, Se, Zn) measured as total and Cyanide in mg/L (see Note 6)	Semi-annually (see Note 7)	24-hour daily composite
Priority Pollutants	(see Note 8)	24-hour daily composite
Whole Effluent Toxicity	Annually (see Note 9)	Acute & chronic

c. Treated Effluent Outfall 002 (Oregon Gardens)

The sampling location is the same as for Outfall 001. It is not intended that duplicate sampling or analysis be performed when the treated wastewater is discharged through both Outfalls.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
$CBOD_5$	2/Week	24-hour Composite
Ammonia-N	2/Week	24-hour Composite
TSS	2/Week	24-hour Composite
pH	3/Week	Grab
E. coli	2/Week	Grab (see Note 1)
UV Radiation Dose	Daily	Reading (see Note 2)
Pounds Discharged ($CBOD_5$ and TSS)	2/Week	Calculation
Average Percent Removed ($CBOD_5$ and TSS)	Monthly	Calculation
Nutrients:		
TKN, NO2+NO3-N, Total Phosphorus	1/Week	24-hour Composite
Toxics:		
Metals (Ag, Cd, Cu, Hg, Pb, Se, Zn) measured as total and Cyanide in mg/L (see Note 6)	Semi-annually (see Note 7)	24-hour daily composite
Priority Pollutants	(see Note 8)	24-hour daily composite
Whole Effluent Toxicity	Annually (see Note 9)	Acute & chronic
Temperature:		
Effluent Temperature, Daily Max	Daily	Continuous (see Note 3)

d. Emergency Overflow Outfall 003 (Surge Basin)

Item or Parameter	Minimum Frequency	Type of Sample
Flow	Daily (during each occurrence)	Duration and volume

e. Biosolids Management (see Note 10)

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total Solids (% dry wt.) Volatile solids (% dry wt.) Biosolids nitrogen for: NH_3-N; NO_3-N; & TKN (% dry wt.) Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units) Sludge metals content for: As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg	Annually	Composite sample to be representative of the product to be land applied from the facultative storage lagoon (see Note 11)
Record of locations where biosolids are applied on each DEQ approved site. (Site location maps to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, volume & locations where sludges were applied recorded on site location map.
Record of % volatile solids reduction accomplished through stabilization	Monthly	Calculation (see Note 12)
Record of digestion days (mean cell residence time)	Monthly	Calculation (see Note 13)
Daily Minimum Sludge Temperature	Daily	Record

f. Silver Creek

Item or Parameter	Minimum Frequency	Type of Sample
Cd, Cu, Pb, Se, Ag, Se, Zn & Cyanide, measured as total in mg/L	Semi-annually during one day of the 3 consecutive days of effluent metals monitoring (See Note 14)	Grab
TSS	See Note 14	Grab
Hardness (mg/L $CaCO_3$)	See Note 14	Grab

g. Oregon Gardens Wetlands (Complexes A, B and C)

Item or Parameter	Minimum Frequency	Type of Sample
Ammonia-N	Once per 2 Weeks	Grab
Dissolved Oxygen	Once per 2 Weeks	Grab
Temperature	Once per 2 Weeks	Record
pH	Once per 2 Weeks	Grab

h. Brush Creek

Item or Parameter	Minimum Frequency	Type of Sample
Ammonia-N	Once per 2 Weeks	Grab
Dissolved Oxygen	Once per 2 Weeks	Grab
Temperature	Once per 2 Weeks	Record
pH	Once per 2 Weeks	Grab

2. **Reporting Procedures**

a. Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Western Region - Salem office by the 15th day of the following month.

b. State monitoring reports shall identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports shall also identify each system classification as found on page one of this permit.

c. Monitoring reports shall also include a record of the quantity and method of use of all sludge removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

3. **Report Submittals**

a. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by June 1 each year which details sewer collection maintenance activities that reduce inflow and infiltration. The report shall state those activities that have been done in the previous year and those activities planned for the following year.

b. For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes solids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-050-0035(6)(a)-(e).

NOTES:

1. *E. coli* monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or an authorized representative:

Method	Reference	Page	Method Number
mTEC agar, MF	Standard Methods, 18th Edition	9-29	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

2. The UV radiation dose passing through the water column will affect the systems ability to kill organisms. To track the UV dose, the UV disinfection system must include a UV intensity meter with a sensor located in the water column at a specified distance from the UV bulbs. This meter will measure the dose of UV radiation in mWatts-seconds/cm2. The daily UV radiation dose shall be determined by reading the meter each day. If more

than one meter is used, the daily recording will be an average of all meter readings each day.

3. When continuous monitors are used, indicate the time interval between temperature readings, and results are to be tabulated and submitted in an annual report. All continuous temperature monitors are to be checked visually monthly to insure that the devices are still in place and submerged. All continuous temperature monitors must be audited quarterly following procedures described in DEQ Procedural Guidance for Water Temperature Monitoring. The Department acknowledges that uninterrupted data collection is not guaranteed due to vandalism, theft, damage or disturbance. In the event of equipment failure or loss, the permittee shall notify the Department and deploy new equipment to minimize interruption of data collection.

4. Calculated as follows:
(Weekly average of daily maximum effluent temperatures in °C - applicable stream temperature standard, 18°C) X (Weekly average of daily flow in MGD) X 3.785 = Excess Thermal Load, in Million Kcals/day.

5. Calculated as follows:
(Weekly average of daily maximum effluent temperatures in °C - applicable stream temperature standard, 13°C) X (Weekly average of daily flow in MGD) X 3.785 = Excess Thermal Load, in Million Kcals/day.

6. For effluent cyanide samples, at least six (6) discrete grab samples shall be collected over the operating day. Each aliquot shall not be less than 100 mL and shall be collected and composited into a larger container which has been preserved with sodium hydroxide for cyanide samples to insure sample integrity.

7. During the first two years after permit issuance, special monitoring for cyanide, cadmium, copper, lead, mercury, selenium, silver and zinc shall be conducted on the effluent. TSS and hardness shall be monitored simultaneously. The special monitoring for cyanide, cadmium, copper, lead, selenium, silver and zinc shall be conducted using a "clean" sampling method, an "ultra-clean" sampling method, EPA method 1669 or any other test method approved by the Department. The special monitoring for mercury shall be conducted in accordance with EPA Method 1631. After the first two years, special monitoring of the effluent may be eliminated unless otherwise notified in writing by the Department. For all tests, the method detection limit shall be reported along with the sample result.

8. The permittee shall perform all testing required in Part D of EPA Form 2A. The testing includes all metals (total recoverable), cyanide, phenols, hardness and the 85 pollutants included under volatile organic, acid extractable and base-neutral compounds. In addition, the permittee shall monitor for the pesticide pollutants listed in Table II of Appendix D of 40 CFR Part 122. The monitoring needs to be conducted using EPA Methods 624 for volatile organic compounds, EPA Method 625 for semi-volatile organic compounds and Polycyclic Aromatic Hydrocarbons, and EPA Method 608 for pesticides. Three scans are required during the 4 ½ years after permit issuance. Two of the three scans must be performed no fewer than 4 months and no more than 8 months apart. The effluent samples shall be 24-hour daily composites, except where sampling volatile compounds. In this case, six (6) discrete samples (not less than 40 mL) collected over the operating day are acceptable. The permittee shall take special precautions in compositing the individual grab samples for the volatile organics to insure sample integrity (i.e. no exposure to the outside air). Alternately, the discrete samples collected for volatiles may be analyzed separately and averaged.

9. Beginning no later than calendar year 2005, the permittee shall conduct Whole Effluent Toxicity testing for a period of four (4) years in accordance with the frequency specified above. If the Whole Effluent Toxicity tests show that the effluent samples are not toxic at the dilutions determined to occur at the Zone of Immediate Dilution and the Mixing Zone, no further Whole Effluent Toxicity testing will be required during this permit cycle. Note that four Whole Effluent Toxicity test results will be required along with the next NPDES permit renewal application.

10. Biosolids monitoring results shall be tabulated and submitted with the annual biosolids report as required in Schedule B.3.b. Submittal of biosolids monitoring results with the monthly Discharge Monitoring Report is not required.

11. Composite samples from the storage lagoon or pond shall be taken from reference areas in the Storage lagoon or pond pursuant to Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.

    Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and third Edition (1986) with Revision I.

12. Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line (as defined in note 3 above).

13. The days of digestion shall be calculated by dividing the effective digester volume by the average daily volume of sludge production.

14. During the first year after permit issuance, special monitoring for cyanide, cadmium, copper, lead, selenium, silver and zinc shall be conducted on the receiving stream. TSS and hardness shall be monitored simultaneously. The special monitoring for cyanide, cadmium, copper, lead, selenium, silver and zinc shall be conducted using a "clean" sampling method, an "ultra-clean" sampling method, EPA method 1669 or any other test method approved by the Department. After the first year, special monitoring of the receiving stream may be eliminated. For all tests, the method detection limit shall be reported along with the sample result.

SCHEDULE C

Compliance Schedules and Conditions

1. By November 2, 2005, the permittee shall submit to the Department a report which either identifies known sewage overflow locations and a plan for estimating the frequency, duration and quantity of sewage overflowing, or confirms that there are no overflow points. The report shall also provide a schedule to eliminate the overflow(s), if any.

2. Industrial Waste Survey/Pretreatment Program

   a. As soon as practicable, but by February 2, 2005, the permittee shall submit to the Department an industrial waste survey as described in 40 CFR 403.8(f)(2)(i-iii) suitable to make a determination as to the need for development of a pretreatment program.

   b. Should the Department determine that a pretreatment program is required, the permit shall be reopened and modified in accordance with 40 CFR 403.8(e) to incorporate a compliance schedule to require development of a pretreatment program. The compliance schedule requiring program development shall be developed in accordance with the provisions of 40 CFR 403.12(k), and shall not exceed twelve (12) months.

3. By no later than six (6) months after notification that the Molalla-Pudding Total Maximum Daily Load (TMDL) has been approved, the permittee shall submit to the Department an evaluation of whether or not the treatment facilities can consistently comply with the ammonia limitations, any Waste Load Allocation (WLA) established by the TMDL and all other requirements of the TMDL. If the evaluation indicates the permittee is not able to consistently comply with the ammonia limits and TMDL, the permittee shall complete the following schedule:

   a. By no later than one (1) year after notification that the TMDL has been approved, the permittee shall submit to the Department an evaluation of alternatives for facility improvements necessary to comply with the ammonia limits and TMDL.

   b. By no later than two (2) years after notification that the TMDL has been approved, the permittee shall submit to the Department for approval final engineering plans and specifications for any necessary improvements.

   c. By no later than three (3) years after notification that the TMDL has been approved, the permittee shall submit documentation to the Department that contracts for the construction of necessary improvements have been awarded.

   d. By no later than four (4) years after notification that the TMDL has been approved, the permittee shall complete construction of all necessary improvements and comply with the ammonia limits and TMDL.

4. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he/she determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. All biosolids shall be managed in accordance with the current, DEQ approved biosolids management plan, and the site authorization letters issued by the DEQ. Any changes in solids management activities that significantly differ from operations specified under the approved plan require the prior written approval of the DEQ.

   All new biosolids application sites shall meet the site selection criteria set forth in OAR 340-050-0070 and must be located within western Oregon. All currently approved sites are located in Marion County. No new public notice is required for the continued use of these currently approved sites. Property owners adjacent to any newly approved application sites shall be notified, in writing or by any method approved by DEQ, of the proposed activity prior to the start of application. For proposed new application sites that are deemed by the DEQ to be sensitive with respect to residential housing, runoff potential or threat to groundwater, an opportunity for public comment shall be provided in accordance with OAR 340-050-0030.

   Sludge disposal in a Department approved landfill as a solid waste (either in a landfill cell or is used as interim cover) must be in accordance with OAR Chapter 340, Division 93. Proper waste monitoring would be prescribed by the landfill in accordance with that rule. Monitoring of such sludge as biosolids is not required under this permit.

2. This permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under section 405(d) of the Clean Water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

3. **Whole Effluent Toxicity Testing**

   a. The permittee shall conduct whole effluent toxicity tests as specified in Schedule B of this permit.

   b. Whole effluent toxicity (WET) tests may be dual end-point tests, only for the fish tests, in which both acute and chronic end-points can be determined from the results of a single chronic test (the acute end-point shall be based upon a 48-hour time period).

   c. Acute Toxicity Testing - Organisms and Protocols

      (1) The permittee shall conduct 48-hour static renewal tests with the *Ceriodaphnia dubia* (water flea) and the *Pimephales promelas* (fathead minnow).

      (2) The presence of acute toxicity will be determined as specified in **Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms**, Fourth Edition, EPA/600/4-90/027F, August 1993.

      (3) An acute WET tests shall be considered to show toxicity if there is a statistically significant difference in survival between the control and 100 percent effluent, unless the permit specifically provides for a Zone of Immediate Dilution (ZID) for toxicity. If the permit specifies such a ZID, acute toxicity shall be indicated when a statistically significant difference in survival occurs at dilutions greater than that which is found to occur at the edge of the ZID.

   d. Chronic Toxicity Testing - Organisms and Protocols

      (1) The permittee shall conduct tests with: *Ceriodaphnia dubia* (water flea) for reproduction and survival test endpoint, *Pimephales promelas* (fathead minnow) for growth and survival test endpoint, and *Raphidocelis subcapitata* (green alga formerly known as *Selanastrum capricornutum*) for growth test endpoint.

(2) The presence of chronic toxicity shall be estimated as specified in **Short-Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms**, Third Edition, EPA/600/4-91/002, July 1994.

(3) A chronic WET tests shall be considered to show toxicity if a statistically significant difference in survival, growth, or reproduction occurs at dilutions greater than that which is known to occur at the edge of the mixing zone. If there is no dilution data for the edge of the mixing zone, any chronic WET test that shows a statistically significant effect in 100 percent effluent as compared to the control shall be considered to show toxicity.

e. Quality Assurance

(1) Quality assurance criteria, statistical analyses and data reporting for the WET tests shall be in accordance with the EPA documents stated in this condition and the Department's **Whole Effluent Toxicity Testing Guidance Document**, January 1993.

f. Evaluation of Causes and Exceedances

(1) If toxicity is shown, as defined in sections c.(3) or d.(3) of this permit condition, another toxicity test using the same species and Department approved methodology shall be conducted within two weeks, unless otherwise approved by the Department. If the second test also indicates toxicity, the permittee shall follow the procedure described in section f.(2) of this permit condition.

(2) If two consecutive WET test results indicate acute and/or chronic toxicity, as defined in sections c.(3) or d.(3) of this permit condition, the permittee shall evaluate the source of the toxicity and submit a plan and time schedule for demonstrating compliance with water quality standards. Upon approval by the Department, the permittee shall implement the plan until compliance has been achieved. Evaluations shall be completed and plans submitted to the Department within 6 months unless otherwise approved in writing by the Department.

g. Reporting

(1) Along with the test results, the permittee shall include: 1. the dates of sample collection and initiation of each toxicity test; 2. the type of production; and 3. the flow rate at the time of sample collection. Effluent at the time of sampling for WET testing should include samples of required parameters stated under Schedule B, condition 1. of this permit.

(2) The permittee shall make available to the Department, on request, the written standard operating procedures they, or the laboratory performing the WET tests, are using for all toxicity tests required by the Department.

h. Reopener

(1) If WET testing indicates acute and/or chronic toxicity, the Department may reopen and modify this permit to include new limitations and/or conditions as determined by the Department to be appropriate, and in accordance with procedures outlined in Oregon Administrative Rules, Chapter 340, Division 45.

4. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification and grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

b. The permittee's wastewater system may not be without supervision (as required by Special Condition 4.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

d. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

e. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program, 811 SW 6th Ave, Portland, OR 97204. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required by 4.b. above.

5. The permittee shall notify the DEQ Western Region - Salem Office (phone: (503) 378-8240) in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

6. The permittee shall not be required to perform a hydrogeologic characterization or groundwater monitoring during the term of this permit provided:

a. The facilities are operated in accordance with the permit conditions, and;

b. There are no adverse groundwater quality impacts (complaints or other indirect evidence) resulting from the facility's operation.

If warranted, at permit renewal the Department may evaluate the need for a full assessment of the facilities impact on groundwater quality.

7. All reclaimed water used at the treatment plant site for landscape irrigation shall be exempt from OAR 340-055 provided the reclaimed water receives secondary treatment and disinfection. All landscape irrigation shall be confined to the treatment plant site. No spray or drift shall be allowed off the treatment plant site. Landscape irrigation shall be conducted following sound irrigation practices.

City of Silverton

Wastewater System Facility Master Plan

Appendix B

ONE COMPANY
Many Solutions℠

Telephone Record

Project:	Silverton Facilities Plan	Project No:	39068
Date:	September 13, 2006	Subject:	Acceptance of sludge/biosolids
Call to:	Marion County Waste-to-Energy Facility	Phone No:	503-393-0890 x214 (Darby R.)
Call from:	Greg Moen, HDR	Phone No:	425-450-6222

Discussion, Agreement and/or Action:

Spoke with Lori Wallace, who referred me to Darby Rancliff(spelling?).

Are biosolids accepted? Darby was not sure, and would have to check with their staff.

Darby called back September 25, 2006:

No biosolids are currently accepted. Staff are unwilling to accept them at this time due to concerns about operational and maintenance impacts on the incineration equipment.

HDR | ONE COMPANY *Many Solutions*℠

Telephone Record

Project:	Silverton Facilities Plan	Project No:	39068
Date:	October 10, 2006	Subject:	Acceptance of biosolids
Call to:	Brown's Island Demolition Landfill	Phone No:	503-588-5169
Call from:	Greg Moen, HDR	Phone No:	425-450-6222

Discussion, Agreement and/or Action:

Left a message with Jeff Bickford:

1. Are treated biosolids accepted? No

Jeff recommended the Coffin Butte landfill, which is privately operated, as an alternative disposal site.

Telephone Record

Project:	Silverton Facilities Plan	Project No:	39068
Date:	October 11, 2006	Subject:	Acceptance of biosolids
Call to:	Coffin Butte Landfill	Phone No:	1-800-204-4242 x116
Call from:	Greg Moen, HDR	Phone No:	425-450-6222

Discussion, Agreement and/or Action:

Spoke with Joe Griffith, Allied Waste, at 11:00 a.m.

1. Are treated biosolids accepted? Yes

2. Analytical requirements? Paint Filter Test and other testing. Joe will send me a form via email to be filled out by the City.

3. Long-term contract? Standard contract for one-year. However, customer can develop their own contract with longer period if desired.

Mark Arena (sales) will get back to me regarding tipping fees.

Telephone Record

Project:	Silverton Facilities Plan	Project No:	39068
Date:	October 11, 2006	Subject:	Acceptance of biosolids
Call to:	Coffin Butte Landfill	Phone No:	503-288-1234
Call from:	Greg Moen, HDR	Phone No:	425-450-6222

Discussion, Agreement and/or Action:

Spoke with Melissa, 11:30 a.m.

1. Are treated biosolids accepted? Yes

2. Are treated biosolids accepted as alternative daily cover (ADC)? No, Oregon DEQ requires a one-year trial with the material before it can accepted.

3. Analytical requirements? Paint Filter Test and other testing. Need to talk with Joe Griffith at the corporate office for specific requirements and forms. (1-800-204-4242)

4. Are untreated biosolids accepted? Need to talk with Joe Griffith at the corporate office for specific requirements and forms. (1-800-204-4242)

City of Silverton

Wastewater System Facility Master Plan

Appendix C

ALLIED WASTE

GENERATOR WASTE PROFILE SHEET

Waste Profile #

Requested Disposal Facility: ______________________
an Allied Waste Company

I. Generator Information

Generator Name:		Date:	
Generator Site Address:			
City:	County:	State:	Zip:
Generator State ID Number:		SIC Code Number:	
Generator Mailing Address (if different):			
City:	County:	State:	Zip:
Generator Contact Name:			
Phone Number:		Fax Number:	

II. Transporter Information

Transporter Name:			
Transporter Address:			
City:	County:	State:	Zip:
Transporter Contact Name:			
Phone Number:		Fax Number:	
State Transportation Number:			

III. Waste Stream Information

Name of Waste:	
Process Generating Waste:	
Type of Waste:	☐ INDUSTRIAL PROCESS WASTE or ☐ POLLUTION CONTROL WASTE
Physical State:	☐ SOLID ☐ SEMI-SOLID ☐ POWDER ☐ LIQUID ☐ OTHER: _____
Method of Shipment:	☐ BULK ☐ DRUM ☐ BAGGED ☐ OTHER: _____
Estimated Annual Volume:	☐ CUBIC YARDS: _____ ☐ TONS: _____ ☐ OTHER: _____
Frequency:	☐ ONE TIME ☐ DAILY ☐ WEEKLY ☐ MONTHLY ☐ OTHER: _____
Special Handling Instructions:	

IV. Representative Sample Certification

☐ NO SAMPLE TAKEN

Is the representative sample collected to prepare this profile and laboratory analysis, collected in accordance with U.S. EPA 40 CFR 261.20(c) guidelines or equivalent rules?	☐ YES or ☐ NO
Sample Date:	Type of Sample: ☐ COMPOSITE SAMPLE ☐ GRAB SAMPLE
Sampler's Employer:	
Sampler's Name (printed):	Signature:

ALLIED WASTE

GENERATOR WASTE PROFILE SHEET (continued)

Waste Profile #

V. Physical Characteristics of Waste

Characteristic Components % by Weight (range)

1.

2.

3.

Color:	Odor (describe):	Free Liquids: ☐ YES or ☐ NO Content _____%	% Solids:	pH:	Flash Point: _____ °F	Phenol _____ppm

Attach Laboratory Analytical Report (and/or Material Safety Data Sheet)
Including Required Parameters Provided for this Profile

Does this waste or generating process contain regulated concentrations of the following Pesticides and/or Herbicides: Chlordane, Endrin, Heptachlor (and it epoxides), Lindane, Methoxychlor, Toxaphene, 2,4-D, or 2,4,5-TP Silvex as defined in 40 CFR 261.33?	☐ YES or ☐ NO
Does this waste or generating process cause it to exceed OSHA exposure limits from high levels of Hydrogen Sulfide or Hydrogen Cyanide as defined in 40 CFR 261.23?	☐ YES or ☐ NO
Does this waste contain regulated concentrations of Polychlorinated Biphenyls (PCBs) as defined in 40 CFR Part 761?	☐ YES or ☐ NO
Does this waste contain regulated concentrations of listed hazardous wastes defined in 40 CFR 261.31, 261.32, 261.33, including RCRA F-Listed Solvents?	☐ YES or ☐ NO
Does this waste contain regulated concentrations of 2,3,7,8-Tetrachlorodibenzodioxin (2,3,7,8-TCCD), or any other dioxin as defined in 40 CFR 261.31?	☐ YES or ☐ NO
Is this a regulated Toxic Material as defined by Federal and/or State regulations?	☐ YES or ☐ NO
Is this a regulated Radioactive Waste as defined by Federal and/or State regulations?	☐ YES or ☐ NO
Is this a regulated Medical or Infectious Waste as defined by Federal and/or State regulations?	☐ YES or ☐ NO
Is this waste generated at a Federal Superfund Clean Up Site?	☐ YES or ☐ NO

VI. Generator Certification

I hereby certify that to the best of my knowledge and belief, the information contained herein is a true and accurate description of the waste material being offered for disposal. I further certify that by utilizing this profile, neither myself nor any other employee of the company will deliver for disposal or attempt to deliver for disposal any waste which is classified as toxic waste, hazardous waste or infectious waste, or any other waste material this facility is prohibited from accepting by law. Our company hereby agrees to fully indemnify this disposal facility against any damages resulting from this certification being inaccurate or untrue. I further certify that the company has not altered the form or content of this profile sheet as provided by Allied Waste Industries, Inc.

AUTHORIZED REPRESENTATIVE NAME AND TITLE (Printed) COMPANY NAME

AUTHORIZED REPRESENTATIVE SIGNATURE DATE

VII. Allied Waste Decision

☐ Approved ☐ Rejected Expiration: ______________________________

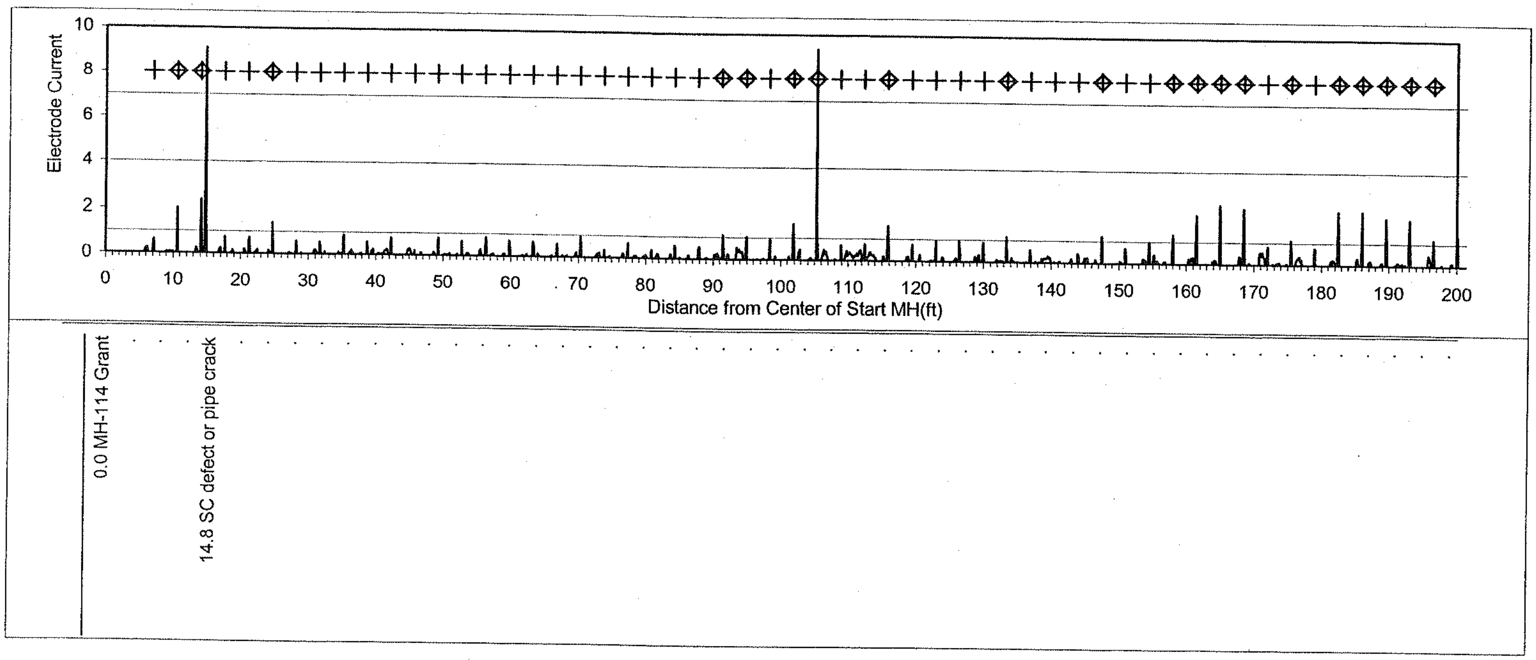
Conditions:

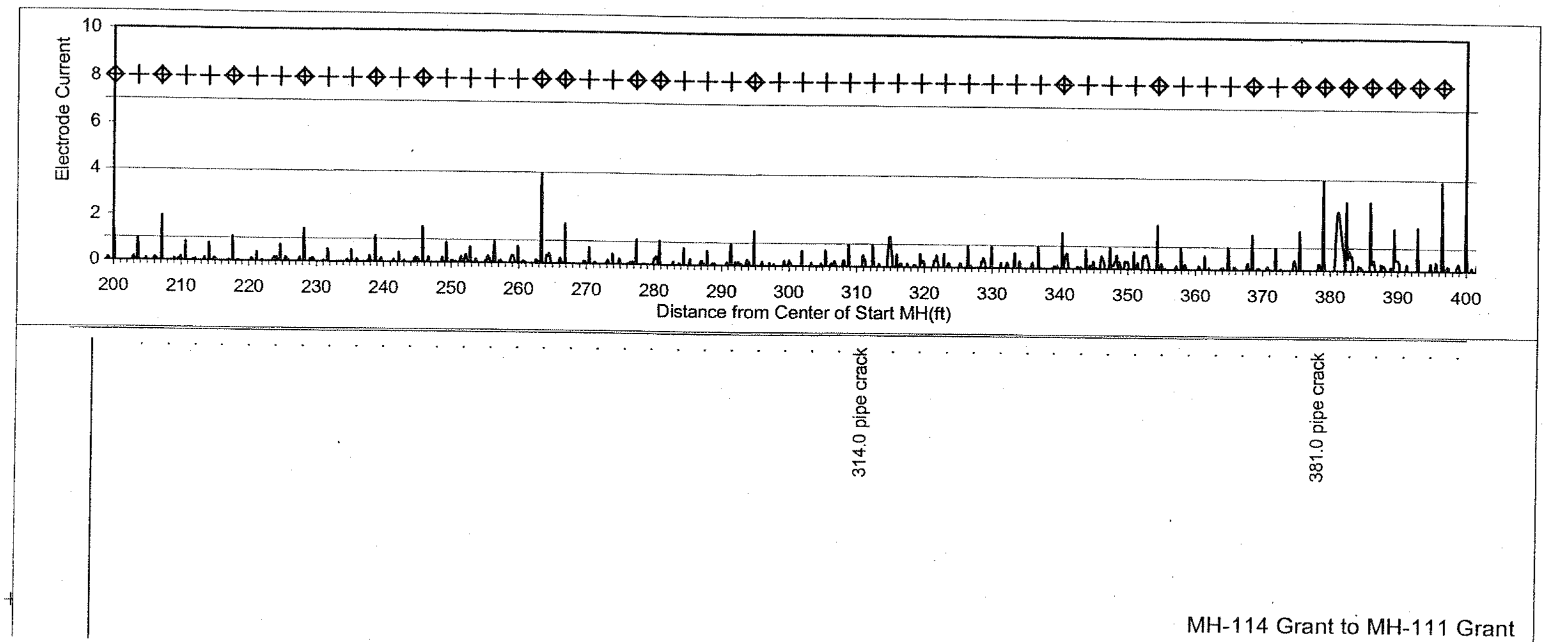
Name, Title Signature Date

City of Silverton

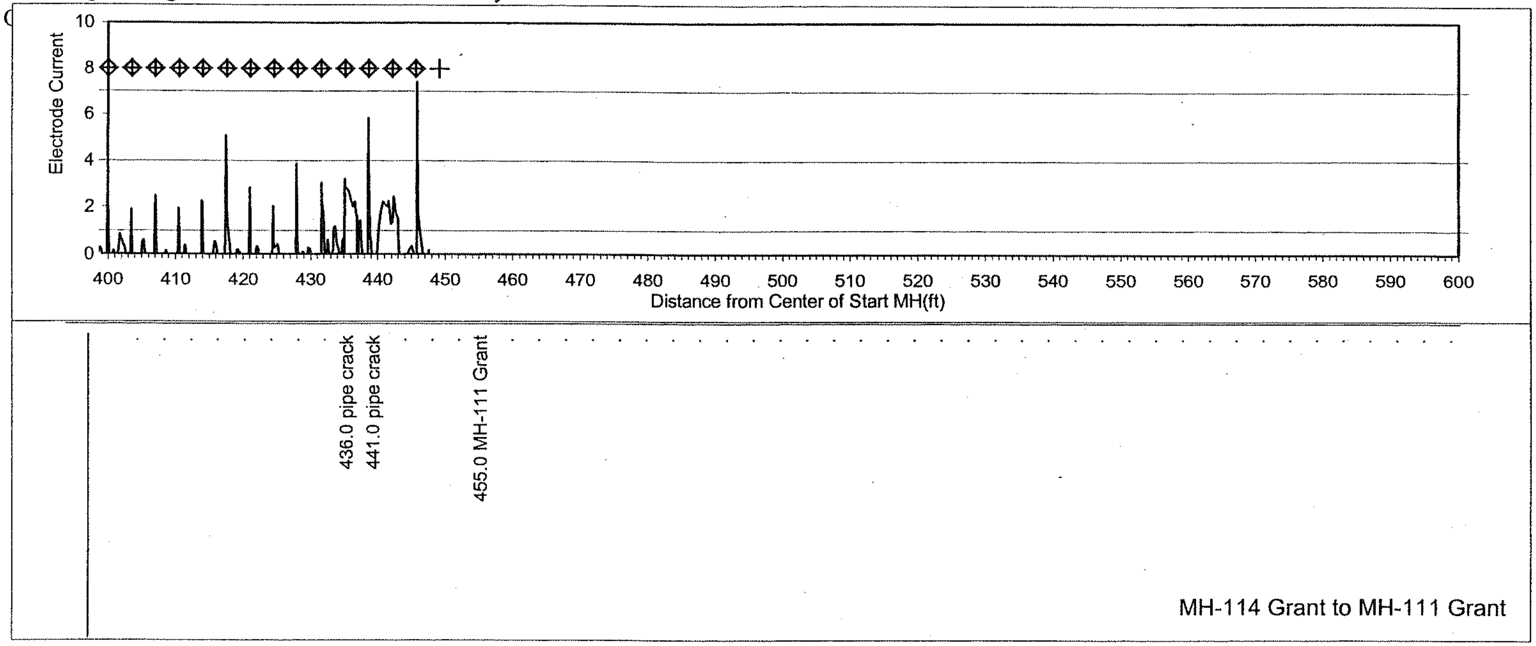
Wastewater System Facility Master Plan

Appendix D





MH-114 Grant to MH-111 Grant



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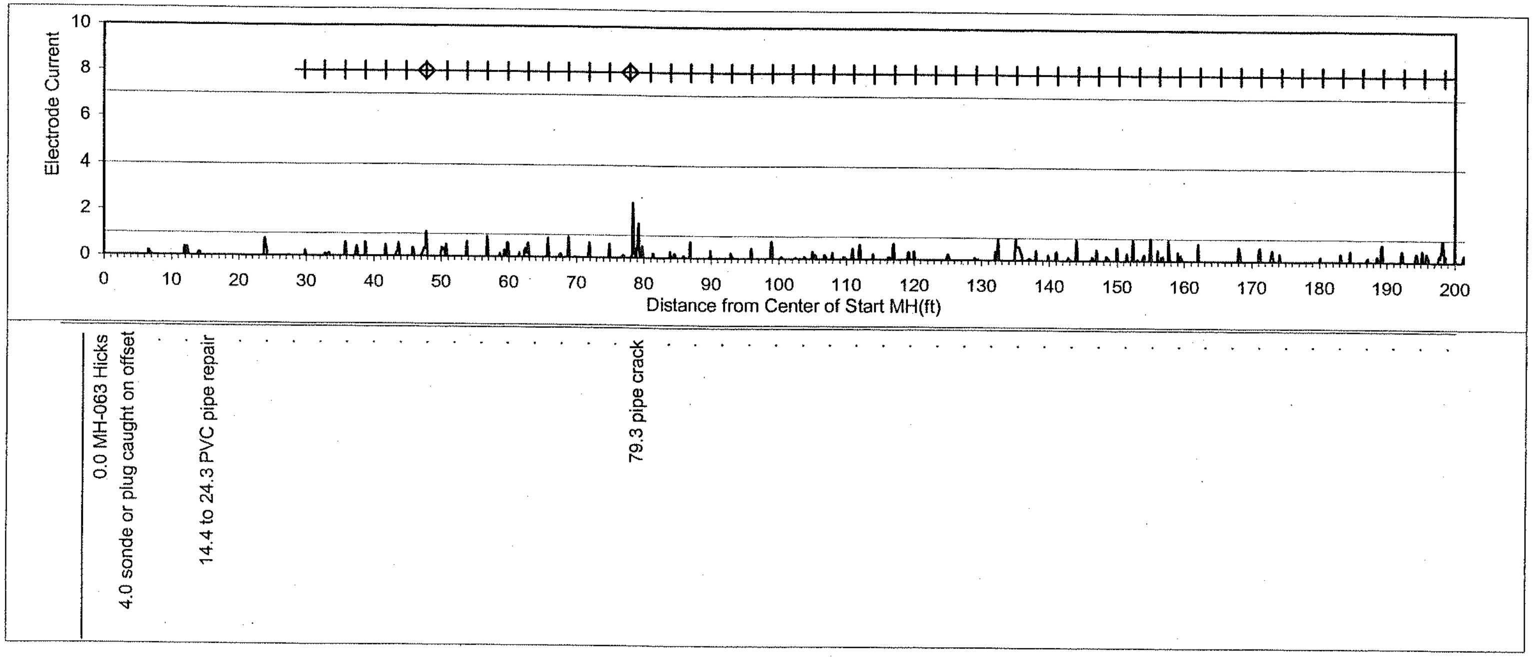

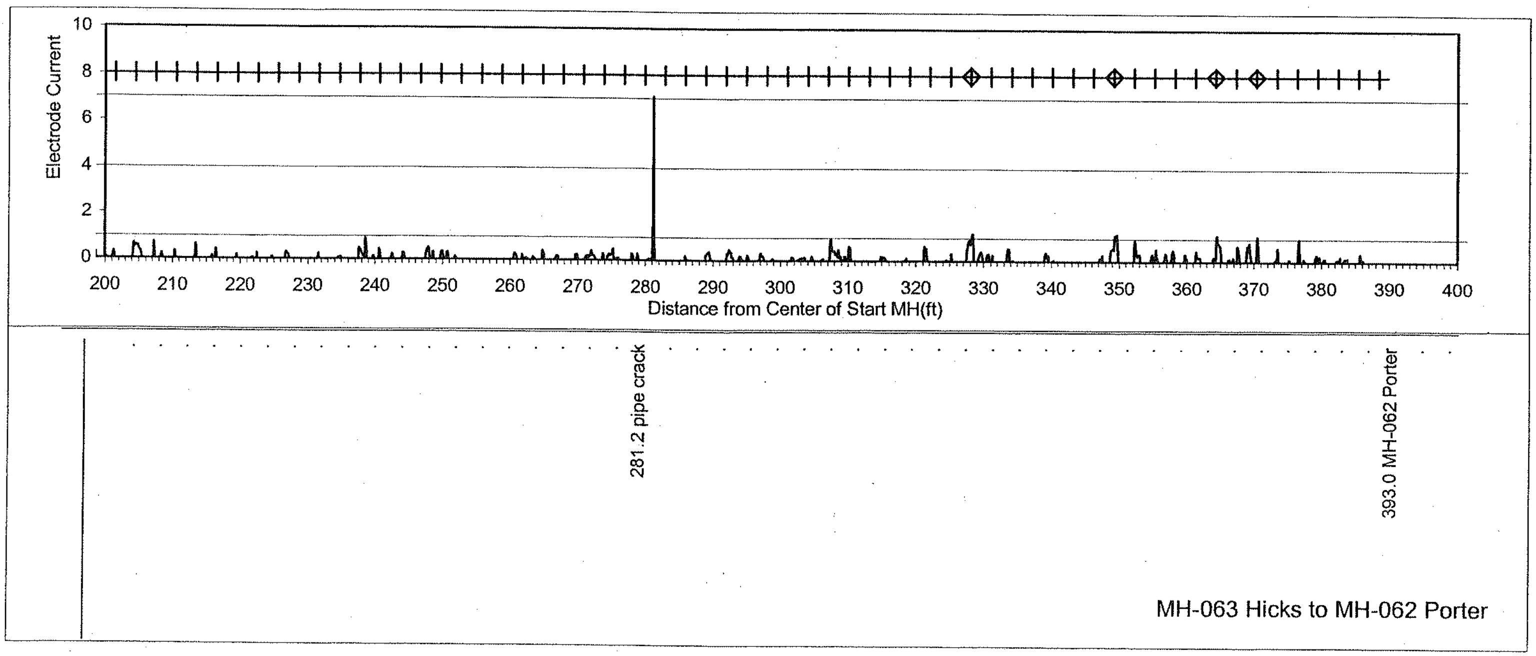

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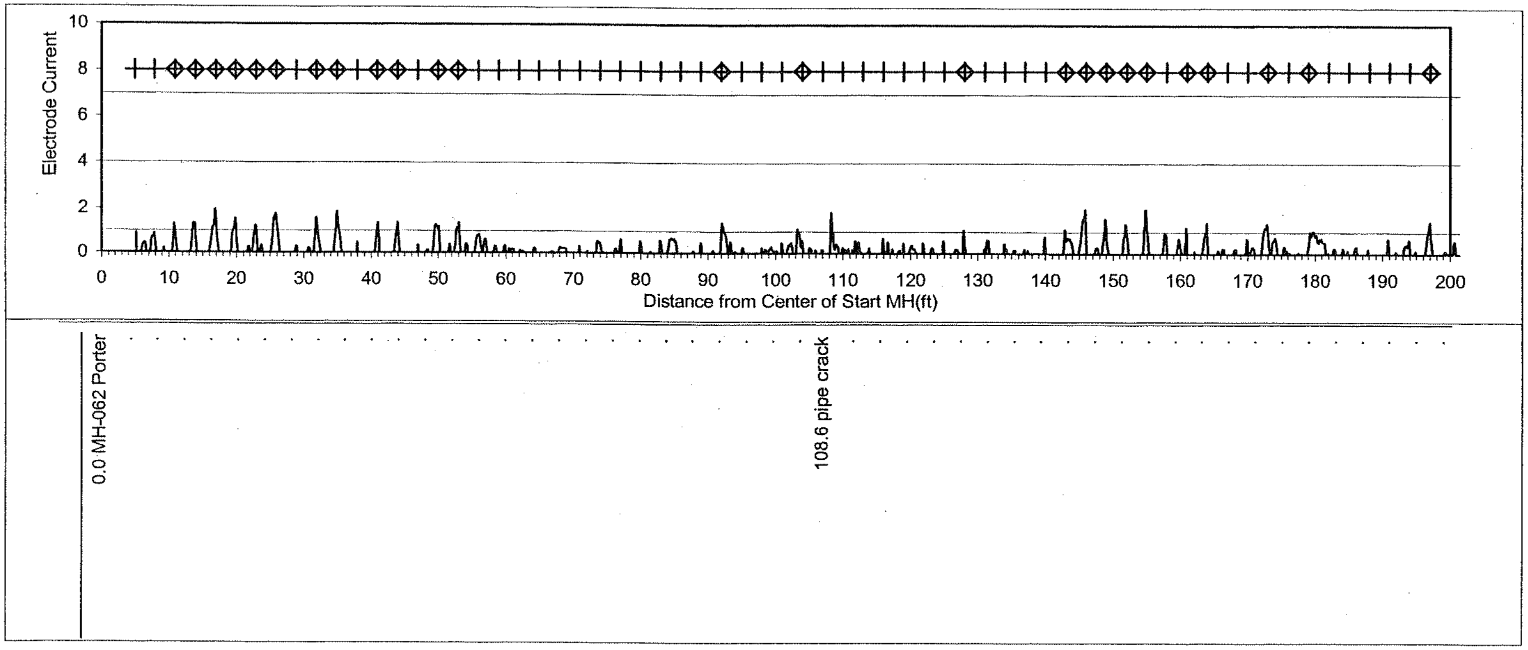

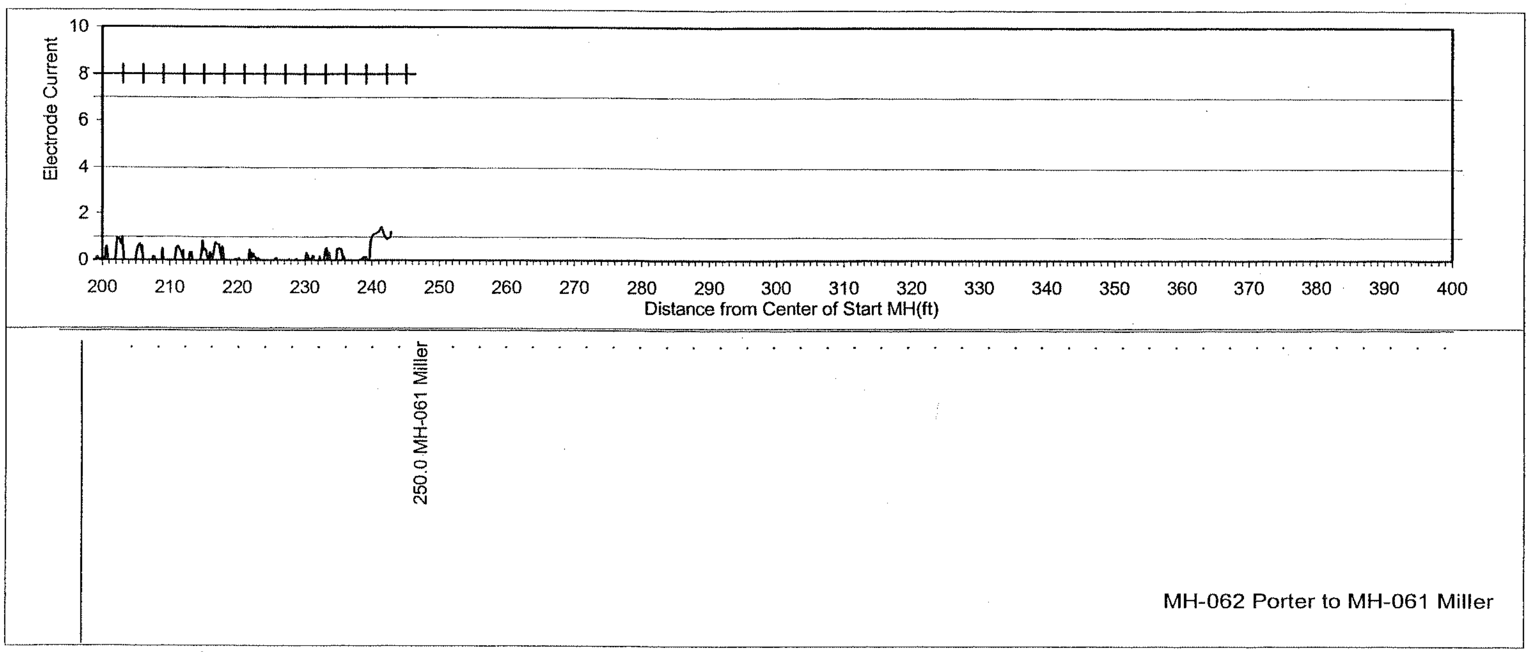

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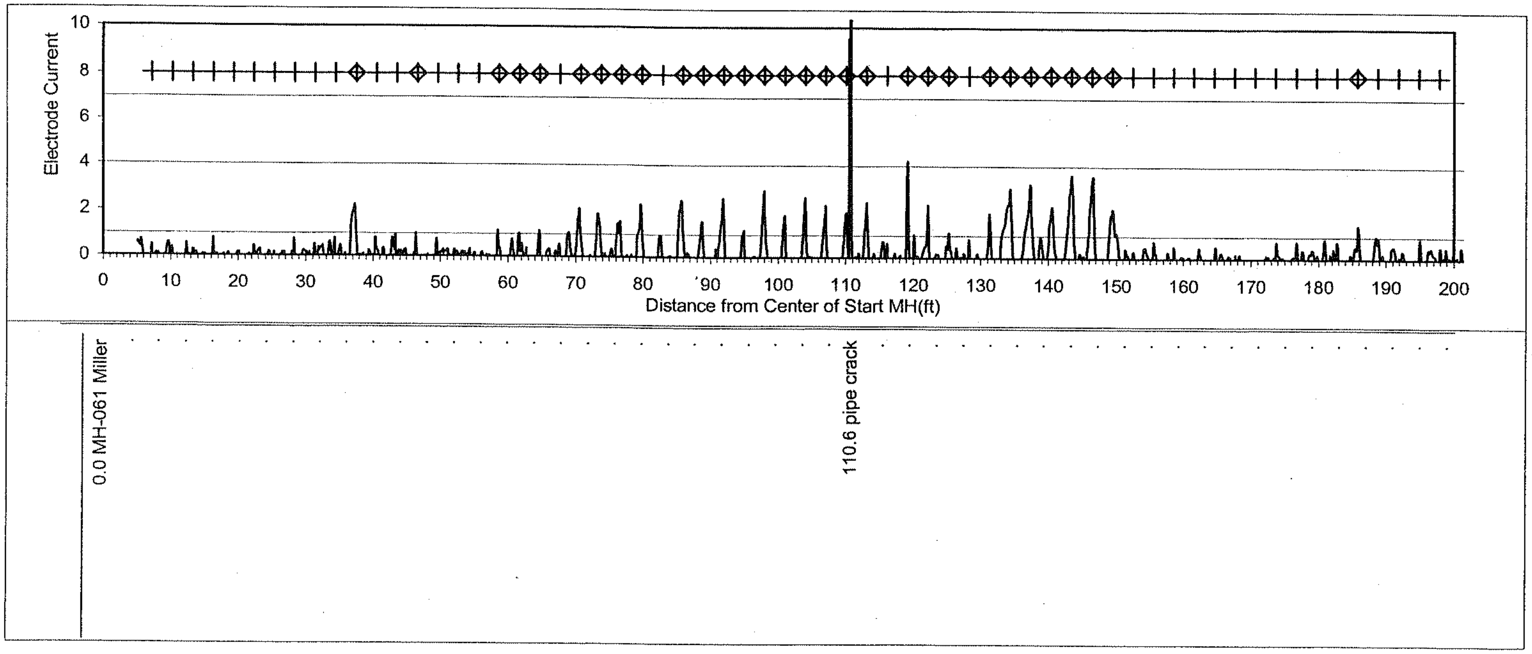

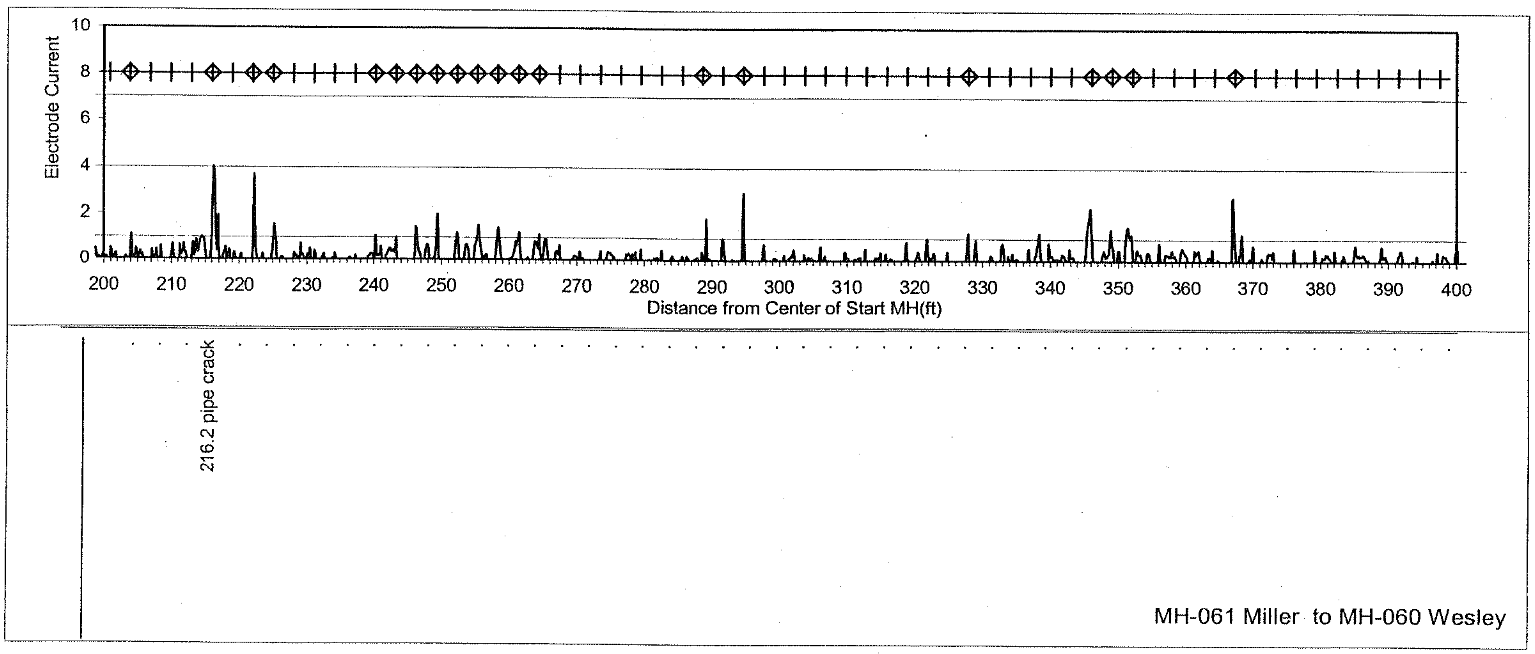

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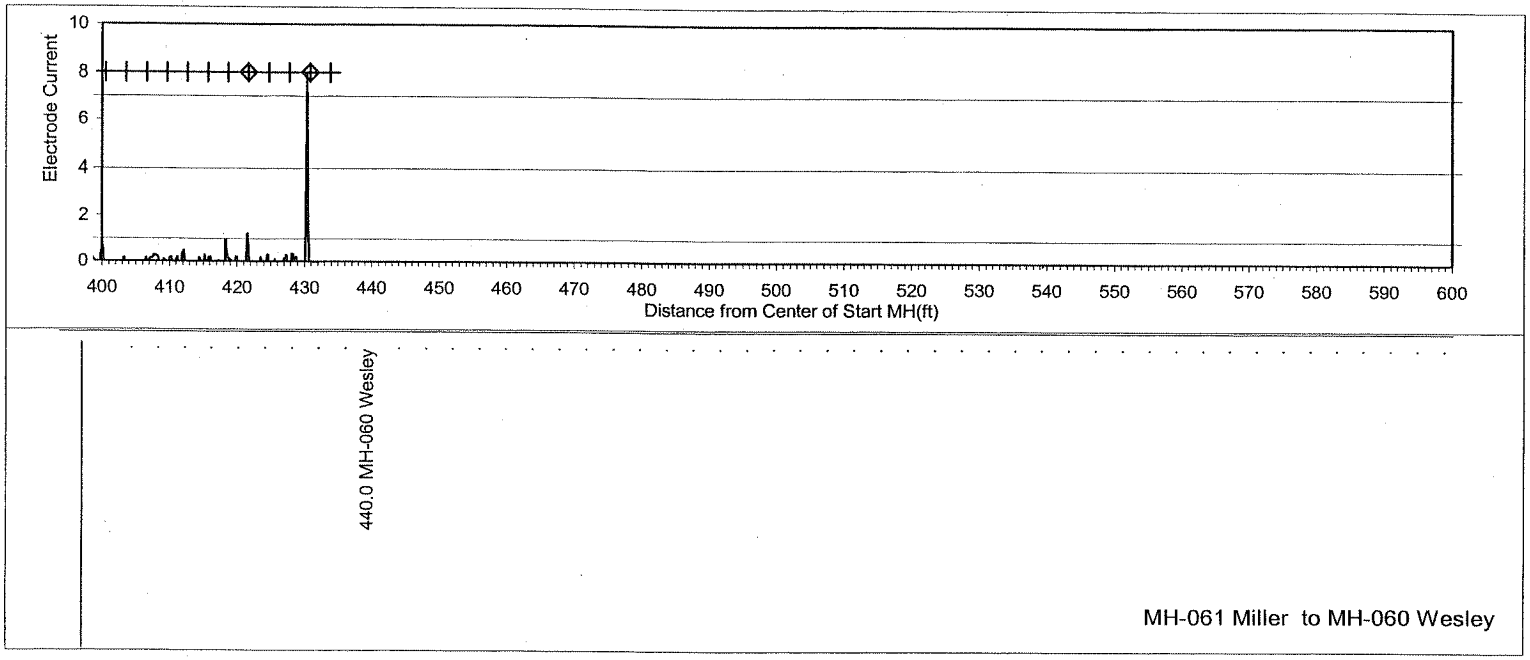

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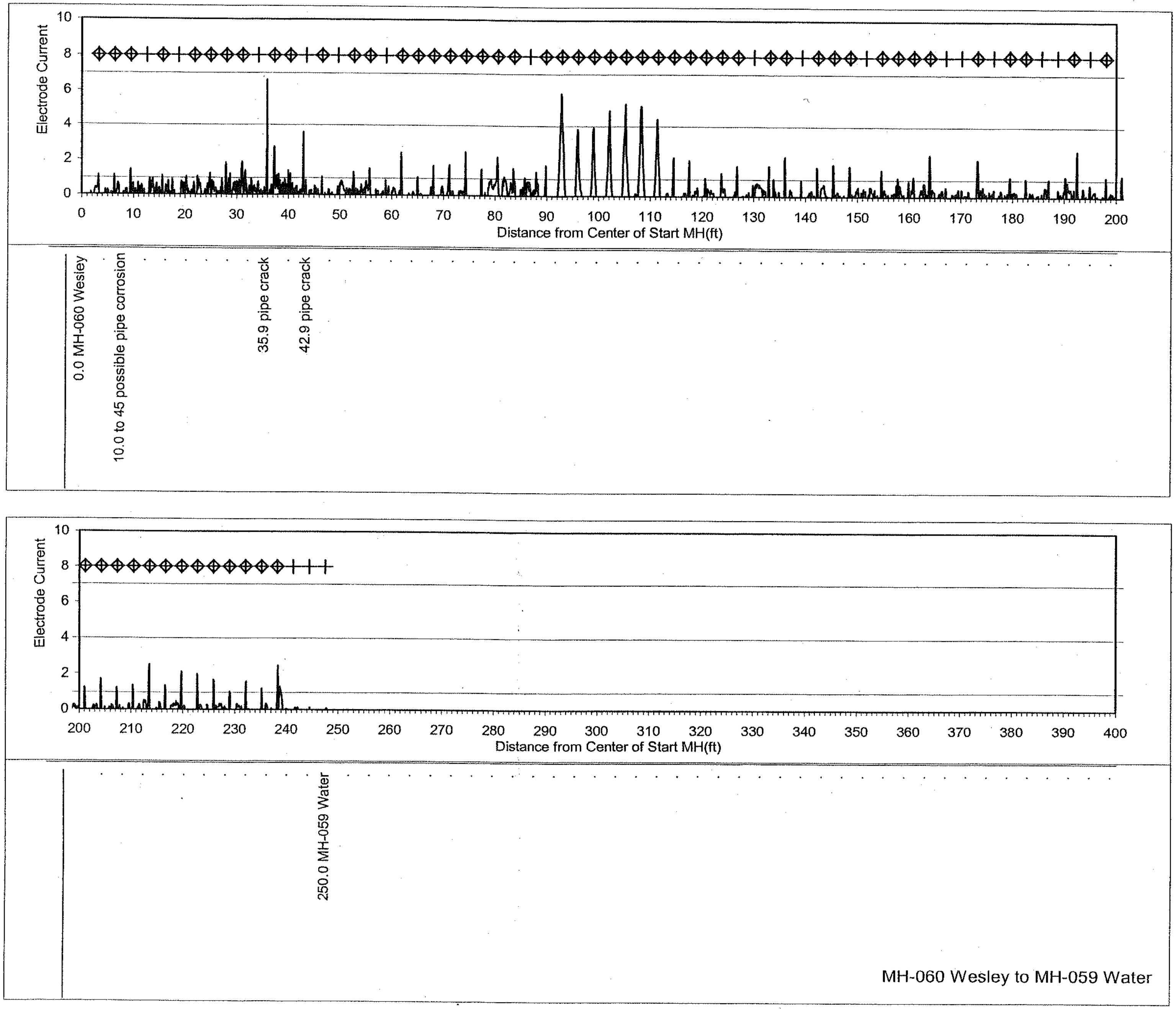

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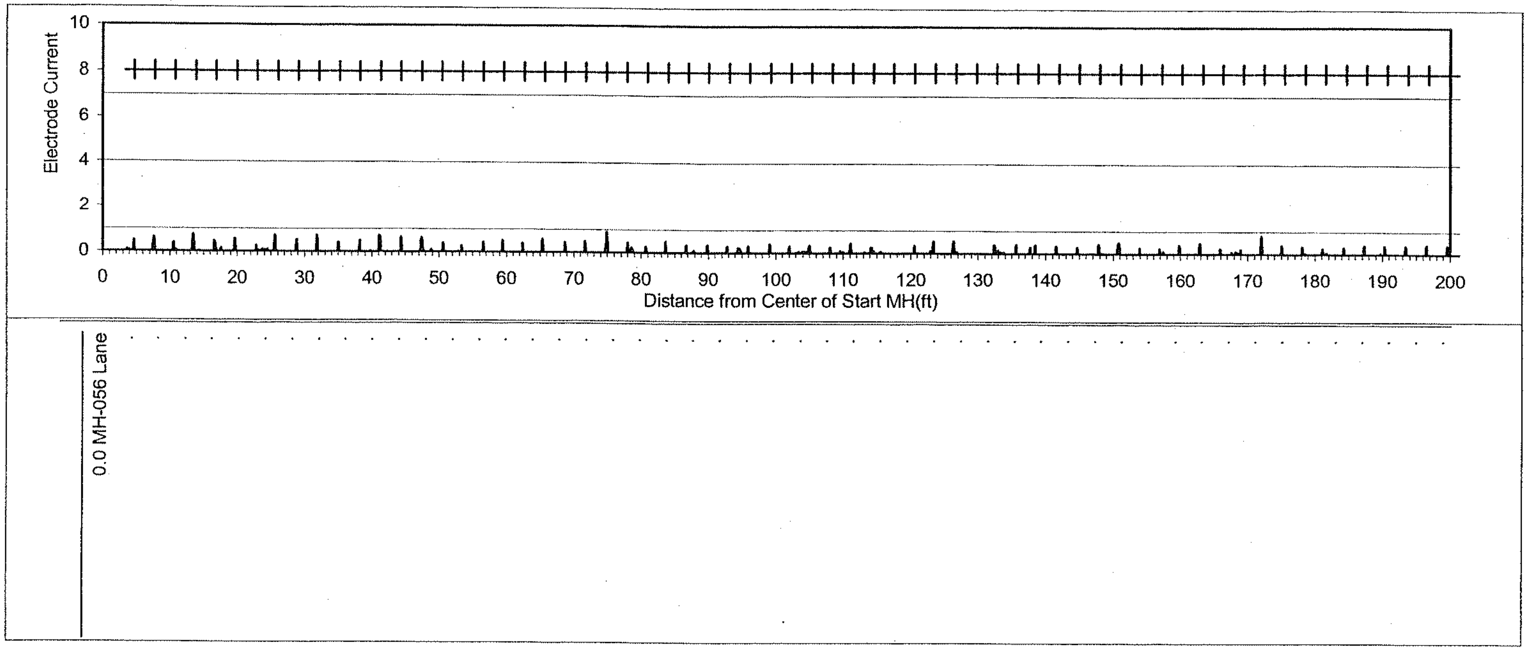

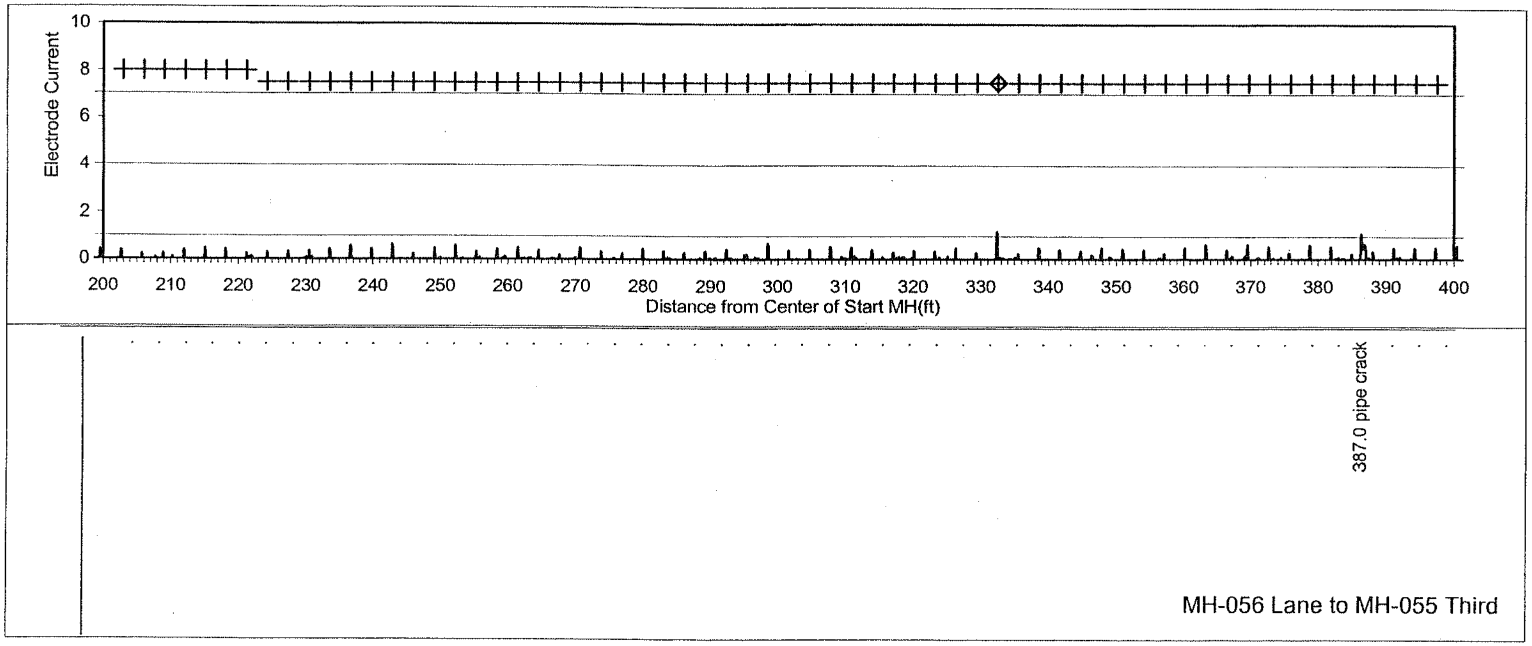

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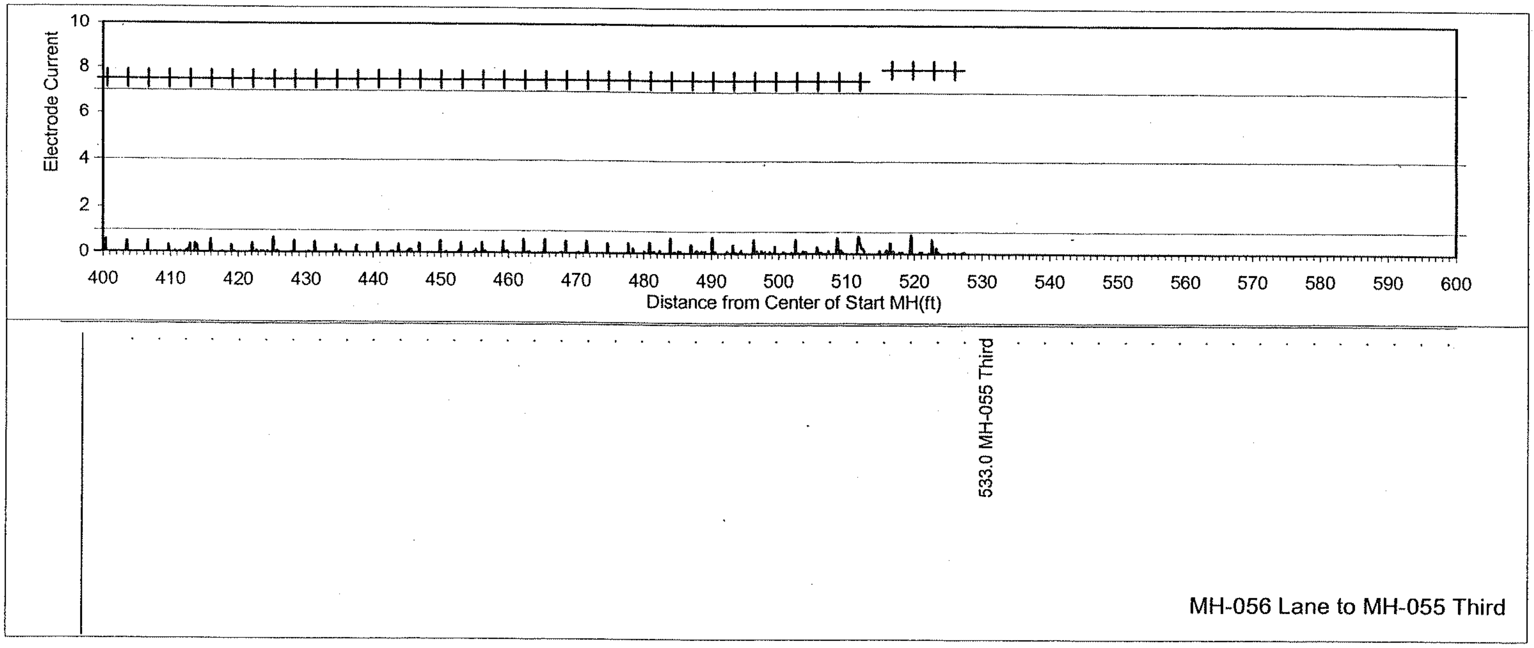

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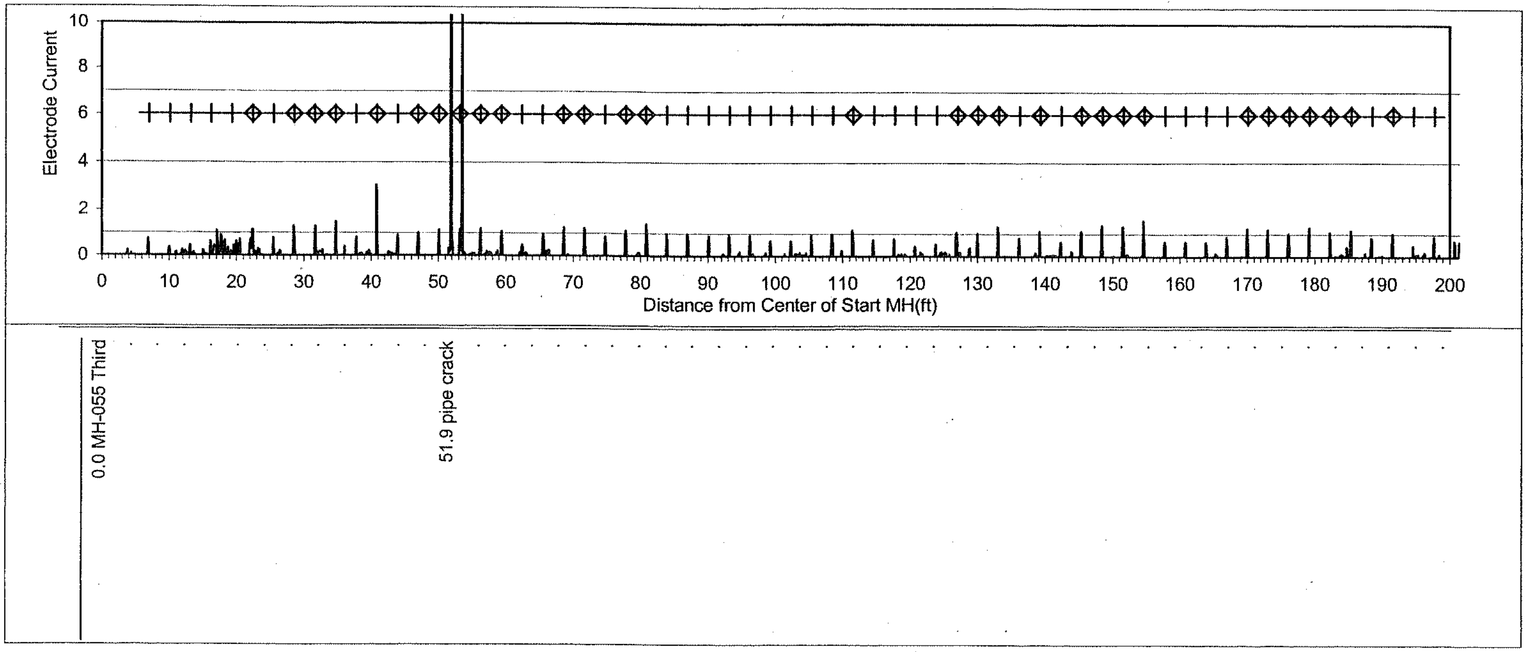

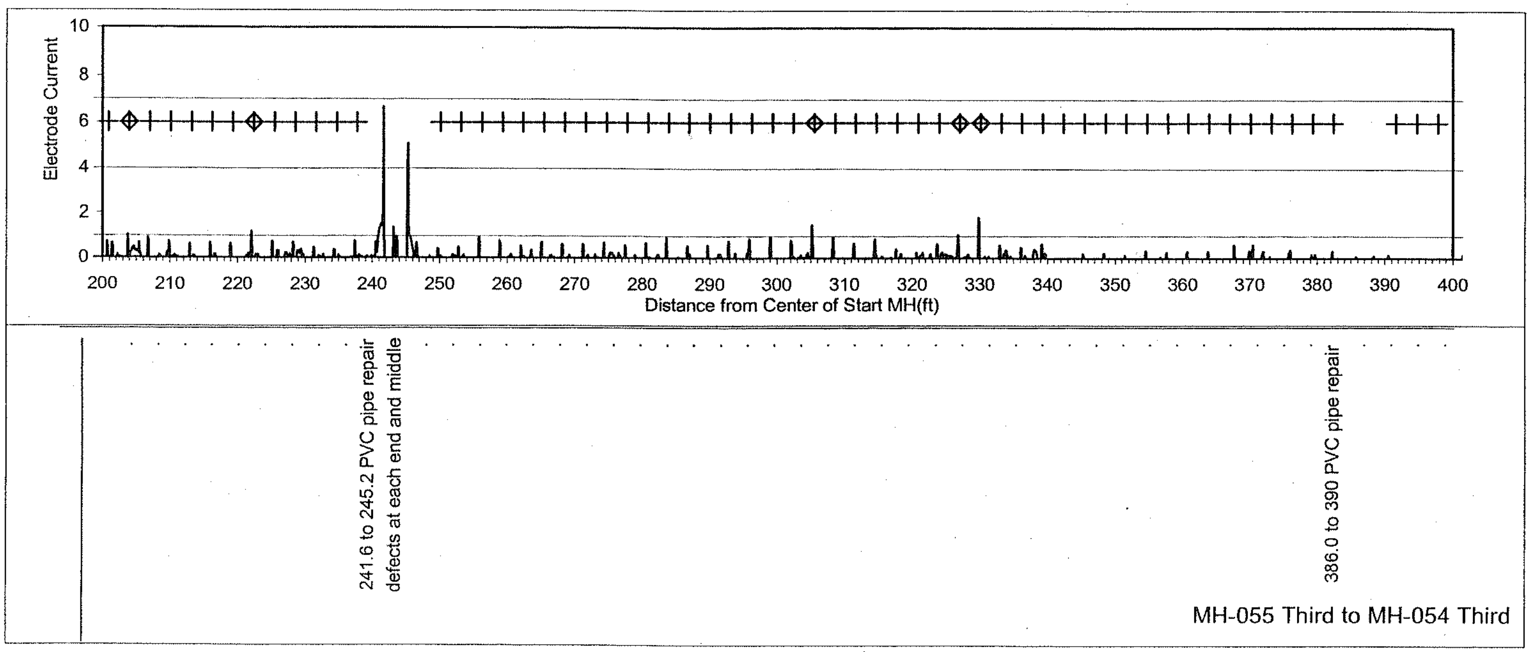

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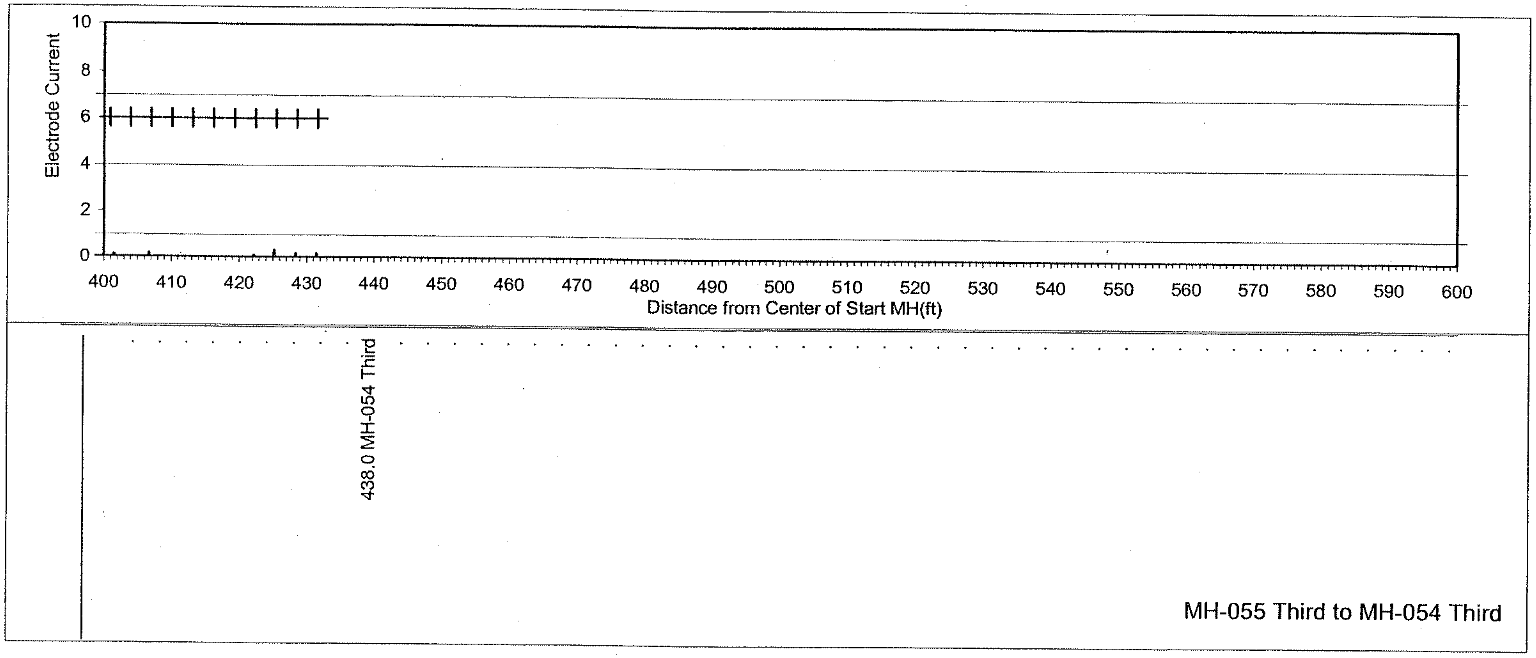

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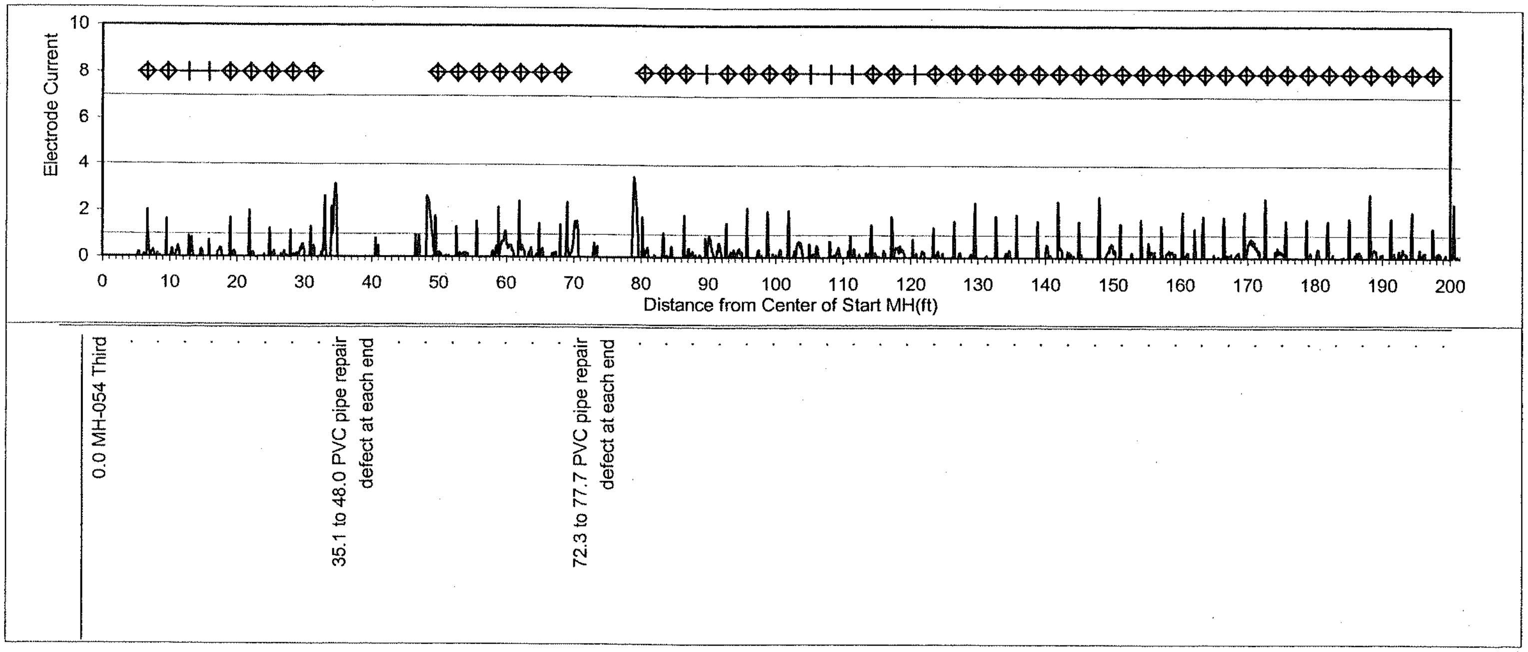

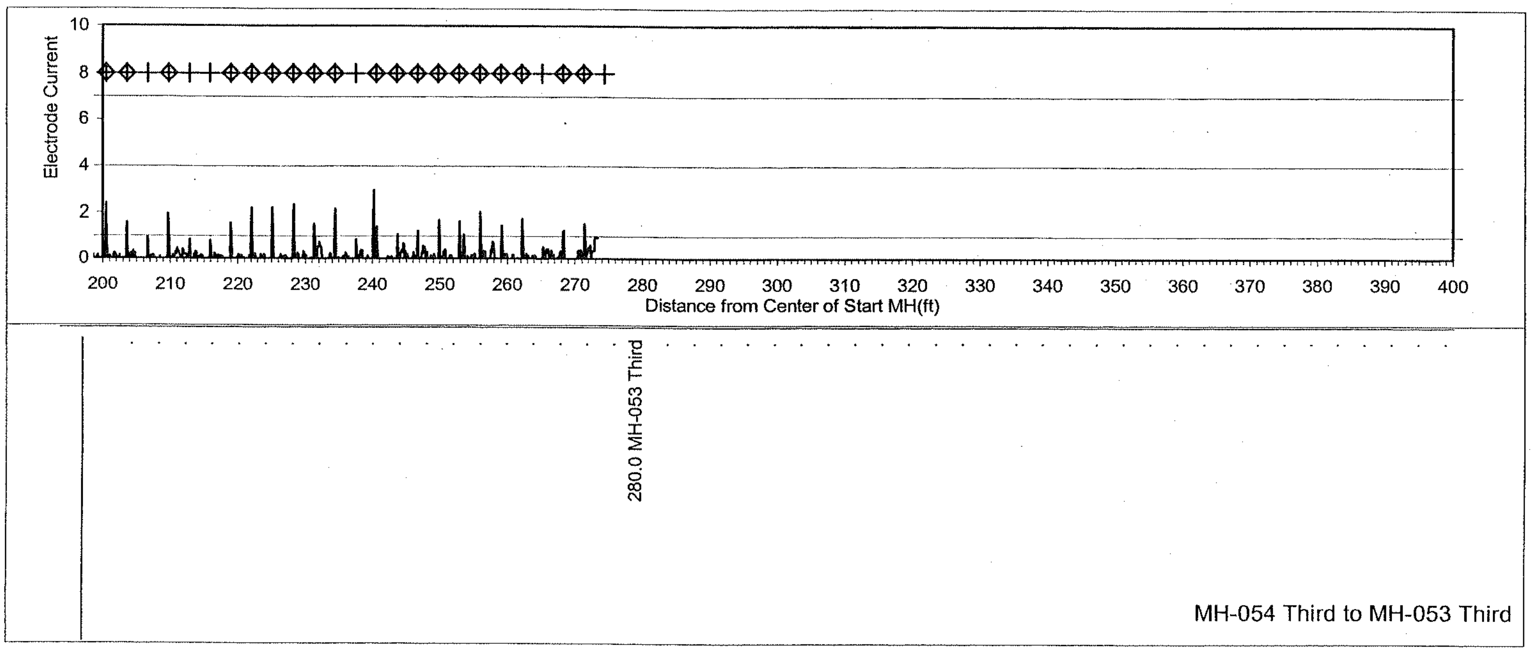

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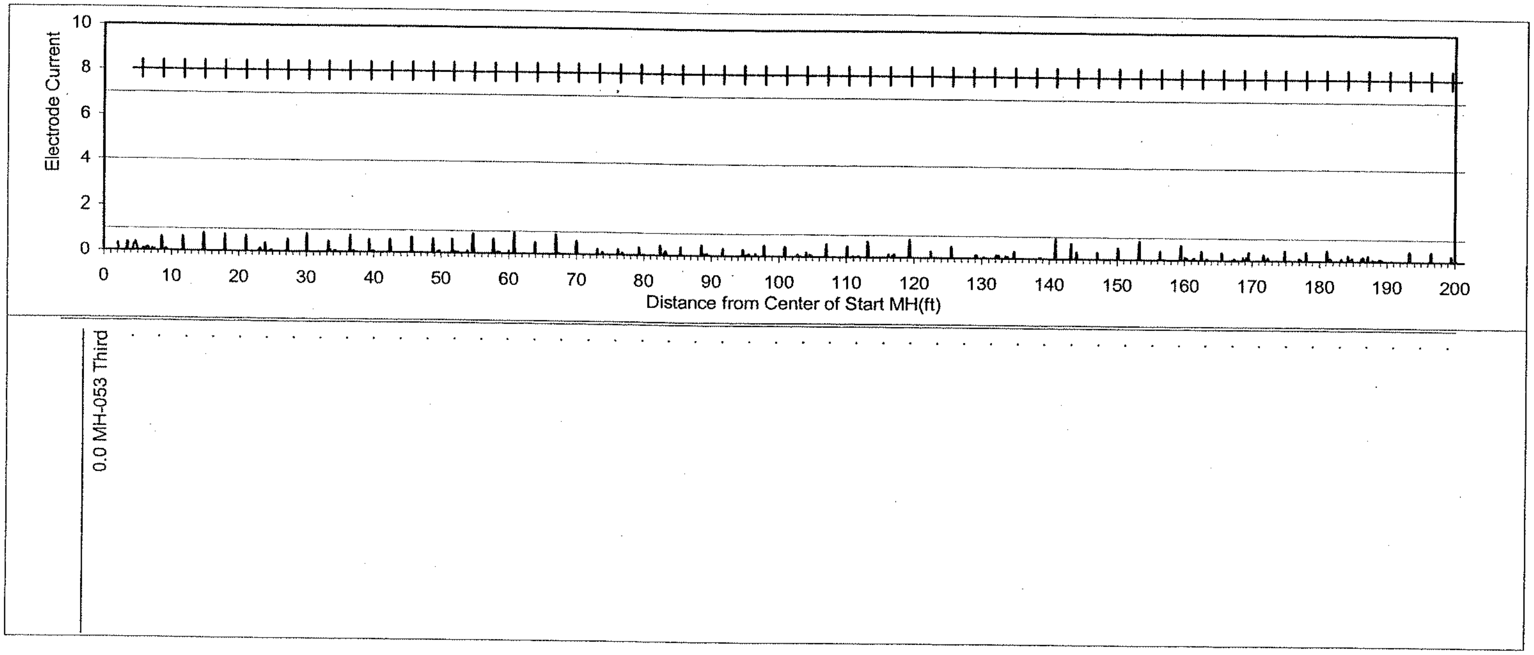

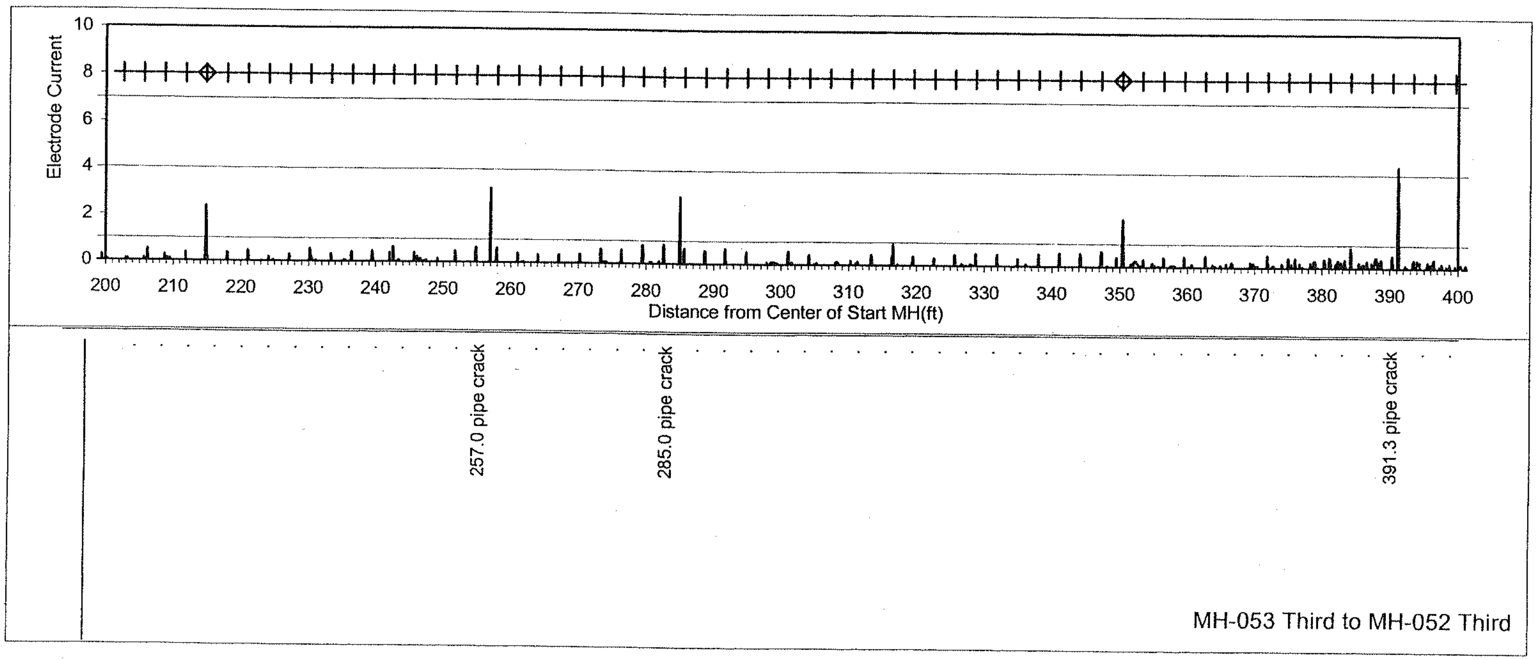

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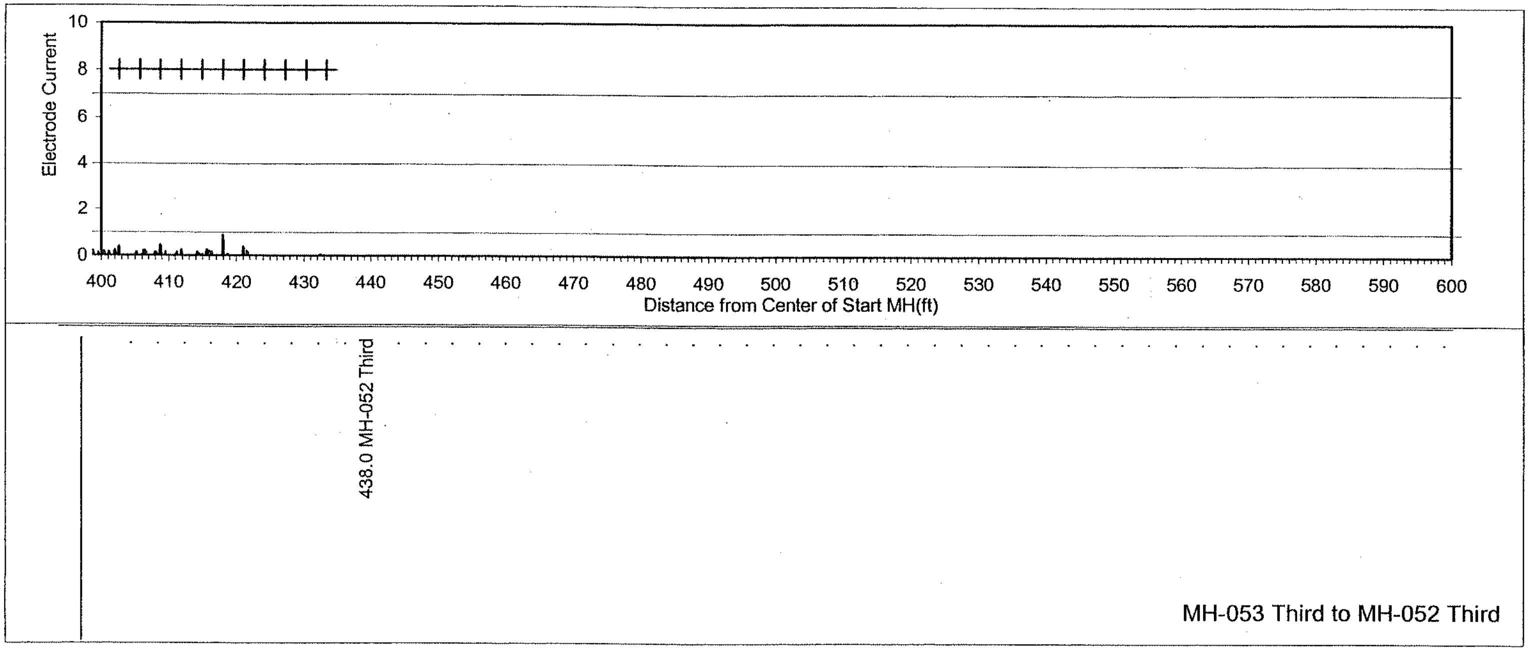

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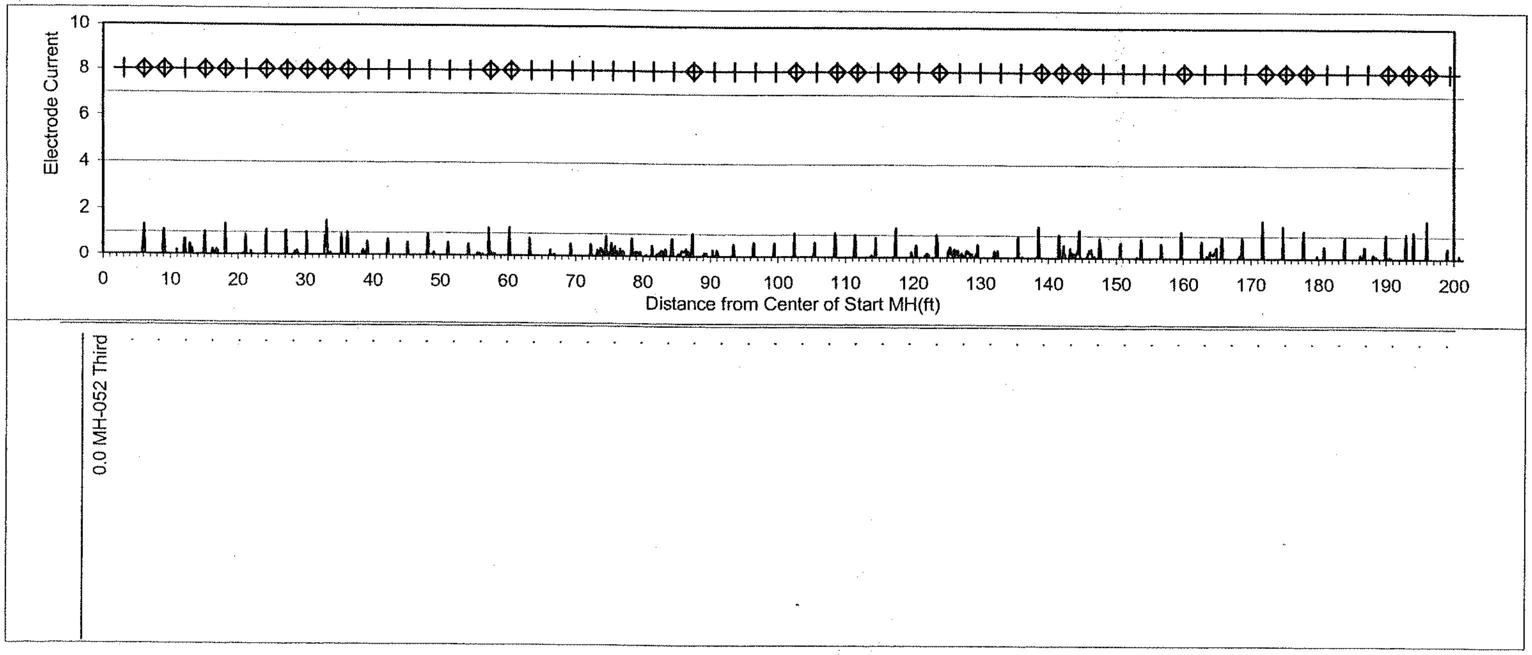

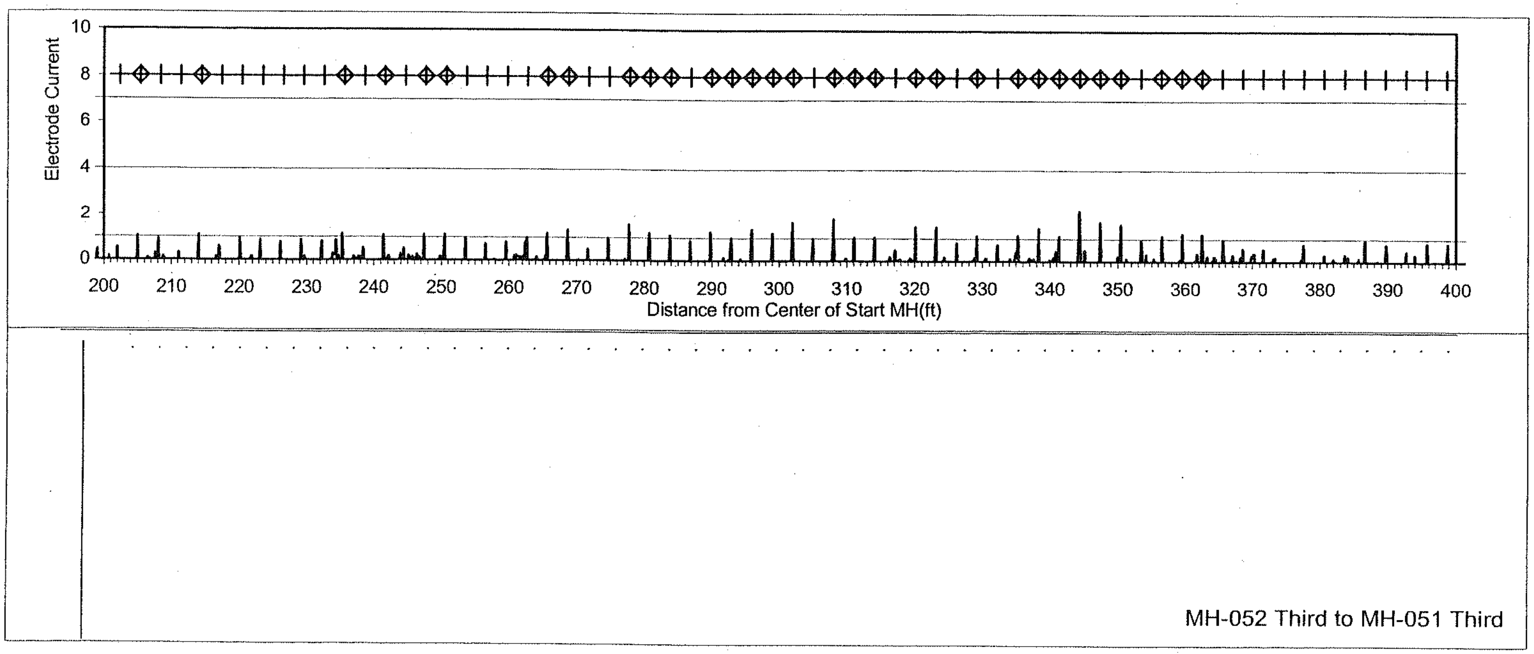

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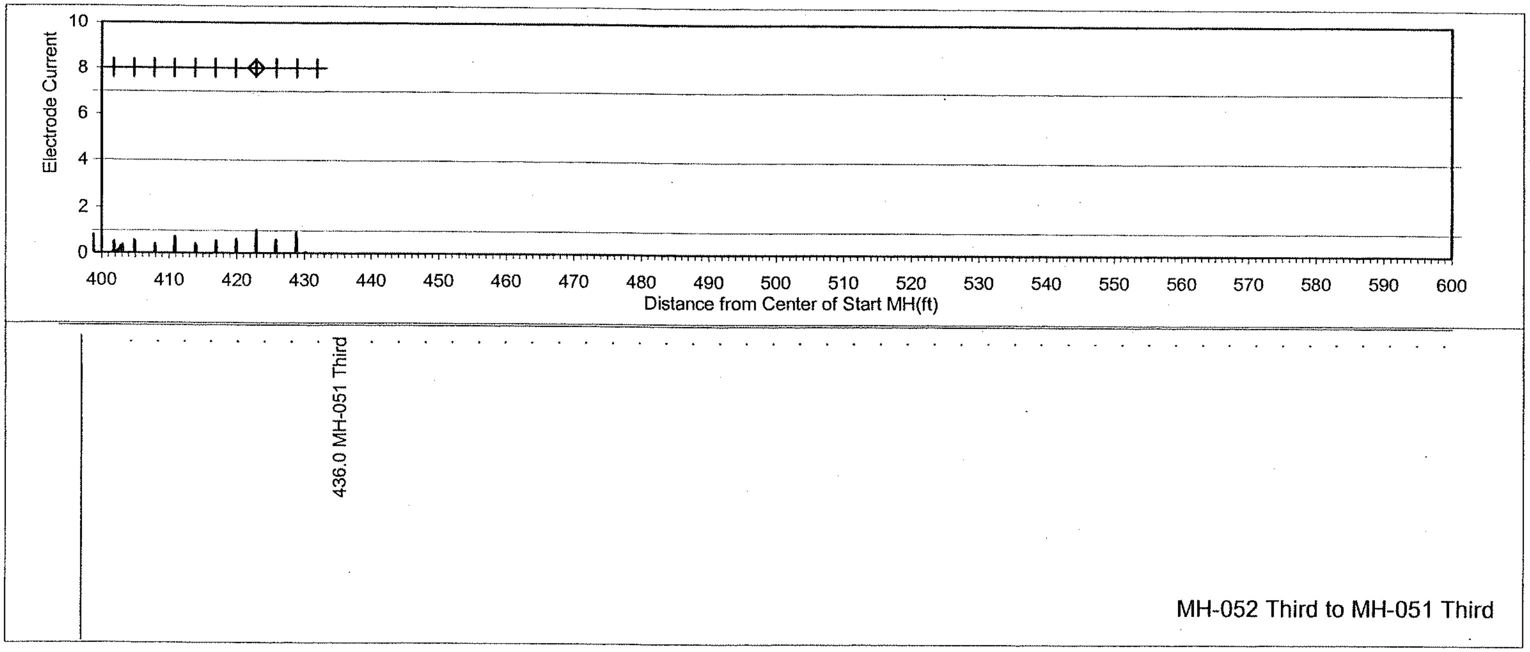

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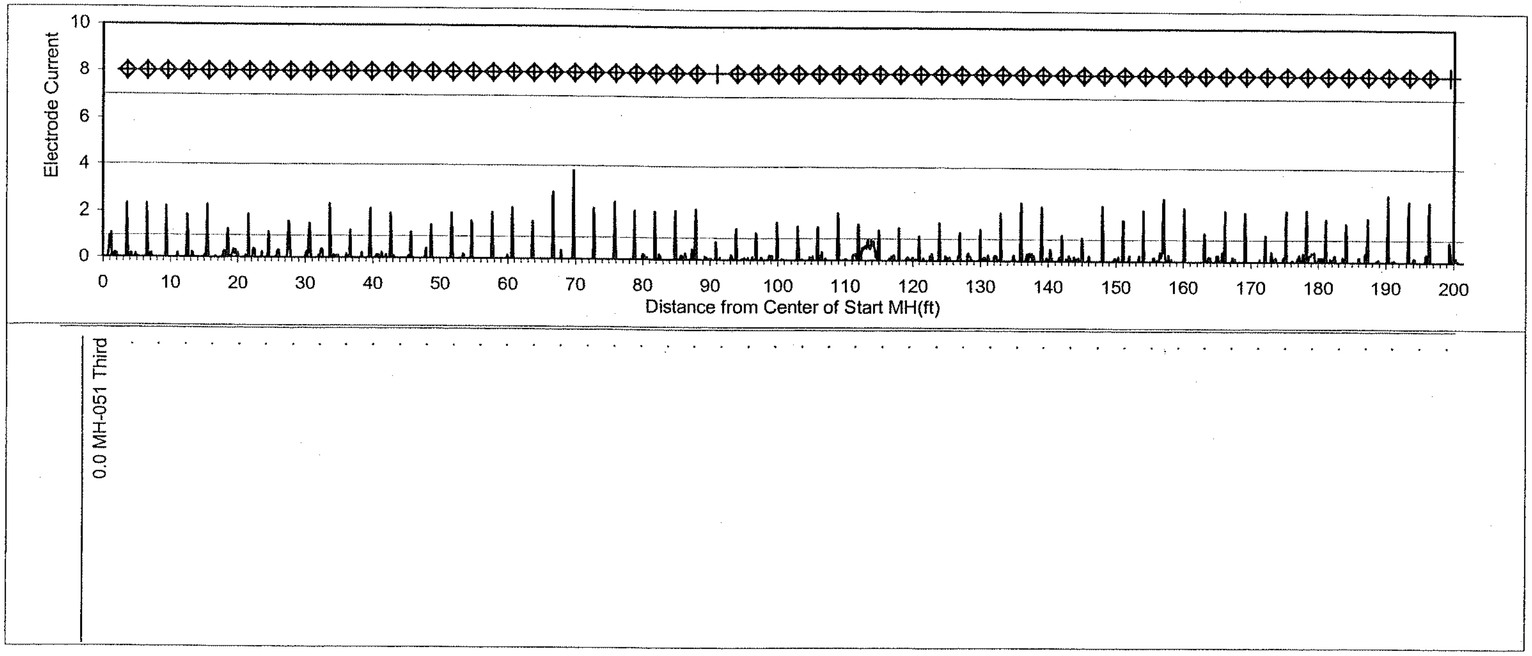

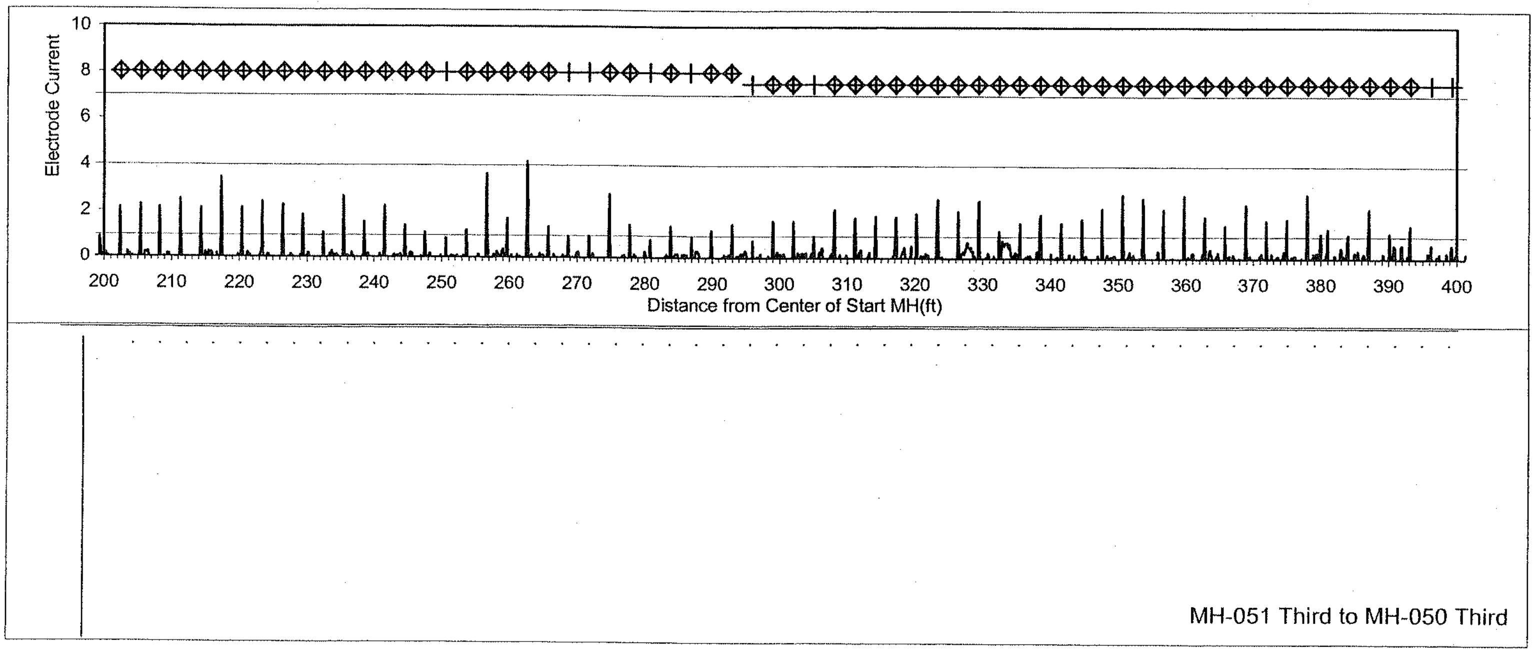

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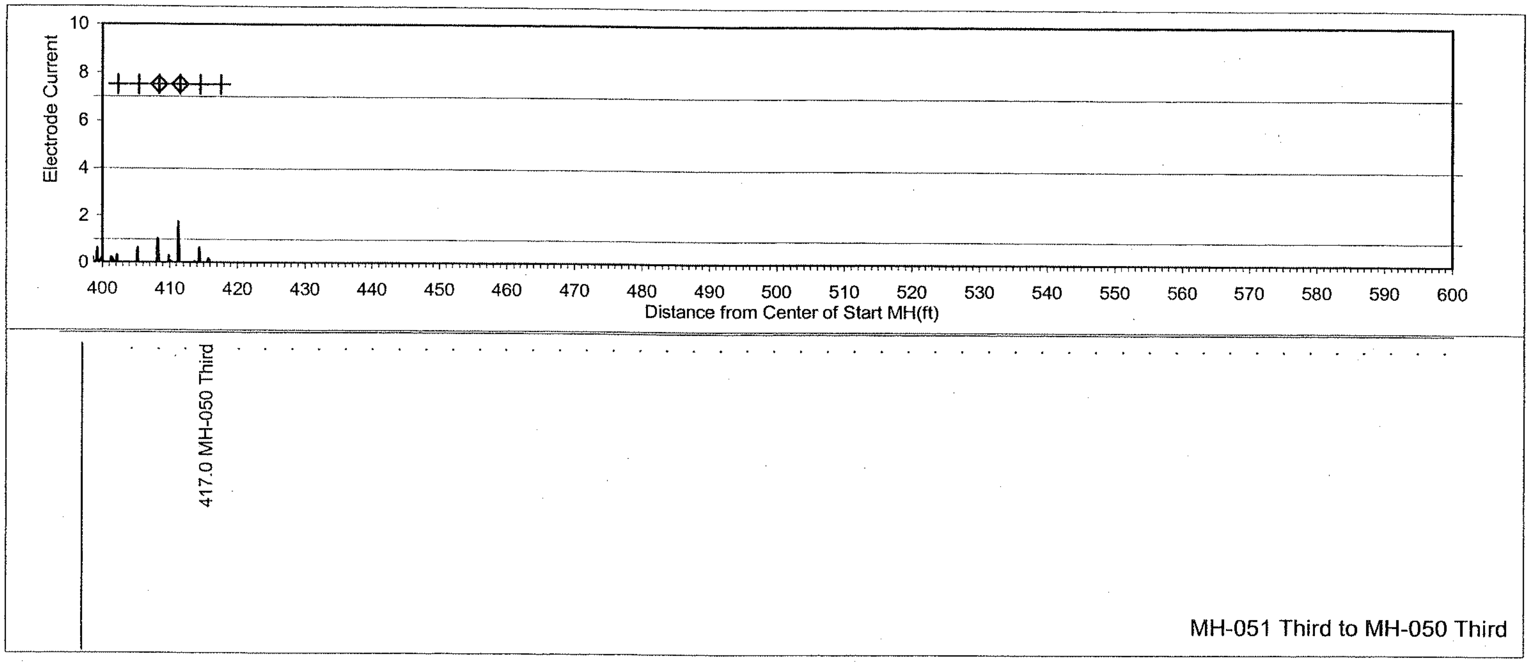

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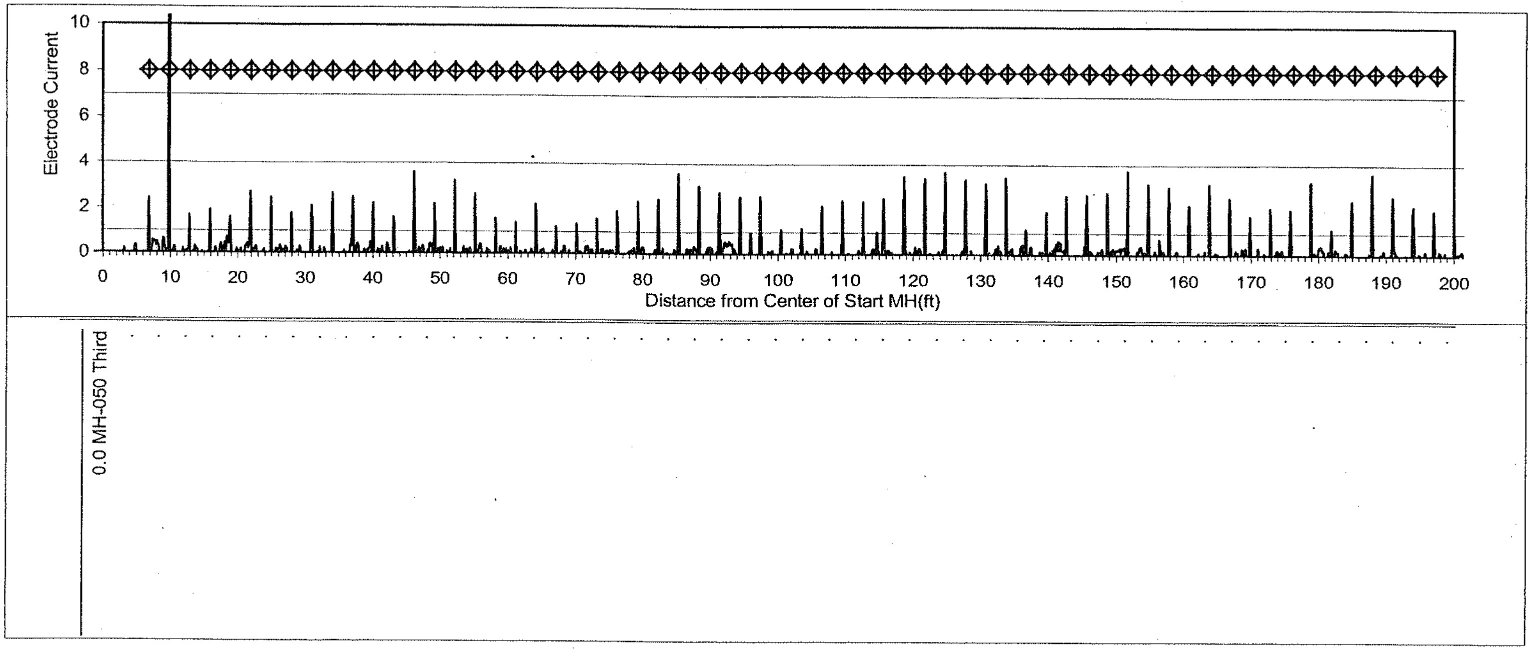

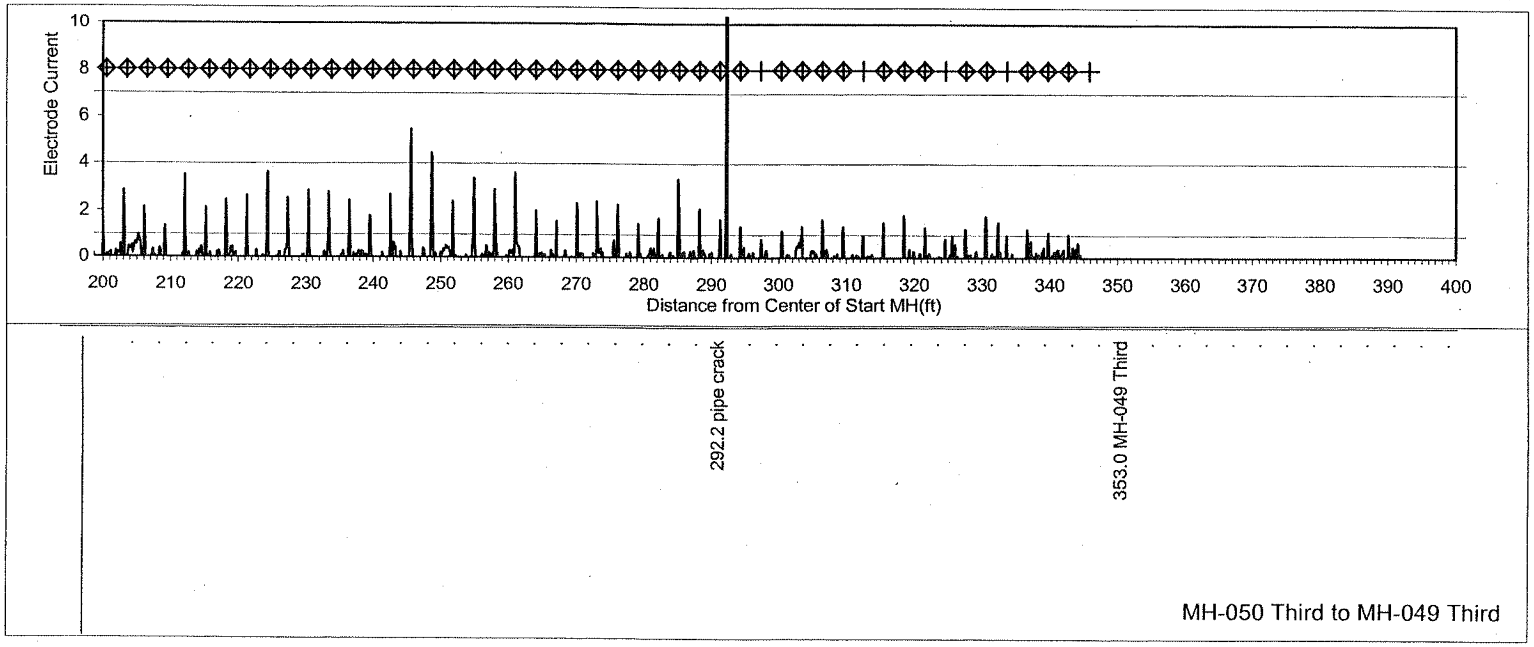

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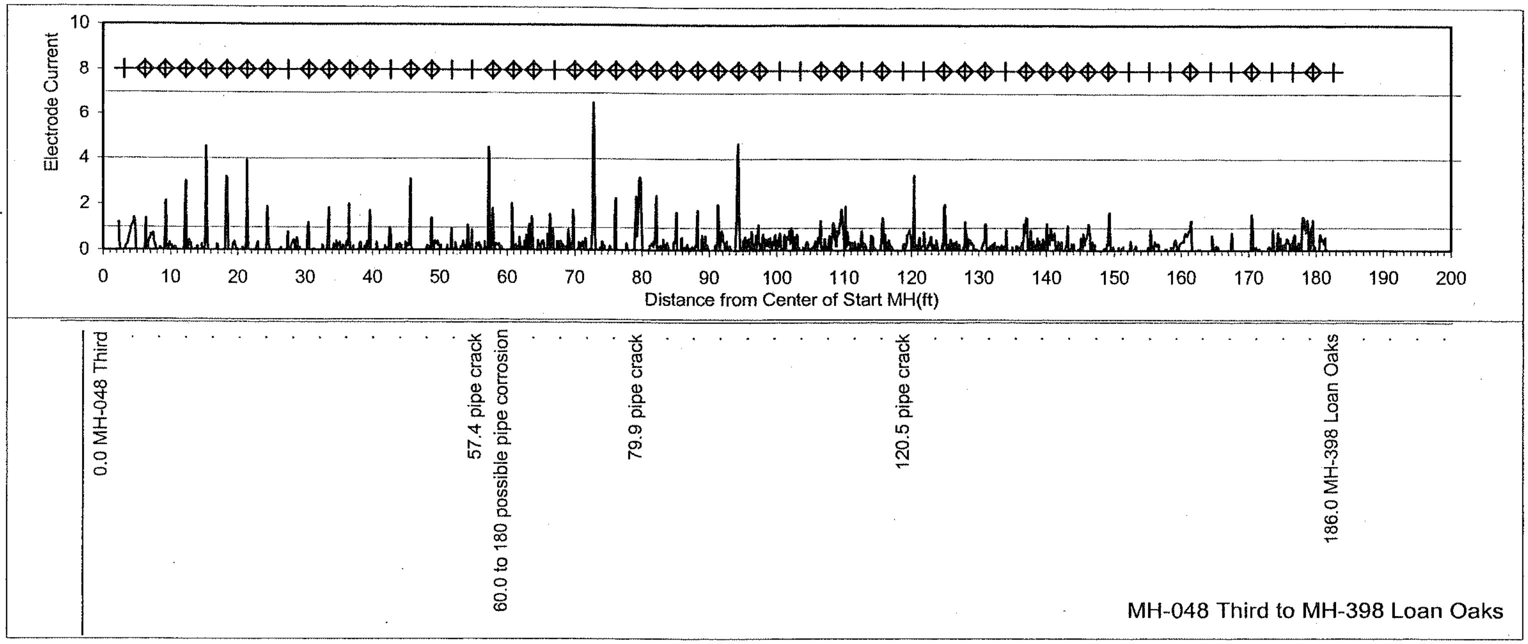

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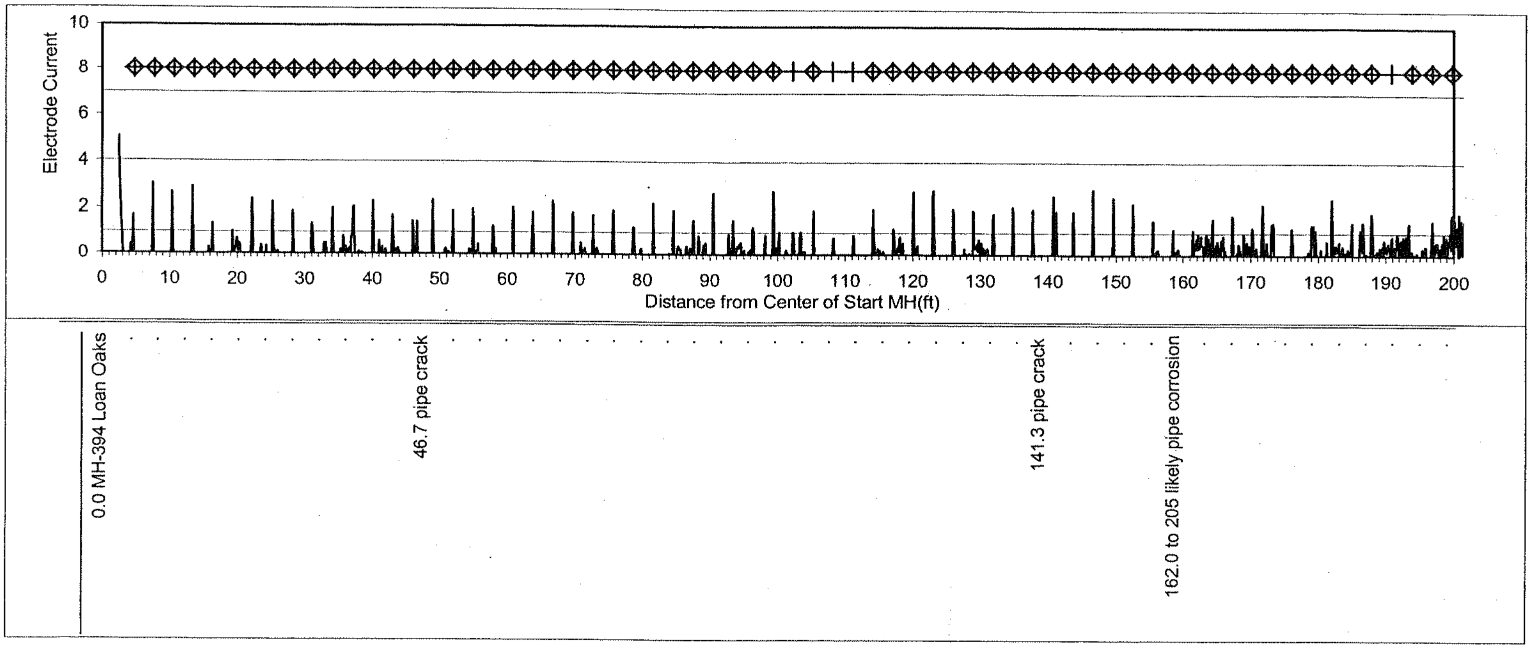

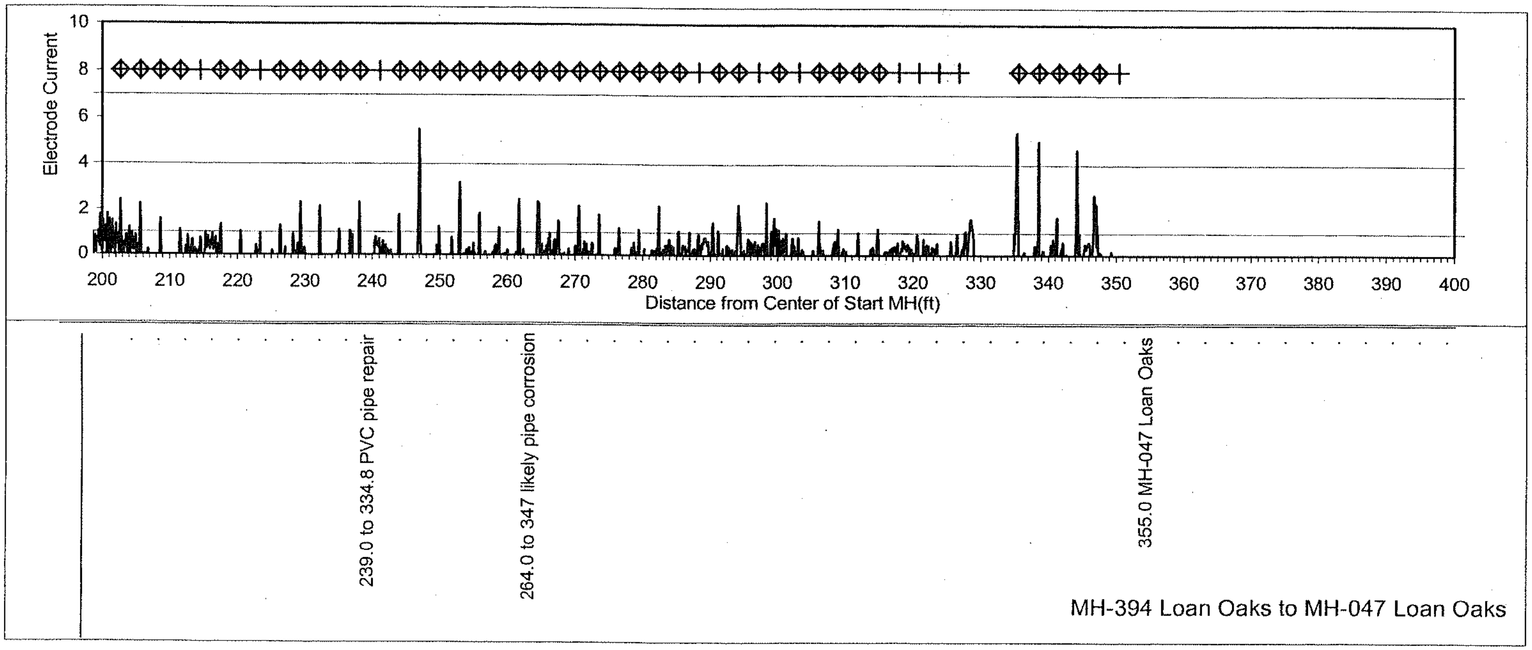

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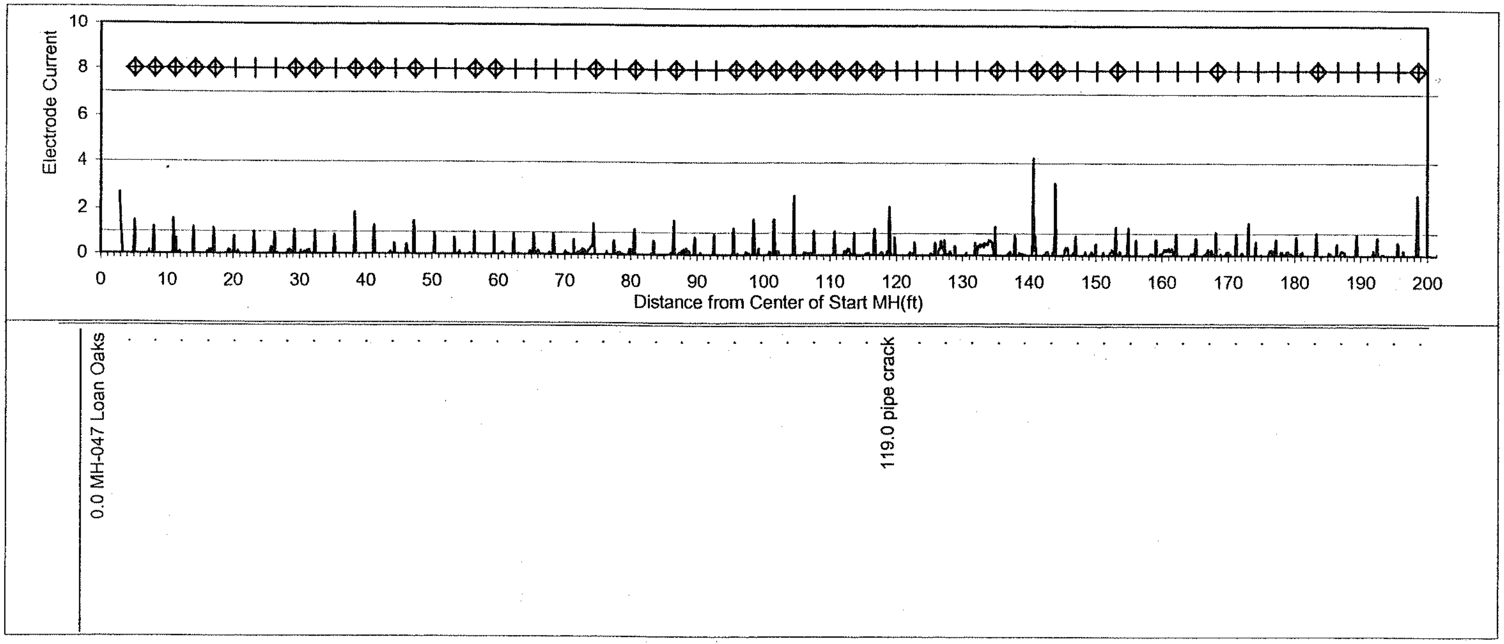

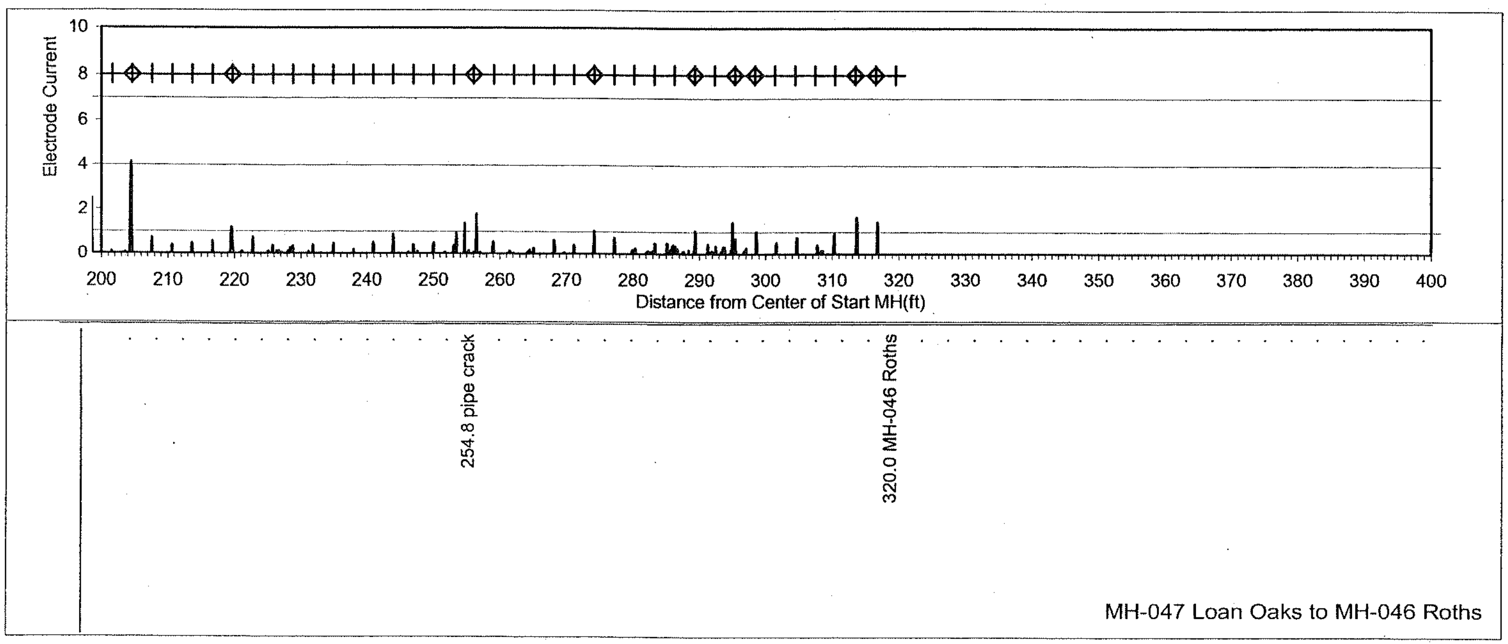

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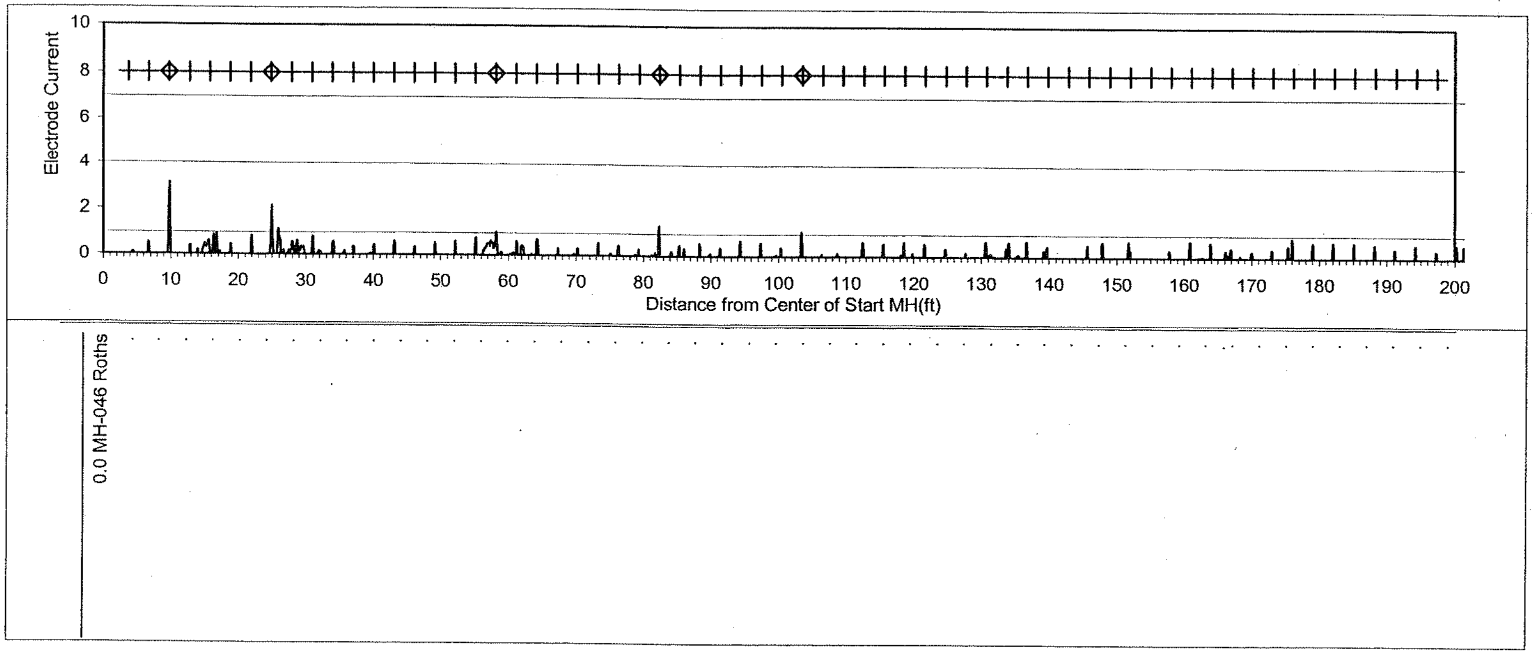

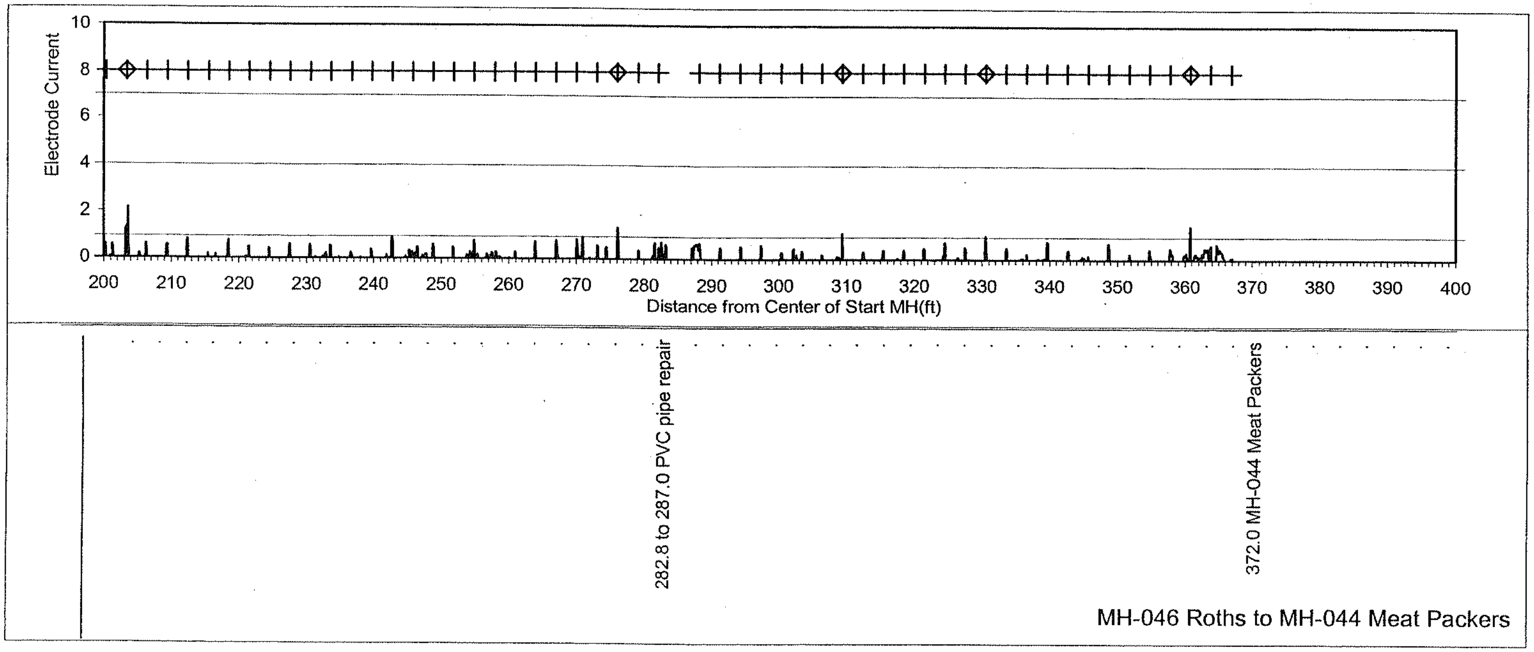

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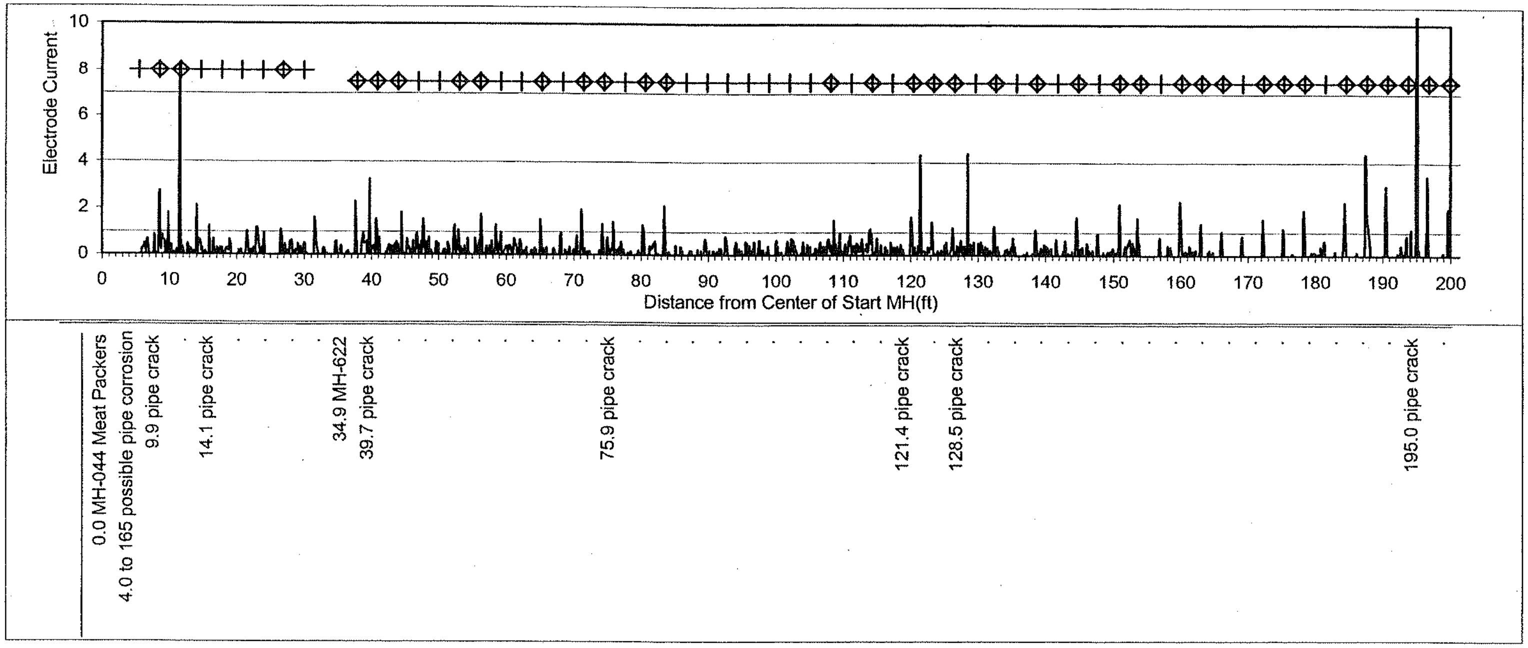

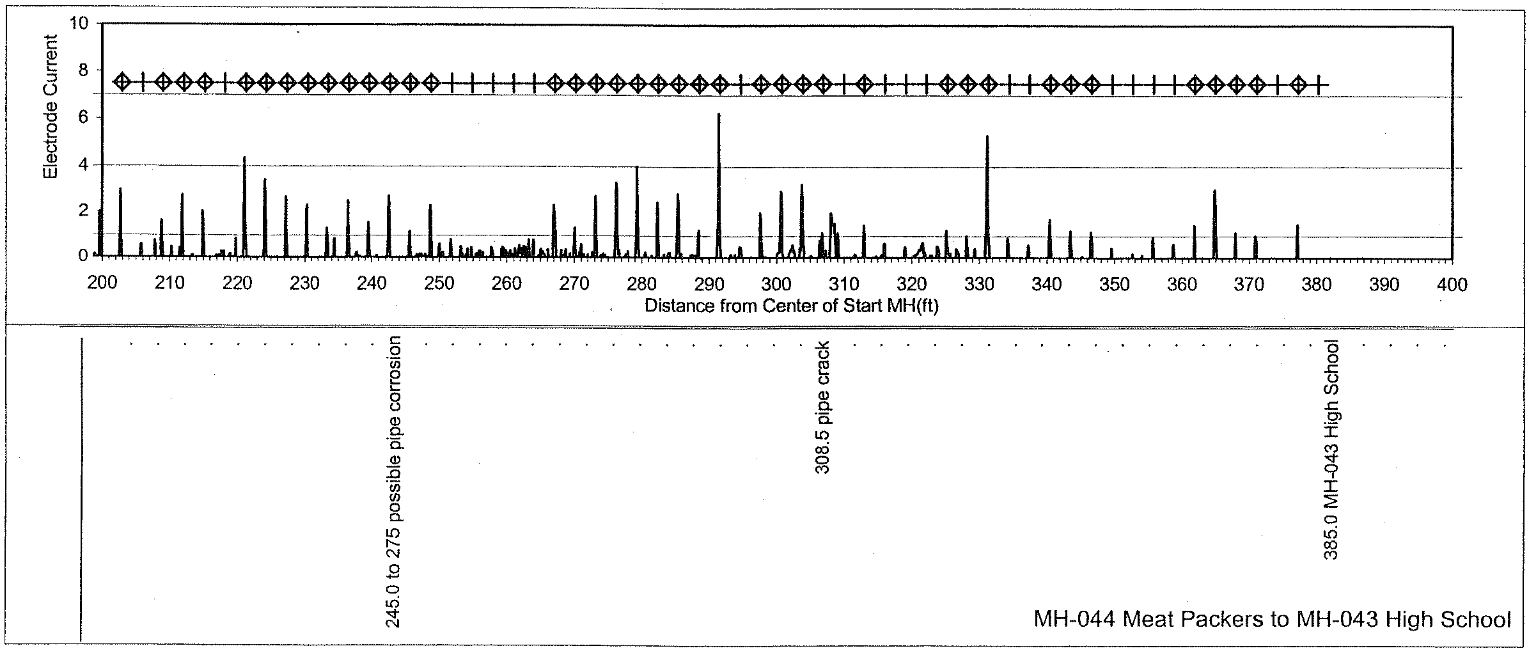

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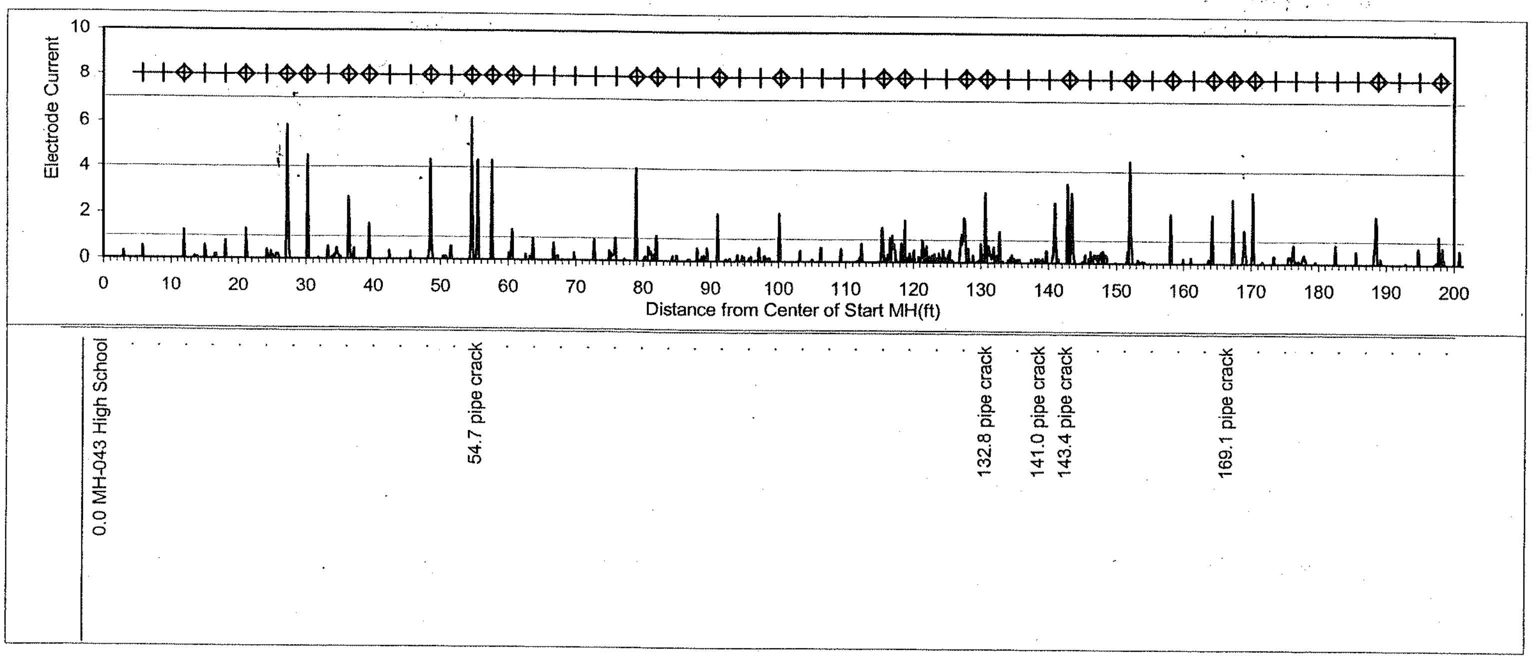

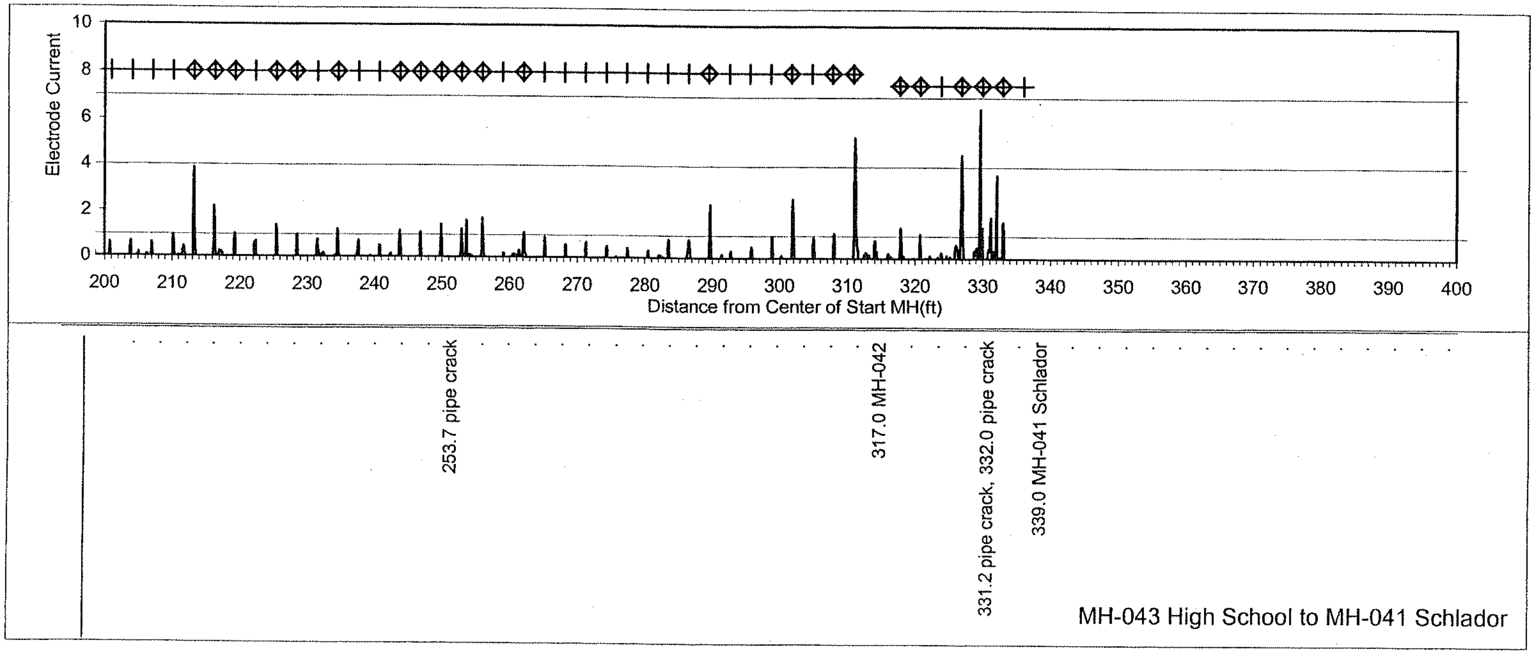

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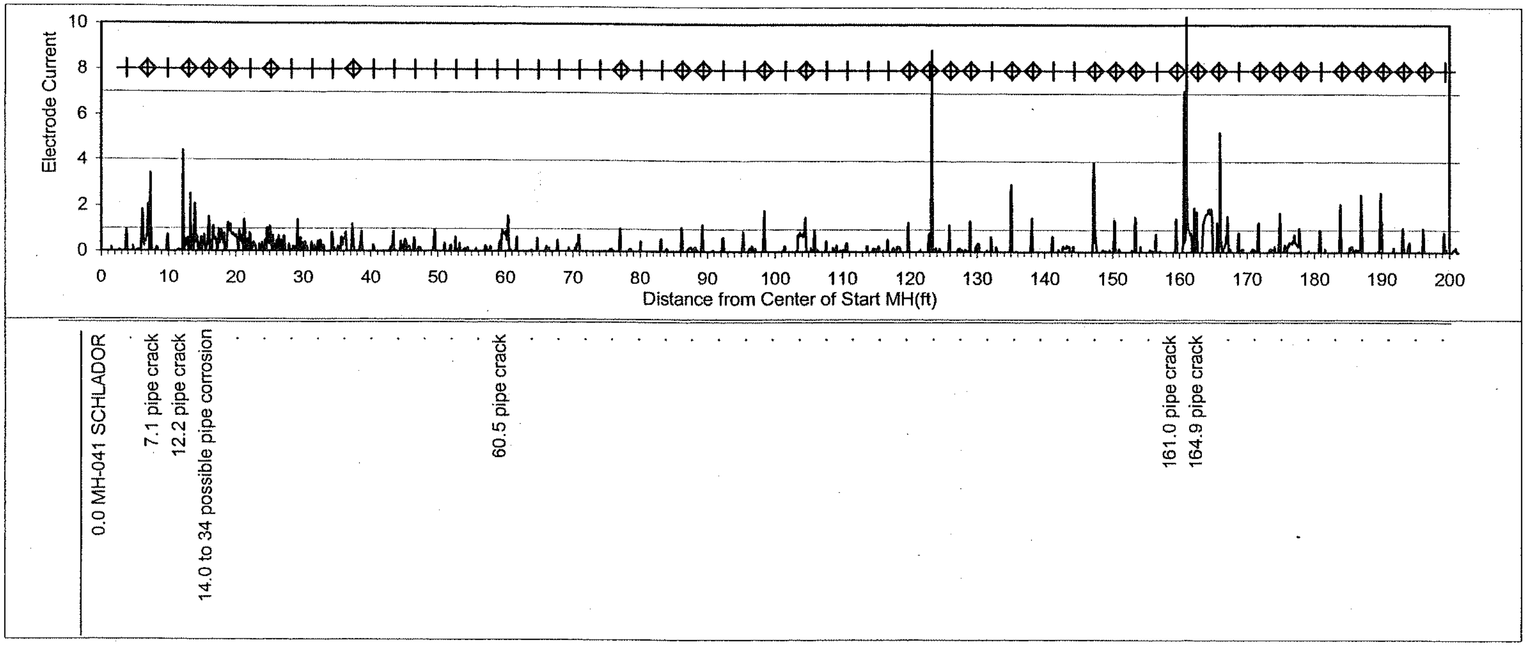

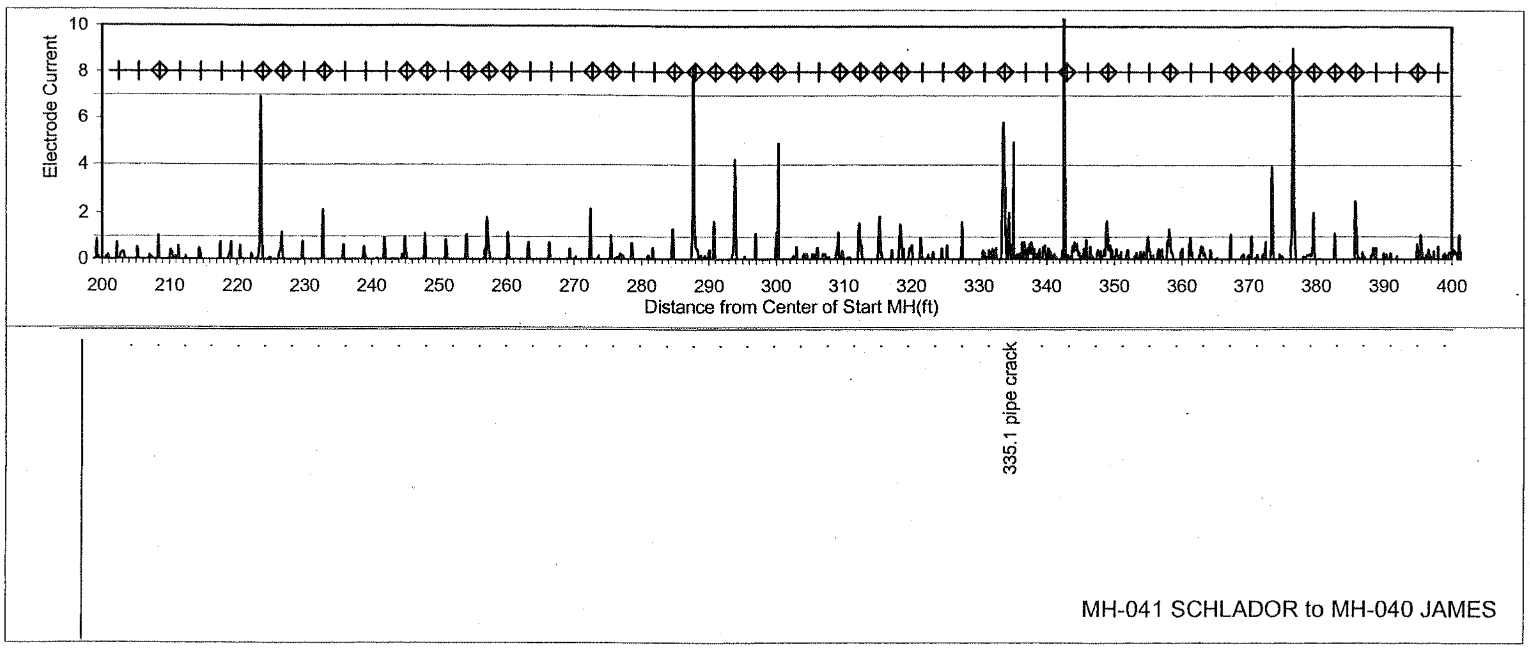

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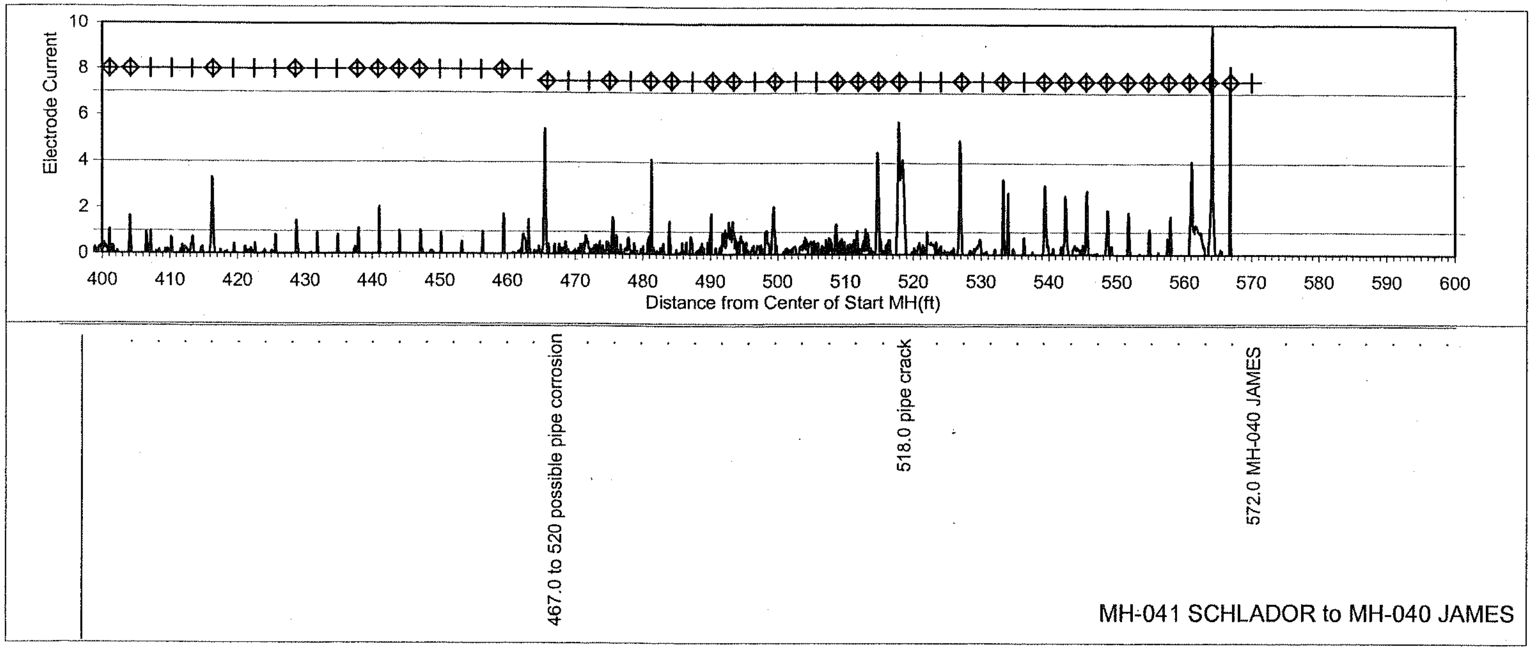

+ Joint Marker ◇ Anomaly at Joint

City of Silverton

Wastewater System Facility Master Plan

Appendix E

Cost Calculations for Pipe: **IMP-1 Westfield St**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2006
Length: 910 ft
Conduit Type: Gravity Sewer
Depth of Cover: 5 ft
Trench Backfill Type: Native
Disposal Type: No Disposal Cost
Manhole Spacing: None
Existing Utilities: Average
Dewatering: None
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	6.88 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 2005)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	843	CY	12.00	10,100
Backfill	490	CY	8.00	3,920
Complete Pavement Restoration	570	SY	55.00	31,400
Trench Safety	12,513	SF	0.50	6,260
Spoil Load and Haul	352	CY	12.00	4,230
Pipe Unit Material Cost	910	lf	8.00	7,280
Pipe Installation	910	lf	18.00	16,400
Place Pipe Zone Fill	332	CY	32.00	10,600
Existing Utilities	910	lf	25.00	22,800
Traffic Control	910	lf	8.00	7,280
			Year 2005 subtotal	120,000

Mobilization/Demobilization at 10%	1.10
Multiplier from ENRCCI 8390 (2005) to 8655 (2006)	1.03
Effective Multiplier	1.13
Subtotal	136,000

Total: $136,000

Cost Calculations for Pipe: **IMP-2 Oregon Gardens Force Main**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2006
Length: 909 ft
Conduit Type: Force Main
Depth of Cover: 5 ft
Trench Backfill Type: Native
Disposal Type: No Disposal Cost
Manhole Spacing: None
Existing Utilities: Average
Dewatering: None
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in. Actual Pipe 6"
(not available in Tabula database)

Geometry

Outer Diameter	0.754 ft
Trench Width	3.48 ft
Excavation Depth	6.75 ft
Complete Surface Rest. Width	5.48 ft

Unit Costs (Basis 2005)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	791	CY	12.00	9,500
Backfill	469	CY	8.00	3,750
Complete Pavement Restoration	554	SY	55.00	30,400
Trench Safety	12,279	SF	0.50	6,140
Spoil Load and Haul	323	CY	12.00	3,870
Pipe Unit Material Cost	909	lf	18.00	16,400
Pipe Installation	909	lf	18.00	16,400
Place Pipe Zone Fill	308	CY	32.00	9,850
Existing Utilities	909	lf	25.00	22,700
Traffic Control	909	lf	8.00	7,270
			Year 2005 subtotal	126,000

Mobilization/Demobilization at 10%	1.10
Multiplier from ENRCCI 8390 (2005) to 8655 (2006)	1.03
Effective Multiplier	1.13
Subtotal	143,000

Total: $143,000 — Adjust to 6" based on cost per diameter inch

$$\frac{\$143{,}000}{8'' \times 909\text{ft}} = \$19.66/\text{in-Lf}$$

$$\frac{\$19.66}{\text{in-Lf}} \times 6'' \times 909\text{ ft} = \$107{,}250 \approx \boxed{\$108{,}000}$$

Cost Calculations for Pipe: **IMP-3 James St**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2006
Length: 576 ft
Conduit Type: Gravity Sewer
Depth of Cover: 5 ft
Trench Backfill Type: Native
Disposal Type: No Disposal Cost
Manhole Spacing: None
Existing Utilities: Average
Dewatering: None
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 18 in.

Geometry

Outer Diameter	1.92 ft
Trench Width	4.99 ft
Excavation Depth	7.92 ft
Complete Surface Rest. Width	6.99 ft

Unit Costs (Basis 2005)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	843	CY	12.00	10,100
Backfill	426	CY	8.00	3,410
Complete Pavement Restoration	447	SY	55.00	24,600
Trench Safety	9,120	SF	0.50	4,560
Spoil Load and Haul	417	CY	12.00	5,010
Pipe Unit Material Cost	576	lf	22.00	12,700
Pipe Installation	576	lf	27.00	15,600
Place Pipe Zone Fill	356	CY	32.00	11,400
Existing Utilities	576	lf	35.00	20,200
Traffic Control	576	lf	8.00	4,610
			Year 2005 subtotal	112,000

Mobilization/Demobilization at 10%	1.10
Multiplier from ENRCCI 8390 (2005) to 8655 (2006)	1.03
Effective Multiplier	1.13
Subtotal	127,000

Total: $127,000

Cost Calculations for Pipe: **IMP-4 Sherman St**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2006
Length: 175 ft
Conduit Type: Gravity Sewer
Depth of Cover: 8 ft
Trench Backfill Type: Native
Disposal Type: No Disposal Cost
Manhole Spacing: None
Existing Utilities: Average
Dewatering: None
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 18 in.

Geometry

Outer Diameter	1.92 ft
Trench Width	4.99 ft
Excavation Depth	10.9 ft
Complete Surface Rest. Width	6.99 ft

Unit Costs (Basis 2005)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	353	CY	12.00	4,240
Backfill	226	CY	8.00	1,810
Complete Pavement Restoration	136	SY	55.00	7,480
Trench Safety	3,821	SF	0.50	1,910
Spoil Load and Haul	127	CY	12.00	1,520
Pipe Unit Material Cost	175	lf	22.00	3,850
Pipe Installation	175	lf	27.00	4,730
Place Pipe Zone Fill	108	CY	32.00	3,460
Existing Utilities	175	lf	35.00	6,130
Traffic Control	175	lf	8.00	1,400
			Year 2005 subtotal	36,500

Mobilization/Demobilization at 10%	1.10
Multiplier from ENRCCI 8390 (2005) to 8655 (2006)	1.03
Effective Multiplier	1.13
Subtotal	41,400

Total: $41,400

Cost Calculations for Pipe: **IMP-5 Adams St**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2006
Length: 850 ft
Conduit Type: Gravity Sewer
Depth of Cover: 8 ft
Trench Backfill Type: Native
Disposal Type: No Disposal Cost
Manhole Spacing: None
Existing Utilities: Average
Dewatering: None
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 12 in.

Geometry

Outer Diameter	1.42 ft
Trench Width	4.34 ft
Excavation Depth	10.4 ft
Complete Surface Rest. Width	6.34 ft

Unit Costs (Basis 2005)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,424	CY	12.00	17,100
Backfill	957	CY	8.00	7,650
Complete Pavement Restoration	599	SY	55.00	32,900
Trench Safety	17,708	SF	0.50	8,850
Spoil Load and Haul	467	CY	12.00	5,600
Pipe Unit Material Cost	850	lf	16.00	13,600
Pipe Installation	850	lf	24.00	20,400
Place Pipe Zone Fill	417	CY	32.00	13,400
Existing Utilities	850	lf	25.00	21,300
Traffic Control	850	lf	8.00	6,800
			Year 2005 subtotal	148,000

Mobilization/Demobilization at 10%	1.10
Multiplier from ENRCCI 8390 (2005) to 8655 (2006)	1.03
Effective Multiplier	1.13
Subtotal	167,000

Total: $167,000

Cost Calculations for Project: **Silverton Capacity Improvements**

Project year: 2006

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Project Year: 2006
Comments: Silverton Improvements

Sub Items

Name	Type	Year	Cost	Multiplier	2006 Cost
IMP-1 Westfield St	Pipe	2006	136,000	1.00	136,000
IMP-2 Oregon Gardens Force Main	Pipe	2006	143,000	1.00	143,000
IMP-3 James St	Pipe	2006	127,000	1.00	127,000
IMP-4 Sherman St	Pipe	2006	41,400	1.00	41,400
IMP-5 Adams St	Pipe	2006	167,000	1.00	167,000
				Subtotal	616,000

Total: $616,000

City of Silverton

Wastewater System Facility Master Plan

Appendix F

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
102	101	MAP12-SH	UNKNOWN	N 1ST ST	UNKNOWN	CLAY	330.0
310	98	S-316	NOV1977	MILL ST	8	CLAY	680.0
546	545	MAP23-SH	UNKNOWN	CENTRAL ST	8	CLAY	520.0
220	98	S-084	AUG1940	S 3RD ST	8	CLAY	500.0
544	542	MAP12-SH	UNKNOWN	S JAMES AVE	8	CLAY	405.0
545	524	MAP12-SH	UNKNOWN	S JAMES AVE	8	CLAY	385.0
180	179	MAP07-SH	UNKNOWN	JEFFERSON ST	8	CLAY	375.0
162	161	S-245	OCT1973	ORCHARD ST	8	CLAY	370.0
173	172	MAP18-SH	UNKNOWN	LEWIS ST	8	CLAY	300.0
174	173	MAP18-SH	UNKNOWN	LEWIS ST	8	CLAY	285.0
172	22	MAP07-SH	UNKNOWN	WASHINGTON ST	8	CLAY	280.0
171	170	MAP18-SH	UNKNOWN	E MAIN ST	8	CLAY	270.0
543	542	MAP12-SH	UNKNOWN	BROOK ST	8	CLAY	105.0
600	548	MAP06-SH	UNKNOWN	FLORIDA DR	6	CLAY	350.0
319	AlderPS	S-060	SEP1922	SILVER ST	6	CLAY	325.0
97	96	MAP17-SH	UNKNOWN	WELCH ST	6	CLAY	240.0
98	274	MAP17-SH	UNKNOWN	WELCH ST	6	CLAY	180.0
602	558	MAP13-SH	UNKNOWN	PARK ST	6	CLAY	155.0
318	319	S-060	SEP1922	SILVER ST	6	CLAY	20.0
318A	318	S-060	SEP1922	SILVER ST	6	CLAY	0.0
						Mat. Total	**6075**
100	99	MAP22-SH	JAN1998	OREGON GARDENS	UNKNOWN	UNKNOWN	170.0
101	100	MAP22-SH	JAN1998	OREGON GARDENS	UNKNOWN	UNKNOWN	140.0
104	103	P-535	JAN1995	LONE OAKS LP	21	UNKNOWN	121.0
110	108	S-084	AUG1940	PINE ST	18	UNKNOWN	457.7
1136	1135	S-084	AUG1940	PINE ST	18	UNKNOWN	457.7
112	111	S-084	AUG1940	N JAMES AVE	18	UNKNOWN	336.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
107	36	S-084	AUG1940	SHERMAN ST	18	UNKNOWN	210.0
113	112	P-535	JAN1995	LONE OAKS LP	18	UNKNOWN	206.0
108	107	S-084	AUG1940	PINE ST	18	UNKNOWN	178.0
106	105	S-084	AUG1940	SHERMAN ST	18	UNKNOWN	165.0
105	104	S-084	AUG1940	SHERMAN ST	18	UNKNOWN	144.5
1135	1134	S-084	AUG1940	N JAMES AVE	18	UNKNOWN	104.0
115	114	S-351E	MAY1983	TODD CT	15	UNKNOWN	369.5
124	38	S-428	AUG1988	ESKA WAY	15	UNKNOWN	352.5
118	117	S-351E	MAY1983	N CHURCH ST	15	UNKNOWN	347.0
123	122	S-428	AUG1988	ESKA WAY	15	UNKNOWN	339.6
119	118	S-351E	MAY1983	N CHURCH ST	15	UNKNOWN	334.7
117	FloridaPS	S-351E	MAY1983	MILL ST	15	UNKNOWN	268.1
122	37	S-351E	MAY1983	ESKA WAY	15	UNKNOWN	267.2
116	60	S-351E	MAY1983	MILL ST	15	UNKNOWN	266.5
120	09	S-351E	MAY1983	N CHURCH ST	15	UNKNOWN	264.4
121	120	S-351E	MAY1983	N CHURCH ST	15	UNKNOWN	15.9
129	124	S-084	AUG1940	BROOK ST	12	UNKNOWN	676.0
130	273	S-084	AUG1940	S JAMES AVE	12	UNKNOWN	575.5
131	82	S-084	AUG1940	McCLAINE ST	12	UNKNOWN	550.0
135	83	S-084	AUG1940	S WATER ST	12	UNKNOWN	368.7
137	136	S-522	APR1981	W MAIN ST	12	UNKNOWN	295.5
128	126	S-084	AUG1940	ALDER ST	12	UNKNOWN	266.0
132	131	S-084	AUG1940	WESTFIELD ST	12	UNKNOWN	174.5
126	125	S-084	AUG1940	MAPLE ST	12	UNKNOWN	167.0
138	137	S-522	APR1981	W MAIN ST	12	UNKNOWN	145.2
134	133	S-084	AUG1940	S WATER ST	12	UNKNOWN	100.0
136	135	S-522	APR1981	W MAIN ST	12	UNKNOWN	44.0
125	124	S-084	AUG1940	SHERMAN ST	12	UNKNOWN	0.0
140	139	S-084	AUG1940	N 1ST ST	10	UNKNOWN	375.0
139	81	S-084	AUG1940	N 1ST ST	10	UNKNOWN	310.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
141	140	S-084	AUG1940	N 1ST ST	10	UNKNOWN	220.0
142	148	S-084	AUG1940	N 1ST ST	10	UNKNOWN	220.0
550	525	S-084	AUG1940	MADISON ST	8	UNKNOWN	643.0
154	153	S-084	AUG1940	GRANT ST	8	UNKNOWN	635.0
241	238	S-084	AUG1940	NORWAY AVE	8	UNKNOWN	585.0
549	600	S-084	AUG1940	MADISON ST	8	UNKNOWN	552.0
236	235B	S-084	AUG1940	BARTLETT ST	8	UNKNOWN	522.0
35	35A	S-428	AUG1988	OAK ST	8	UNKNOWN	501.5
164	163	S-084	AUG1940	PINE ST	8	UNKNOWN	487.0
507	506	MAP12-SH	UNKNOWN	ALDER ST	8	UNKNOWN	485.0
148	81	S-084	AUG1940	S CHURCH ST	8	UNKNOWN	483.0
402	401	S-351E	MAY1983	N CHURCH ST	8	UNKNOWN	468.2
39	38	S-428	AUG1988	OAK ST	8	UNKNOWN	467.5
577	568	S-428	AUG1988	OAK ST	8	UNKNOWN	467.5
38	37	S-428	AUG1988	OAK ST	8	UNKNOWN	460.6
526	550	S-195D	FEB1967	WEIBY AVE	8	UNKNOWN	451.0
559	547	P-487	NOV1991	SWEDEN CIR	8	UNKNOWN	445.0
501	538	S-183	JUN1964	EUREKA AVE	8	UNKNOWN	437.1
534	533	S-195C	DEC1966	ROSS AVE	8	UNKNOWN	436.7
576	575	S-351E	MAY1983	BARTLETT ST	8	UNKNOWN	425.5
155	95	P-521	UNKNOWN	EASEMENT	8	UNKNOWN	415.0
502	501	S-183	JUN1964	EUREKA AVE	8	UNKNOWN	407.6
400	399	S-428	AUG1988	N STEELHAMMER RD	8	UNKNOWN	407.2
215	214	S-084	AUG1940	S 1ST ST	8	UNKNOWN	400.0
218	217	S-084	AUG1940	S 3RD ST	8	UNKNOWN	400.0
283	10	S-084	AUG1940	WELL ST	8	UNKNOWN	400.0
415	414	P-250	FEB1975	W MAIN ST	8	UNKNOWN	398.0
580	579	MAP13-SH	UNKNOWN	N 2ND ST	8	UNKNOWN	395.0
407	406	S-428	AUG1988	OAK ST	8	UNKNOWN	394.2
40	39	S-428	AUG1988	OAK ST	8	UNKNOWN	392.5

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
401	400	S-428	AUG1988	N STEELHAMMER RD	8	UNKNOWN	390.0
532	531	S-195C	DEC1966	ASH ST	8	UNKNOWN	388.0
144	143	MAP13-SH	UNKNOWN	OAK ST	8	UNKNOWN	385.0
228	101	S-082	SEP1964	BARGER ST	8	UNKNOWN	383.0
523	522	S-195D	FEB1967	EDGEWOOD DR	8	UNKNOWN	381.0
315	274	MAP13-SH	UNKNOWN	N 3RD ST	8	UNKNOWN	380.0
560	561	P-487	NOV1991	SWEDEN CIR	8	UNKNOWN	375.0
409	408	S-428	AUG1988	WALL ST	8	UNKNOWN	360.2
413	412	P-250	FEB1975	WESTFIELD ST	8	UNKNOWN	351.0
147	142	S-084	AUG1940	S CHURCH ST	8	UNKNOWN	350.0
551	625	S-084	AUG1940	SMITH ST	8	UNKNOWN	350.0
145	144	MAP13-SH	UNKNOWN	OAK ST	8	UNKNOWN	345.0
27A	27	S-286	NOV1976	N 2ND ST	8	UNKNOWN	342.9
235B	235A	S-084	AUG1940	N CHURCH ST	8	UNKNOWN	336.3
237	236	S-084	AUG1940	NORWAY AVE	8	UNKNOWN	331.7
534	535	S-195C	DEC1966	HAZEL ST	8	UNKNOWN	331.0
313	270	S-286	NOV1976	N 2ND ST	8	UNKNOWN	327.5
542	523	S-181	APR1964	JEROME AVE	8	UNKNOWN	327.1
224	223	S-084	AUG1940	BARGER ST	8	UNKNOWN	326.0
168	167	S-084	AUG1940	BROWN ST	8	UNKNOWN	325.0
287	180	S-084	AUG1940	BROWN ST	8	UNKNOWN	325.0
412	411	P-250	FEB1975	WESTFIELD ST	8	UNKNOWN	325.0
556	555	S-084	AUG1940	S WATER ST	8	UNKNOWN	324.0
557	556	S-084	AUG1940	S WATER ST	8	UNKNOWN	324.0
525	524	S-195D	FEB1967	EDGEWOOD DR	8	UNKNOWN	323.0
37	36	S-428	AUG1988	OAK ST	8	UNKNOWN	320.0
513	512	S-181	APR1964	JEROME AVE	8	UNKNOWN	311.8
177	176	MAP07-SH	UKNOWN	CHESTER ST	8	UNKNOWN	310.0
406	405	S-351E	MAY1983	LIBERTY ST	8	UNKNOWN	307.7
208	401	MAP13-SH	UNKNOWN	OAK ST	8	UNKNOWN	305.0

City of Silverton
Wastewater Facility Masterplan
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Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
234	53	P-220A	AUG1976	MADISON ST	8	UNKNOWN	300.0
235A	235	S-084	AUG1940	N CHURCH ST	8	UNKNOWN	300.0
36	35	S-428	AUG1988	OAK ST	8	UNKNOWN	300.0
414	413	P-250	FEB1975	W MAIN ST	8	UNKNOWN	300.0
167	166	S-084	AUG1940	N JAMES AVE	8	UNKNOWN	298.0
232	231	P-220A	AUG1976	MADISON ST	8	UNKNOWN	293.0
233	52	P-220A	AUG1976	MADISON ST	8	UNKNOWN	290.0
552	551	S-084	AUG1940	S WATER ST	8	UNKNOWN	289.0
527	626	S-195E	FEB1967	ANDERSON DR	8	UNKNOWN	287.0
454	455	S-181	APR1964	W MAIN ST	8	UNKNOWN	281.0
156	95	S-084	AUG1940	N JAMES AVE	8	UNKNOWN	280.0
244	243	MAP13-SH	UNKNOWN	MILL ST	8	UNKNOWN	280.0
195	194	S-082	JUN1949	C ST	8	UNKNOWN	276.0
403	402	S-351E	MAY1983	NORWAY AVE	8	UNKNOWN	273.5
161	18	S-245	OCT1973	ORCHARD ST	8	UNKNOWN	272.0
404	403	S-351E	MAY1983	NORWAY AVE	8	UNKNOWN	268.2
399	396	S-428	AUG1988	OAK ST	8	UNKNOWN	263.6
272	39	S-084	AUG1940	E MAIN ST	8	UNKNOWN	263.0
160	159	S-082	JUN1965	FLORIDA DR	8	UNKNOWN	260.0
520	519	S-195C	DEC1966	S CENTER ST	8	UNKNOWN	258.0
265	263	S-084	AUG1940	5TH ST	8	UNKNOWN	244.5
511	510A	S-181	APR1964	W MAIN ST	8	UNKNOWN	242.2
539	519	S-195C	DEC1966	KEENE AVE	8	UNKNOWN	242.0
192	63	S-084	AUG1940	MAPLE ST	8	UNKNOWN	241.0
405	404	S-351E	MAY1983	NORWAY AVE	8	UNKNOWN	235.3
143	168	MAP13-SH	UNKNOWN	MILL ST	8	UNKNOWN	230.0
182	175	MAP07-SH	UNKNOWN	JEFFERSON ST	8	UNKNOWN	230.0
214	250	MAP18-SH	UNKNOWN	JERSEY ST	8	UNKNOWN	230.0
301	597A	MAP18-SH	UNKNOWN	JAY ST	8	UNKNOWN	230.0
243	236	S-084	AUG1940	OAK ST	8	UNKNOWN	225.0

City of Silverton
Wastewater Facility Masterplan
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US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
288	96	MAP13-SH	UNKNOWN	B ST	8	UNKNOWN	225.0
529	528	S-195D	FEB1967	KEENE AVE	8	UNKNOWN	225.0
269	85	S-084	AUG1940	5TH ST	8	UNKNOWN	221.0
152	153	S-084	AUG1940	PINE ST	8	UNKNOWN	220.0
530	529	S-195D	FEB1967	KEENE AVE	8	UNKNOWN	219.0
225	224	S-084	AUG1940	BARGER ST	8	UNKNOWN	218.0
554	553	P-487	NOV1991	NORWAY AVE	8	UNKNOWN	210.0
547	525	S-083	DEC1966	S 1ST ST	8	UNKNOWN	206.0
398	399	S-428	AUG1988	OAK ST	8	UNKNOWN	202.1
518	517	S-195J	DEC1966	S CENTER ST	8	UNKNOWN	200.5
159	97	S-082	JUN1965	FLORIDA DR	8	UNKNOWN	200.0
190	64	MAP06-SH	UNKNOWN	N 2ND ST	8	UNKNOWN	200.0
221	310	S-084	AUG1940	COWING ST	8	UNKNOWN	200.0
226	225	S-084	AUG1940	CENTRAL ST	8	UNKNOWN	200.0
229	228	S-084	AUG1940	CENTRAL ST	8	UNKNOWN	200.0
35A	39	S-428	AUG1988	OAK ST	8	UNKNOWN	200.0
563	562	MAP06-SH	UNKNOWN	N 2ND ST	8	UNKNOWN	200.0
548	547	S-084	AUG1940	COWING ST	8	UNKNOWN	197.0
153	94	S-084	AUG1940	PINE ST	8	UNKNOWN	195.0
519	518	S-195J	DEC1966	S CENTER ST	8	UNKNOWN	195.0
503	502	S-183	JUN1964	EUREKA AVE	8	UNKNOWN	192.8
553	552	P-487	NOV1991	NORWAY AVE	8	UNKNOWN	190.0
594	591	MAP12-SH	UNKNOWN	McCLAINE ST	8	UNKNOWN	190.0
536	535	S-195C	DEC1966	ROSS AVE	8	UNKNOWN	182.0
303	292	P-278A	MAR1978	W CENTER ST	8	UNKNOWN	181.0
178	177	S-316	UKNOWN	CHESTER ST	8	UNKNOWN	180.0
504	503	S-183	JUN1964	EUREKA AVE	8	UNKNOWN	171.2
558	551	P-487	NOV1991	SWEDEN CIR	8	UNKNOWN	170.0
166	20	S-084	AUG1940	BROWN ST	8	UNKNOWN	166.0
250	209	S-084	AUG1940	5TH ST	8	UNKNOWN	165.5

City of Silverton
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US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
522	521	S-195C	DEC1966	EDGEWOOD DR	8	UNKNOWN	164.0
509	508	P-248	JAN1975	PHELPS ST	8	UNKNOWN	161.5
235	234	P-220A	AUG1976	PEACH ST	8	UNKNOWN	156.0
151	120	S-084	AUG1940	S CHURCH ST	8	UNKNOWN	152.0
222	221	S-084	AUG1940	HICKS ST	8	UNKNOWN	150.0
528	527	S-195D	FEB1967	WEIBY AVE	8	UNKNOWN	150.0
217	214	S-083	DEC1966	S 1ST ST	8	UNKNOWN	148.0
555	554	P-487	NOV1991	SWEDEN CIR	8	UNKNOWN	146.0
194	193	S-082	JUN1949	C ST	8	UNKNOWN	140.6
531	530	S-195C	DEC1966	ROSS AVE	8	UNKNOWN	139.0
508	597	S-084	AUG1940	ALDER ST	8	UNKNOWN	136.0
521	520	S-195C	DEC1966	ROSS AVE	8	UNKNOWN	136.0
230	229	S-084	AUG1940	CENTRAL ST	8	UNKNOWN	130.0
179	619	S-316	NOV1977	MILL ST	8	UNKNOWN	125.0
270	269	S-084	AUG1940	5TH ST	8	UNKNOWN	124.0
538	121	S-195C	DEC1966	KEENE AVE	8	UNKNOWN	124.0
408	407	S-428	AUG1988	WALL ST	8	UNKNOWN	121.6
292	293	MAP18-SH	UNKNOWN	KOONS ST	8	UNKNOWN	120.0
297	186	P-305	JUL1975	N SILVER LP	8	UNKNOWN	120.0
302	291	MAP18-SH	UNKNOWN	JAY ST	8	UNKNOWN	119.0
304	510A	P-305	JUL1975	S SILVER LP	8	UNKNOWN	119.0
578	573	S-428	AUG1988	N STEELHAMMER RD	8	UNKNOWN	115.0
537	AlderPS	S-195C	DEC1966	KEENE AVE	8	UNKNOWN	114.0
187	297	MAP07-SH	UNKNOWN	MILL ST	8	UNKNOWN	105.0
223	222	S-084	AUG1940	COWING ST	8	UNKNOWN	103.5
316	54	P-535	JAN1995	LONE OAKS LP	8	UNKNOWN	102.3
510	509	P-248	JAN1975	PHELPS ST	8	UNKNOWN	96.5
309	138	P-305	JUL1975	N SILVER LP	8	UNKNOWN	90.0
298	297	S-300	SEP1976	BARGER ST	8	UNKNOWN	81.0
246	237	MAP13-SH	UNKNOWN	MILL ST	8	UNKNOWN	80.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
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US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
312	248	MAP13-SH	UNKNOWN	MILL ST	8	UNKNOWN	78.0
517	516	S-195J	DEC1966	S CENTER ST	8	UNKNOWN	77.0
314	313	S-286	NOV1976	N 2ND ST	8	UNKNOWN	75.0
560	528	S-195C	DEC1966	KEENE AVE	8	UNKNOWN	75.0
516	502	S-195J	DEC1966	S CENTER ST	8	UNKNOWN	72.5
216	215	S-083	DEC1966	S 1ST ST	8	UNKNOWN	61.0
165	164	S-084	AUG1940	PINE ST	8	UNKNOWN	55.0
541	540	P-278A	MAR1978	W CENTER ST	8	UNKNOWN	48.0
163	19	S-084	AUG1940	PINE ST	8	UNKNOWN	33.0
58	25	MAP13-SH	UNKNOWN	C ST	8	UNKNOWN	30.0
274	97	S-286	NOV1976	N 2ND ST	8	UNKNOWN	29.7
512	511	S-181	APR1964	JEROME AVE	8	UNKNOWN	23.3
568	567	MAP07-SH	UNKNOWN	WHITTIER ST	8	UNKNOWN	20.0
524	624	S-195D	FEB1967	EDGEWOOD DR	8	UNKNOWN	18.9
157	156	S-351F	MAY1982	N JAMES AVE	8	UNKNOWN	14.0
231	230	P-220A	AUG1976	MADISON ST	8	UNKNOWN	13.0
453	454	S-522	APR1981	W MAIN ST	8	UNKNOWN	2.0
184	620	MAP07-SH	UNKNOWN	LINCOLN ST	8	UNKNOWN	0.0
185	184	MAP07-SH	UNKNOWN	LINCOLN ST	8	UNKNOWN	0.0
191	62	MAP06-SH	UNKNOWN	JEFFERSON ST	8	UNKNOWN	0.0
213	212	MAP18-SH	UNKNOWN	JERSEY ST	8	UNKNOWN	0.0
410	409	P-250	FEB1975	WESTFIELD ST	8	UNKNOWN	0.0
411	410	P-250	FEB1975	WESTFIELD ST	8	UNKNOWN	0.0
416	933	MAP12-SH	UNKNOWN	S JAMES AVE	8	UNKNOWN	0.0
417	415	MAP12-SH	UNKNOWN	S JAMES AVE	8	UNKNOWN	0.0
418	417	MAP12-SH	UNKNOWN	S JAMES AVE	8	UNKNOWN	0.0
419	418	MAP12-SH	UNKNOWN	McCLAINE ST	8	UNKNOWN	0.0
420	417	MAP12-SH	UNKNOWN	McCLAINE ST	8	UNKNOWN	0.0
421	420	MAP12-SH	UNKNOWN	McCLAINE ST	8	UNKNOWN	0.0
457	456	MAP17-SH	UNKNOWN	W MAIN ST	8	UNKNOWN	0.0

City of Silverton
Wastewater Facility Masterplan
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US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
458	457	MAP17-SH	UNKNOWN	W MAIN ST	8	UNKNOWN	0.0
459	458	MAP17-SH	UNKNOWN	W MAIN ST	8	UNKNOWN	0.0
510A	510	MAP17-SH	UNKNOWN	W MAIN ST	8	UNKNOWN	0.0
56A	56	MAP07-SH	UNKNOWN	WHITTIER ST	8	UNKNOWN	0.0
596	601		UNKNOWN		8	UNKNOWN	0.0
597	507		1998		8	UNKNOWN	0.0
603	537	MAP13-SH	UNKNOWN	HIGH ST	6	UNKNOWN	565.0
91	90	S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	497.5
CO265	265	S-084	AUG1940	PHELPS ST	6	UNKNOWN	475.0
93	92	S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	452.0
		S-084	AUG1940	FAIRVIEW ST	6	UNKNOWN	382.0
		S-084	AUG1940	FAIRVIEW ST	6	UNKNOWN	365.0
83	82	S-084	AUG1940	N CHURCH ST	6	UNKNOWN	360.0
96	95	MAP18-SH	UNKNOWN	COOLIDGE ST	6	UNKNOWN	345.0
88	87	MAP18-SH	UNKNOWN	DRAKE ST	6	UNKNOWN	330.0
		P-511	JAN1994	WESTFIELD ST	6	UNKNOWN	325.0
689	685	S-084	AUG1940	JERSEY ST	6	UNKNOWN	315.0
		P-465	JUN1990	CHERRY ST	6	UNKNOWN	314.0
		P-465	JUN1990	CHERRY ST	6	UNKNOWN	300.0
65	64	S-084	AUG1940	N CHURCH ST	6	UNKNOWN	298.0
68	67	S-084	AUG1940	OAK ST	6	UNKNOWN	291.5
626	625	S-084	AUG1940	N CHURCH ST	6	UNKNOWN	282.0
81	JeffersonPS	S-084	AUG1940	E MAIN ST	6	UNKNOWN	276.0
693	692	S-084	AUG1940	RESERVE ST	6	UNKNOWN	256.0
84	83	S-082	OCT1962	S 2ND ST	6	UNKNOWN	247.6
92	44	S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	238.0
686	685	S-084	AUG1940	KENT ST	6	UNKNOWN	236.6
620	56	S-083	DEC1966	S 1ST ST	6	UNKNOWN	230.0
94	93	MAP18-SH	UNKNOWN	FISKE ST	6	UNKNOWN	220.0
624	MainPS	S-084	AUG1940	N CHURCH ST	6	UNKNOWN	211.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
CO225	225	S-084	AUG1940	PHELPS ST	6	UNKNOWN	206.0
		S-084	AUG1940	PHELPS ST	6	UNKNOWN	206.0
90	89	S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	202.5
		S-084	AUG1940	W CENTER ST	6	UNKNOWN	200.0
		S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	200.0
601	551	P-155	MAY1967	N 2ND ST	6	UNKNOWN	198.5
692	689	S-084	AUG1940	ROCK ST	6	UNKNOWN	194.0
Hobart PS	861	S-084	AUG1940	WELCH ST	6	UNKNOWN	192.5
JeffersonPS	45	S-084	AUG1940	WELCH ST	6	UNKNOWN	192.5
68A	68	S-084	AUG1940	JERSEY ST	6	UNKNOWN	188.5
701	587	S-084	AUG1940	ROCK ST	6	UNKNOWN	177.0
67	66	S-084	AUG1940	OAK ST	6	UNKNOWN	174.0
66	65	S-084	AUG1940	MILL ST	6	UNKNOWN	164.5
690	689	S-084	AUG1940	UNKNOWN	6	UNKNOWN	150.0
		S-084	AUG1940	RESERVE ST	6	UNKNOWN	150.0
933	415	P-248	JAN1975	PHELPS ST	6	UNKNOWN	145.0
		S-084	AUG1940	OAK ST	6	UNKNOWN	130.1
609	194	MAP18-SH	UNKNOWN	S 2ND ST	6	UNKNOWN	112.0
85	84	S-300	SEP1976	BARGER ST	6	UNKNOWN	111.0
69	68A	S-084	AUG1940	UNKNOWN	6	UNKNOWN	96.5
89	88	S-351E	MAY1983	NORWAY AVE	6	UNKNOWN	93.9
CO288	288	S-084	AUG1940	CHERRY ST	6	UNKNOWN	90.0
		P-572	OCT1997	WELCH ST	6	UNKNOWN	89.0
82	81	S-084	AUG1940	ROCK ST	6	UNKNOWN	75.0
660	506	S-084	AUG1940	OAK ST	6	UNKNOWN	62.0
606	516	MAP13-SH	UNKNOWN	OAK ST	6	UNKNOWN	60.0
625	624	S-084	AUG1940	N CHURCH ST	6	UNKNOWN	50.0
685	212	S-084	AUG1940	DIGERNESS ST	6	UNKNOWN	50.0
691	690	S-084	AUG1940	UNKNOWN	6	UNKNOWN	50.0
		P-487	NOV1991	STARK ST	6	UNKNOWN	45.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
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924	453	S-084	AUG1940	WESTFIELD ST	6	UNKNOWN	18.0
95	94	MAP17-SH	UNKNOWN	COOLIDGE ST	6	UNKNOWN	15.0
597A	597	MAP06-SH	UNKNOWN	MONTE VISTA ST	6	UNKNOWN	0.0
619	178	MAP18-SH	UNKNOWN	S 2ND ST	6	UNKNOWN	0.0
935	421	MAP18-SH	UNKNOWN	W MAIN ST	6	UNKNOWN	0.0
CO34	34	MAP17-SH	UNKNOWN	CHERRY ST	6	UNKNOWN	0.0
FloridaPS	116	S-181	APR1964	APPLE AVE	6	UNKNOWN	0.0
		S-084	AUG1940	FAIRVIEW ST	6	UNKNOWN	0.0
		MAP17-SH	UNKNOWN	CHERRY ST	6	UNKNOWN	0.0
		MAP18-SH	UNKNOWN	S 2ND ST	6	UNKNOWN	0.0
		MAP06-SH	UNKNOWN	N 2ND ST	6	UNKNOWN	0.0
		S-084	AUG1940	FAIRVIEW ST	6	UNKNOWN	0.0
		S-351G	JUN1982	N WATER ST	6	UNKNOWN	0.0
		MAP17-SH	UNKNOWN	FAIRVIEW ST	6	UNKNOWN	0.0
		MAP17-SH	UNKNOWN	FAIRVIEW ST	6	UNKNOWN	0.0
		MAP17-SH	UNKNOWN	FAIRVIEW ST	6	UNKNOWN	0.0
			UNKNOWN		6	UNKNOWN	0.0
		S-082	JUN1965	N JAMES AVE	4	UNKNOWN	0.0
		MAP17-SH	UNKNOWN	WESTFIELD ST	4	UNKNOWN	0.0
868	867	MAP17-SH	UNKNOWN	WESTFIELD ST	4	UNKNOWN	0.0
934	933	MAP17-SH	UNKNOWN	WESTFIELD ST	4	UNKNOWN	0.0
			UNKNOWN		2	UNKNOWN	0.0
			UNKNOWN			UNKNOWN	0.0
						UNKNOWN	
						Mat. Total	**63525**
103	102	MAP06-SH	UNKNOWN	N 2ND ST	UNKNOWN	CONC	30.0
133	132	S-084	AUG1940	LANE ST	12	CONC	345.0
591	590	S-351G	JUN1982	S WATER ST	8	CONC	685.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
		S-084	AUG1940	STARK ST	8	CONC	591.1
212	211	MAP18-SH	UNKNOWN	JERSEY ST	8	CONC	580.0
210	209	MAP18-SH	UNKNOWN	E MAIN ST	8	CONC	520.0
262	263	S-084	AUG1940	CHADWICK ST	8	CONC	448.0
570	569	MAP07-SH	UNKNOWN	MILL ST	8	CONC	440.0
582	532	MAP13-SH	UNKNOWN	B ST	8	CONC	440.0
186	55	MAP07-SH	UNKNOWN	LINCOLN ST	8	CONC	405.0
183	56	MAP06-SH	UNKNOWN	N 2ND ST	8	CONC	400.0
515	514	MAP17-SH	UNKNOWN	WELCH ST	8	CONC	400.0
25	24	S-084	AUG1940	SHERIDAN ST	8	CONC	400.0
196	195	MAP12-SH	UNKNOWN	N 1ST ST	8	CONC	385.0
592	591	MAP18-SH	UNKNOWN	W MAIN ST	8	CONC	385.0
59	58	S-428	AUG1988	WALL ST	8	CONC	380.0
593	592	MAP17-SH	UNKNOWN	W MAIN ST	8	CONC	365.0
567	566	MAP07-SH	UNKNOWN	WHITTIER ST	8	CONC	360.0
211	210	MAP18-SH	UNKNOWN	LEWIS ST	8	CONC	350.0
247	235	S-083	DEC1966	ROCK ST	8	CONC	346.4
		P-474	AUG1990	MILL ST	8	CONC	340.0
240	239	S-272	FEB1976	ROBINSON ST	8	CONC	340.0
209	208	MAP06-SH	UNKNOWN	LINCOLN ST	8	CONC	330.0
583	582	S-290	DEC1977	HILL ST	8	CONC	330.0
		S-522	DEC1982	N JAMES AVE	8	CONC	327.0
		S-084	AUG1940	LONE OAKS LP	8	CONC	323.0
193	192	MAP12-SH	UNKNOWN	C ST	8	CONC	320.0
569	568	MAP07-SH	UNKNOWN	WHITTIER ST	8	CONC	320.0
239	238	S-288	JUL1977	RESERVE ST	8	CONC	315.0
201	200	MAP13-SH	UNKNOWN	HIGH ST	8	CONC	310.0
588	701	S-522	APR1981	W MAIN ST	8	CONC	310.0
295	196	MAP12-SH	UNKNOWN	MEADE ST	8	CONC	308.0
200	199	MAP13-SH	UNKNOWN	PARK ST	8	CONC	300.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
514	513	MAP17-SH	UNKNOWN	W MAIN ST	8	CONC	295.0
571	566	MAP07-SH	UNKNOWN	MILL ST	8	CONC	295.0
176	175	MAP07-SH	UNKNOWN	CHESTER ST	8	CONC	290.0
169	21	MAP07-SH	UNKNOWN	WASHINGTON ST	8	CONC	285.0
175	23	MAP07-SH	UNKNOWN	MILL ST	8	CONC	285.0
238	237	P-350	SEP1979	N 1ST ST	8	CONC	285.0
248	247	S-351G	JUN1982	S WATER ST	8	CONC	282.0
181	176	MAP07-SH	UNKNOWN	JEFFERSON ST	8	CONC	280.0
		S-351	OCT1984	WASTE TREATMENT PLANT	8	CONC	278.0
219	211	P-474	AUG1990	MILL ST	8	CONC	269.0
581	530	MAP13-SH	UNKNOWN	B ST	8	CONC	260.0
273	41	S-290	DEC1977	HILL ST	8	CONC	255.0
253	78	P-301	DEC1973	WALNUT AVE	8	CONC	251.0
		S-260	DEC1975	FISKE ST	8	CONC	250.0
259	253	P-301	DEC1973	WALNUT AVE	8	CONC	249.0
264	220	S-083	DEC1966	5TH ST	8	CONC	240.7
		S-258	APR1975	GRANT ST	8	CONC	233.0
24	23	S-288	JUL1977	RESERVE ST	8	CONC	231.0
564	563	MAP06-SH	UNKNOWN	N 2ND ST	8	CONC	230.0
254	253	P-301	DEC1973	WALNUT AVE	8	CONC	225.0
587	586	S-522	APR1981	W MAIN ST	8	CONC	220.0
		S-351	OCT1984	WASTE TREATMENT PLANT	8	CONC	210.1
263	264	S-083	DEC1966	5TH ST	8	CONC	209.3
266	288	S-095	DEC1957	AMES ST	8	CONC	200.0
281	07	S-083	DEC1966	E MAIN ST	8	CONC	199.0
562	561	MAP06-SH	UNKNOWN	JEFFERSON ST	8	CONC	195.0
589	506	S-428	AUG1988	ESKA WAY	8	CONC	195.0
247A	247	S-083	DEC1966	ROCK ST	8	CONC	192.0
505	504	P-637	MAY1999	EUREKA AVE	8	CONC	190.0
533	532	MAP17-SH	UNKNOWN	W MAIN ST	8	CONC	190.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

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590	589	S-428	AUG1988	OAK ST	8	CONC	185.0
584	582	MAP13-SH	UNKNOWN	N 2ND ST	8	CONC	180.0
168	142	MAP07-SH	UNKNOWN	WASHINGTON ST	8	AC	170.0
566	565	MAP06-SH	UNKNOWN	N 2ND ST	8	CONC	170.0
256	254	P-301	DEC1973	WALNUT AVE	8	CONC	170.0
506	505	P-637	MAY1999	EUREKA AVE	8	CONC	160.0
565	564	MAP06-SH	UNKNOWN	N 2ND ST	8	CONC	145.0
		S-268	JAN1977	MILL ST	8	CONC	144.0
		P-301	DEC1973	FILBERT WAY	8	CONC	143.0
26	25	P-301	DEC1973	WALNUT WAY	8	CONC	132.0
197	195	MAP13-SH	UNKNOWN	N 1ST ST	8	CONC	130.0
199	65	MAP13-SH	UNKNOWN	N 1ST ST	8	CONC	130.0
579	564	MAP12-SH	UNKNOWN	C ST	8	CONC	130.0
260	261	P-301	DEC1973	WALNUT WAY	8	CONC	130.0
		S-211	APR1969	JEFFERSON ST	8	CONC	125.0
146	145	MAP13-SH	UNKNOWN	OAK ST	8	CONC	120.0
586	585	P-507	MAR1994	HOBART RD	8	CONC	120.0
296	295	MAP13-SH	UNKNOWN	N 2ND ST	8	CONC	111.0
203	202	MAP13-SH	UNKNOWN	OAK ST	8	CONC	110.0
585	533	MAP12-SH	UNKNOWN	PINE ST	8	CONC	110.0
299	298	S-084	AUG1940	S 3RD ST	8	CONC	100.0
540	539	S-195C	DEC1966	KEENE AVE	8	CONC	100.0
245	244	S-272	FEB1976	ROBINSON ST	8	CONC	100.0
28	27	S-095	DEC1957	E MAIN ST	8	CONC	80.0
255	254	P-301	DEC1973	WALNUT AVE	8	CONC	75.0
261	262	S-351	OCT1984	WASTE TREATMENT PLANT	8	CONC	57.0
		S-351	OCT1984	WASTE TREATMENT PLANT	8	CONC	50.0
27	311	S-282	FEB1977	D ST	8	CONC	47.0
		S-522	DEC1982	N JAMES AVE	8	CONC	33.0
199	66	P-155	MAY1967	N 2ND ST	8	CONC	20.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
		S-351	OCT1984	WASTE TREATMENT PLANT	8	CONC	10.0
		S-178	JUL1963	RESERVE ST	8	CONC	3.0
201	201	MAP13-SH	UNKNOWN	HIGH ST	8	CONC	0.0
170	169	MAP18-SH	UNKNOWN	E MAIN ST	8	CONC	0.0
		S-351G	JUN1982	S WATER ST	8	CONC	0.0
99	98	MAP17-SH	UNKNOWN	WELCH ST	6	CONC	470.0
87	86	MAP13-SH	UNKNOWN	5TH ST	6	CONC	390.0
86	85	MAP17-SH	UNKNOWN	SOUTH ST	6	CONC	360.0
64	63	S-084	AUG1940	N CHURCH ST	6	CONC	175.0
607	606	MAP18-SH	UNKNOWN	S 2ND ST	6	CONC	165.0
612	241	MAP18-SH	UNKNOWN	S 2ND ST	6	CONC	85.0
						Mat. Total	**24833**
07	06	S-351G	JUN1982	PINE ST	30	DI	865.9
06	05	S-351G	JUN1982	PINE ST	30	DI	456.0
05	04	S-351G	JUN1982	WASTE TREATMENT PLANT	30	DI	301.3
613	612	S-351	OCT1984	WASTE TREATMENT PLANT	30	DI	0.0
859	858	P-547-2	AUG1998	MONSON RD	12	DI	20.0
860	858	S-288	JUL1977	RESERVE ST	8	DI	68.2
		P-395	NOV1984	IKE MOONEY RD	8	DI	45.0
		P-547-2	AUG1998	MONSON RD	8	DI	16.0
GrantPS	110	P-507	MAR1994	ESKA WAY	8	DI	0.0
ORGardPS	513	P-507	MAR1994	ESKA WAY	8	DI	0.0
		P-547-2	AUG1998	MONSON RD	8	DI	0.0
		P-521	UNKNOWN	GRANT ST	6	DI	0.0
		P-521	JUN1983	ALDER ST	4	DI	7.0
		P-547C	JAN1998	OREGON GARDENS	4	DI	0.0
		S-351F	MAY1982	N JAMES AVE	4	DI	0.0
		P-402	SEP1984	W MAIN ST	4	DI	0.0
						Mat. Total	**1779**

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
111	GrantPS	P-699	JUL2004	IKE MOONEY RD	UNKNOWN	PVC	10.0
1116	ORGardPS	P-553	JUN1995	N 1ST ST	UNKNOWN	PVC	0.0
1118	1117	P-547W	MAR2000	WASTE TREATMENT PLANT	30	PVC	297.5
1120	1119	P-547W	MAR2000	WASTE TREATMENT PLANT	30	PVC	82.0
1119	1118	P-547W	MAR2000	WASTE TREATMENT PLANT	30	PVC	35.0
1127	1126	S-705	AUG2004	S WATER ST	18	PVC	470.0
1125	1124	S-705	AUG2004	S WATER ST	18	PVC	297.0
1124	1123	S-705	AUG2004	S WATER ST	18	PVC	291.0
1123	1121	S-705	AUG2004	S WATER ST	18	PVC	265.0
1122	1123	S-705	AUG2004	S WATER ST	18	PVC	260.0
1126	1125	S-705	AUG2004	S WATER ST	18	PVC	209.0
1121	1120	S-705	AUG2004	S WATER ST	18	PVC	30.0
1129	1128	P-547-2	AUG1998	SILVERTON RD	16	PVC	0.0
1129	1128	P-547-2	AUG1998	MONSON RD	16	PVC	0.0
1130	1123	P-547-2	AUG1998	RAILWAY AVE	16	PVC	0.0
1131	1130	P-547-2	AUG1998	W MAIN ST	16	PVC	0.0
1132	1131	P-547-2	AUG1998	MONSON RD	16	PVC	0.0
1133	1132	MAP22-SH	JAN1998	OREGON GARDENS	16	PVC	0.0
1134	1133	P-547W	MAR2000	WASTE TREATMENT PLANT	16	PVC	0.0
1137	1131	P-547W	MAR2000	WASTE TREATMENT PLANT	16	PVC	0.0
1138	1137	P-547W	MAR2000	WASTE TREATMENT PLANT	16	PVC	0.0
1154	80	P-507	MAR1994	ESKA WAY	15	PVC	497.0
1148	1147	S-705	AUG2004	S WATER ST	15	PVC	467.0
1140	654	P-547-2	AUG1998	MONSON RD	15	PVC	350.0
1141	1140	P-547-2	AUG1998	MONSON RD	15	PVC	350.0
1142	1141	P-547-2	AUG1998	MONSON RD	15	PVC	350.0
1149	1150A	S-705	AUG2004	S WATER ST	15	PVC	350.0
1143	1142	P-547-2	AUG1998	MONSON RD	15	PVC	348.0
1150	73	S-705	AUG2004	S WATER ST	15	PVC	324.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

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1152	73	S-705	AUG2004	S WATER ST	15	PVC	261.0
1147	1146	S-705	AUG2004	S WATER ST	15	PVC	258.0
1150A	1150	S-705	AUG2004	S WATER ST	15	PVC	215.0
1144	1145	P-547-2	AUG1998	SILVERTON RD	15	PVC	152.0
1151	1150	S-705	AUG2004	S WATER ST	15	PVC	135.0
1153	80	P-507	MAR1994	ESKA WAY	15	PVC	97.0
1139	1138	S-705	AUG2004	IKE MOONEY RD	15	PVC	53.0
1145	1146	P-547-2	AUG1998	MONSON RD	15	PVC	18.0
1146	70	P-500	SEP1993	LONE OAKS LP	15	PVC	0.0
1155	1154	P-547-2	AUG1998	RAILWAY AVE	15	PVC	0.0
1164	1163	P-699	JUL2004	PIONEER DR	12	PVC	476.0
1159	1158	P-699	JUL2004	S WATER ST	12	PVC	473.0
1170	1169	P-547-2	AUG1998	RAILWAY AVE	12	PVC	440.0
1163	1161	P-699	JUL2004	PIONEER DR	12	PVC	321.0
1302	1301	P-597	JAN1998	SCHEMMEL LN	12	PVC	291.0
1305	1304	P-597	JAN1998	SCHEMMEL LN	12	PVC	245.0
1304	1303	P-597	JAN1998	SCHEMMEL LN	12	PVC	243.0
1307	1306	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	205.0
1158	1157	P-699	JUL2004	S WATER ST	12	PVC	201.0
1160	1158	P-699	JUL2004	S WATER ST	12	PVC	177.0
1303	1302	P-597	JAN1998	SCHEMMEL LN	12	PVC	177.0
1157	1156	P-627	JUL1999	S WATER ST	12	PVC	167.9
1156	1155	P-627	JUL1999	S WATER ST	12	PVC	159.7
1165	1164	P-699	JUL2004	PIONEER DR	12	PVC	156.0
1169	1168	P-699	JUL2004	PIONEER DR	12	PVC	155.0
1162	1161	P-699	JUL2004	PIONEER DR	12	PVC	131.0
1168	1164	P-699	JUL2004	PIONEER DR	12	PVC	123.0
117	1116	P-699	JUL2004	IKE MOONEY RD	12	PVC	110.0
227	226	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	104.0
206	GrantPS	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	93.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

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188B	188A	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	75.0
188C	188B	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	52.0
1161	1160	P-699	JUL2004	S WATER ST	12	PVC	50.0
188A	188	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	38.0
188	27A	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	32.0
1306	1305	P-597	JAN1998	WASTE TREATMENT PLANT	12	PVC	0.0
237	247A	P-547W	MAR2000	WASTE TREATMENT PLANT	12	PVC	0.0
285	28	P-507	MAR1994	ESKA WAY	10	PVC	400.0
251	76	P-507	MAR1994	ESKA WAY	10	PVC	320.0
286	285	P-597	JAN1998	SILVERTON HIGH SCHOOL	10	PVC	298.0
291	75	P-597	JAN1998	SILVERTON HIGH SCHOOL	10	PVC	228.0
252A	252	P-507	MAR1994	ESKA WAY	10	PVC	173.0
252	1153	P-507	MAR1994	ESKA WAY	10	PVC	157.0
275	13	P-507	MAR1994	ESKA WAY	10	PVC	147.0
293	74	P-547W	MAR2000	WASTE TREATMENT PLANT	10	PVC	0.0
299A	299	P-547-2	AUG1998	MONSON RD	10	PVC	0.0
301	MonsonPS	P-547-2	AUG1998	RAILWAY AVE	10	PVC	0.0
863	862	P-565	JUL1997	CHEE CHEE CT	8	PVC	731.0
		S-522	APR1981	W MAIN ST	8	PVC	604.0
450	405	MAP12-SH	UNKNOWN	McCLAINE ST	8	PVC	468.0
616	612	P-633	JAN2001	JOHNA LN	8	PVC	453.0
954	953	P-629	JAN1999	CRESTVIEW DR	8	PVC	438.0
79	78	P-476	NOV1991	EDGEWOOD DR	8	PVC	410.0
904	903	P-507	MAR1994	ESKA WAY	8	PVC	402.0
903	AprilPS	P-507	MAR1994	ESKA WAY	8	PVC	400.0
905	04	P-507	MAR1994	ESKA WAY	8	PVC	400.0
958	957	P-665	MAY2001	IOWA ST	8	PVC	400.0
959	960	P-665	MAY2001	IOWA ST	8	PVC	400.0
867	861	P-583	NOV1997	KLOSHE CT	8	PVC	393.3
941	939	S-382	JUL1981	N 1ST ST	8	PVC	386.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

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650	649	S-191	NOV1966	FENNE LN (PVT)	8	PVC	375.0
672	46	P-305	JUL1975	S SILVER LP	8	PVC	365.0
846	847	P-565	JUL1997	SHELOKUM DR	8	PVC	354.0
802	801	SD-475	AUG1990	N 2ND ST	8	PVC	343.8
845	846	P-565	JUL1997	SHELOKUM DR	8	PVC	343.0
963	188A	P-665	MAY2001	PRINCETON AVE	8	PVC	336.0
SilvertonHSPS	896	P-700	DEC2003	EAST VIEW LN	8	PVC	330.0
946	947	P-629	JAN1999	S STEELHAMMER RD	8	PVC	327.0
800A	800	S-484	NOV1991	EDGEWOOD DR	8	PVC	323.0
864	863	P-565	JUL1997	ADAMS ST	8	PVC	321.0
629	66	P-612	OCT1998	ENSTAD LN	8	PVC	319.0
964	805	P-665	MAY2001	CAMBRIDGE AVE.	8	PVC	316.0
CO227	227	P-700	DEC2003	TILLICUM DR	8	PVC	316.0
70	33B	P-500	SEP1993	LONE OAKS LP	8	PVC	309.7
456	455	MAP17-SH	UNKNOWN	W MAIN ST	8	PVC	308.0
960	961	P-665	MAY2001	SPENCER DR (PVT)	8	PVC	305.0
948	947	P-629	JAN1999	CRESTVIEW DR	8	PVC	303.0
686A	686	P-305	JUL1975	N SILVER LP	8	PVC	300.0
75	74	P-476	NOV1991	ANDERSON DR	8	PVC	295.0
824	286	P-565	JUL1997	TILLICUM DR	8	PVC	290.0
809	810	P-564	JUN1996	DENMARK LP	8	PVC	289.9
688	687	P-525	DEC1994	MILL ST	8	PVC	288.0
922	627	P-566	JUN1995	ESKA WAY	8	PVC	284.0
815	816	P-534	MAR1994	KEENE AVE	8	PVC	274.0
702	701	P-469	JUN1991	ANDERSON DR	8	PVC	272.0
940	939	S-638	NOV2000	ORCHARD ST	8	PVC	270.0
752	751	S-484	NOV1991	LOT	8	PVC	265.0
811	821	P-564	JUN1996	SWEDEN CIR	8	PVC	264.0
944	943	P-629	JAN1999	S STEELHAMMER RD	8	PVC	263.0
951	950	P-629	JAN1999	BREYONNA WAY	8	PVC	262.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
949	418	P-629	JAN1999	BREYONNA WAY	8	PVC	259.0
643	642	P-699	JUL2004	DIVISION ST	8	PVC	256.0
653	652	S-435	AUG1988	WEBB ST	8	PVC	256.0
855	854	P-565	JUL1997	CHIKAMIN LP	8	PVC	256.0
AprilPS	754A	P-700	DEC2003	TILLICUM DR	8	PVC	254.0
393	400	P-547-PH1	JAN1998	OREGON GARDENS	8	PVC	250.0
73	72	S-379	MAY1981	SMITH ST	8	PVC	250.0
617	616	P-633	JAN2001	ENSTAD LN	8	PVC	246.0
754	753	P-476	NOV1991	ANDERSON DR	8	PVC	246.0
822	823	P-565	JUL1997	RESERVE ST	8	PVC	244.0
920	119	P-507	MAR1994	ESKA WAY	8	PVC	244.0
33A	33	P-547C	JAN1998	OREGON GARDENS	8	PVC	242.0
945	944	P-629	JAN1999	S STEELHAMMER RD	8	PVC	242.0
		P-474	AUG1990	MILL ST	8	PVC	240.0
694A	686	P-525	DEC1994	BOEDIE'S DR	8	PVC	238.0
455	408	MAP17-SH	UNKNOWN	W MAIN ST	8	PVC	235.0
972	971	P-700	DEC2003	TILLICUM DR	8	PVC	233.0
961	953	P-665	MAY2001	CAMBRIDGE AVE.	8	PVC	228.0
74	73	P-469	JUN1991	ANDERSON DR	8	PVC	225.0
753	752	S-484	NOV1991	LOT	8	PVC	225.0
79A	79	S-484	NOV1991	EDGEWOOD DR	8	PVC	225.0
71	70	P-469	JUN1991	ANDERSON DR	8	PVC	223.0
392	393	P-547-PH1	JAN1998	OREGON GARDENS	8	PVC	220.0
967A	967	P-665	MAY2001	BREYONNA WAY	8	PVC	220.0
751	905	P-476	NOV1991	ANDERSON DR	8	PVC	219.0
850	410	P-565	JUL1997	CHEE CHEE CT	8	PVC	218.0
957	955	P-665	MAY2001	IOWA ST	8	PVC	213.0
694	694A	P-525	DEC1994	BOEDIE'S DR	8	PVC	212.0
317	318	P-547C	JAN1998	OREGON GARDENS	8	PVC	210.0
754B	754A	P-476	NOV1991	EDGEWOOD DR	8	PVC	210.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
652	651	P-368	FEB1981	IKE MOONEY RD	8	PVC	208.0
866	865	P-565	JUL1997	CHEE CHEE CT	8	PVC	208.0
642	641	P-699	JUL2004	WAGON WHEEL CT	8	PVC	207.0
967	966	P-665	MAY2001	BREYONNA WAY	8	PVC	207.0
825	823	P-565	JUL1997	SHELOKUM DR	8	PVC	206.0
947	941	P-629	JAN1999	S STEELHAMMER RD	8	PVC	206.0
852	853	P-565	JUL1997	SHELOKUM DR	8	PVC	205.0
942	941	P-486	JUL1991	N 2ND ST	8	PVC	205.0
387	388	P-547C	JAN1998	OREGON GARDENS	8	PVC	201.0
574	573	P-702	NOV2003	SHADOW RIDGE CT	8	PVC	199.7
953	952	P-629	JAN1999	HILLSDALE LN	8	PVC	197.0
970	967	P-565	JUL1997	TILLICUM DR	8	PVC	194.0
804	805	P-564	JUN1996	NORWAY AVE	8	PVC	193.6
812	811	P-564	JUN1996	SWEDEN CIR	8	PVC	189.0
640	575	P-612	OCT1998	ENSTAD LN	8	PVC	187.0
819	820	P-631	FEB2000	PINE ST	8	PVC	187.0
388	390	P-547-PH1	JAN1998	OREGON GARDENS	8	PVC	185.0
927	608	P-631	FEB2000	COX WAY	8	PVC	182.0
700	697	SD-475	AUG1990	N 2ND ST	8	PVC	181.2
649	648	S-451	JUL1988	GRANT ST	8	PVC	180.0
950	949	P-629	JAN1999	BREYONNA WAY	8	PVC	178.0
80	79A	S-484	NOV1991	EDGEWOOD DR	8	PVC	175.0
MonsonPS	04	P-700	DEC2003	KALAPUYA DR	8	PVC	175.0
573	572	P-702	NOV2003	SHADOW RIDGE CT	8	PVC	173.3
575	574	P-633	JAN2001	ENSTAD CT	8	PVC	173.0
896P	897P	P-583	NOV1997	KLOSHE CT	8	PVC	172.0
77	76	P-476	NOV1991	EDGEWOOD DR	8	PVC	170.0
938	937	P-631	FEB2000	APRIL LANE NE	8	PVC	166.0
390	391	P-547-PH1	JAN1998	OREGON GARDENS	8	PVC	165.0
CO262	262	P-700	DEC2003	EAST VIEW LN	8	PVC	165.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
818	817	P-631	FEB2000	PINE ST	8	PVC	164.0
687	686A	P-583	NOV1997	JAY ST	8	PVC	163.0
853	854	P-565	JUL1997	CHIKAMIN LP	8	PVC	163.0
		S-690	MAY2002	LIBERTY ST	8	PVC	163.0
703	702	P-469	JUN1991	ANDERSON DR	8	PVC	160.0
848	299A	P-565	JUL1997	SHELOKUM DR	8	PVC	156.0
696	695	P-525	DEC1994	BRYAN CT	8	PVC	154.0
851	852	P-565	JUL1997	SHELOKUM DR	8	PVC	150.0
854	856	P-565	JUL1997	CHIKAMIN LP	8	PVC	148.0
808	809	P-487	NOV1991	DENMARK LP	8	PVC	143.0
816	819	P-534	MAR1994	WEIBY AVE	8	PVC	140.0
820	821	P-565	JUL1997	RESERVE ST	8	PVC	140.0
968	969	S-483	JUN1991	CHARLES ST	8	PVC	138.0
969	965	P-618	JAN2000	CLIFF CT	8	PVC	138.0
76	75	P-476	NOV1991	EDGEWOOD DR	8	PVC	137.0
618	617	P-633	JAN2001	DENTON CT	8	PVC	135.0
813	812	P-564	JUN1996	SWEDEN CIR	8	PVC	134.0
857	856	P-565	JUL1997	CHEE CHEE CT	8	PVC	133.0
823	824	P-565	JUL1997	TILLICUM DR	8	PVC	132.0
Liberty CO	455	P-700	DEC2003	PUBLIC WALKWAY	8	PVC	130.0
695	694	P-525	DEC1994	TODD CT	8	PVC	128.0
803	800	P-487	NOV1991	NORWAY AVE	8	PVC	127.0
698	697	P-525	DEC1994	BRYAN CT	8	PVC	126.0
648	647	P-618	JAN2000	KOONS ST	8	PVC	124.0
971	970	P-700	DEC2003	TILLICUM DR	8	PVC	123.0
936	935	P-631	FEB2000	APRIL LANE NE	8	PVC	121.0
33B	33A	P-547C	JAN1998	OREGON GARDENS	8	PVC	120.0
645	640	S-605	UNKNOWN	KOONS ST	8	PVC	120.0
813A	813	P-564	JUN1996	SWEDEN CIR	8	PVC	120.0
955	954	P-629	JAN1999	CRESTVIEW DR	8	PVC	120.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
572	571	P-702	NOV2003	SHADOW RIDGE CT	8	PVC	119.8
847	848	P-565	JUL1997	SHELOKUM DR	8	PVC	119.0
697	687	P-525	DEC1994	BRYAN CT	8	PVC	117.0
952	951	P-629	JAN1999	HILLSDALE LN	8	PVC	117.0
956	955	P-665	MAY2001	IOWA ST	8	PVC	117.0
389	388	P-547C	JAN1998	OREGON GARDENS	8	PVC	115.0
754A	754	P-476	NOV1991	ANDERSON DR	8	PVC	112.0
78	77	P-476	NOV1991	EDGEWOOD DR	8	PVC	111.0
641	640	P-612	OCT1998	ENSTAD LN	8	PVC	109.0
699	697	P-525	DEC1994	TODD CT	8	PVC	109.0
962	549	P-665	MAY2001	CAMBRIDGE AVE.	8	PVC	106.0
		S-704	MAR2003	DIGERNESS ST	8	PVC	106.0
821	822	P-565	JUL1997	RESERVE ST	8	PVC	100.0
865	864	P-565	JUL1997	ADAMS ST	8	PVC	100.0
MainPS	21	P-700	DEC2003	KALAPUYA DR	8	PVC	100.0
		P-661	DEC2002	CRAIG ST	8	PVC	98.0
817	816	S-568	AUG1995	SHERIDAN ST	8	PVC	96.9
943	942	P-615	JAN1999	SILVER CLIFF DR W (PVT)	8	PVC	96.1
712	711	P-469	JUN1991	ANDERSON DR	8	PVC	96.0
452	453	MAP17-SH	UNKNOWN	McCLAINE ST	8	PVC	94.0
386	387	P-547C	JAN1998	OREGON GARDENS	8	PVC	92.0
858	Hobart PS	P-565	JUL1997	CHEE CHEE CT	8	PVC	90.0
897P	SilvertonHSPS	P-583	NOV1997	JAY ST	8	PVC	89.0
810	811	P-564	JUN1996	DENMARK LP	8	PVC	86.0
391	392	P-547-PH1	JAN1998	OREGON GARDENS	8	PVC	85.0
608	318A	P-633	JAN2001	ENSTAD LN	8	PVC	83.0
805	807	P-564	JUN1996	NORWAY AVE	8	PVC	80.0
647	569	S-605	UNKNOWN	KOONS ST	8	PVC	71.8
627	621	P-633	JAN2001	ENSTAD LN	8	PVC	70.0
966	965	P-665	MAY2001	BREYONNA WAY	8	PVC	68.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
861	862	P-565	JUL1997	CHEE CHEE CT	8	PVC	67.0
807	808	P-564	JUN1996	DENMARK LP	8	PVC	67.0
896	754	P-583	NOV1997	KLOSHE CT	8	PVC	63.0
851	850	P-565	JUL1997	SHELOKUM DR	8	PVC	60.0
856	858	P-565	JUL1997	CHIKAMIN LP	8	PVC	56.0
		P-637	MAY1999	EUREKA AVE	8	PVC	56.0
644	643	P-547C	JAN1998	OREGON GARDENS	8	PVC	50.0
		P-629	JAN1999	S STEELHAMMER RD	8	PVC	43.0
965	964	P-665	MAY2001	CAMBRIDGE AVE.	8	PVC	40.0
72	71	S-382	JUL1981	N 1ST ST	8	PVC	34.0
46A	46	P-702	NOV2003	EDGEWOOD DR	8	PVC	30.3
67	33	MAP18-SH	UNKNOWN	ADAMS ST	8	PVC	30.0
801	800	S-351G	JUN1982	S WATER ST	8	PVC	30.0
937	936	P-631	FEB2000	APRIL LANE NE	8	PVC	24.0
		P-566	JUN1995	ESKA WAY	8	PVC	23.0
925	410	P-597	JAN1998	SILVERTON HIGH SCHOOL	8	PVC	16.0
		P-482	JUN1991	MILL ST	8	PVC	12.0
308	55	P-521	JUN1983	GRANT ST	8	PVC	0.0
395	394	MAP22-SH	JAN2003	OREGON GARDENS	8	PVC	0.0
396	47	MAP22-SH	JAN2003	OREGON GARDENS	8	PVC	0.0
646	645	S-605	UNKNOWN	KOONS ST	8	PVC	0.0
651	647	P-368	FEB1981	IKE MOONEY RD	8	PVC	0.0
654	653	MAP18-SH	UNKNOWN	ADAMS ST	8	PVC	0.0
901	902	P-507	MAR1994	ESKA WAY	8	PVC	0.0
902	903	P-507	MAR1994	ESKA WAY	8	PVC	0.0
923	46A	P-507	MAR1994	HOBART RD	8	PVC	0.0
		P-597	JAN1998	SILVERTON HIGH SCHOOL	8	PVC	0.0
		P-547-2	AUG1998	MONSON RD	8	PVC	0.0
		S-705	AUG2004	S WATER ST	8	PVC	0.0
		S-705	AUG2004	S WATER ST	8	PVC	0.0

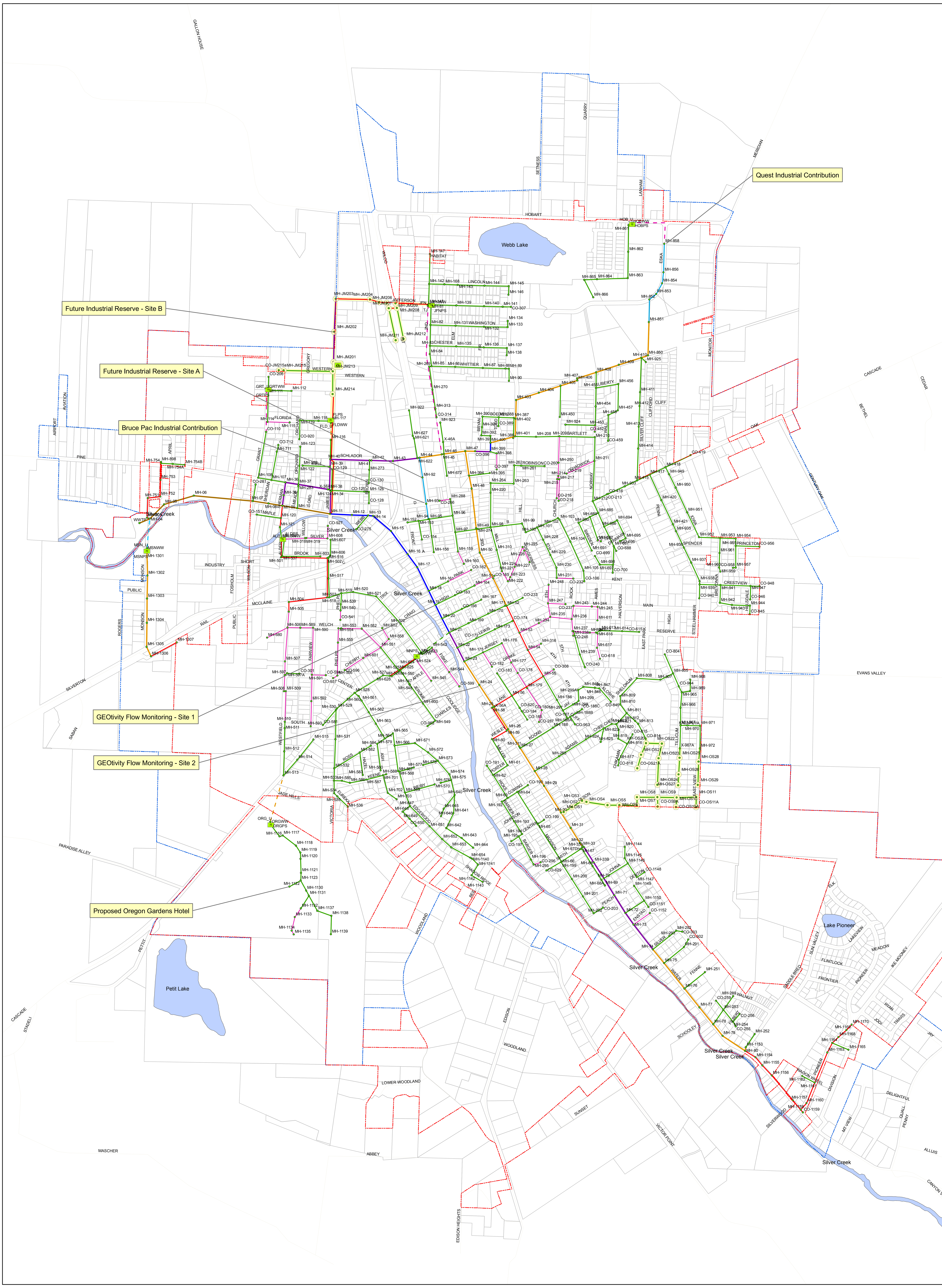
City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
		S-705	AUG2004	IKE MOONEY RD	8	PVC	0.0
		P-631	FEB2000	COX WAY	8	PVC	0.0
		MAP22-SH	JAN2003	OREGON GARDENS	8	PVC	0.0
		P-507	MAR1994	ESKA WAY	8	PVC	0.0
		P-507	MAR1994	ESKA WAY	8	PVC	0.0
		P-637	MAY1999	EUREKA AVE	8	PVC	0.0
		P-665	MAY2001	IOWA ST	8	PVC	0.0
		P-665	MAY2001	IOWA ST	8	PVC	0.0
		P-507	MAR1994	ESKA WAY	6	PVC	800.0
		P-541	JUL1994	AMES CT	6	PVC	220.0
		S-391	MAY1987	WILLOW ST	6	PVC	210.0
		P-547-PH1	JAN1998	OREGON GARDENS	6	PVC	195.0
		P-362	MAY1981	LANE ST	6	PVC	190.0
		P-551	MAR1995	S JAMES AVE	6	PVC	174.0
		P-541	JUL1994	AMES CT	6	PVC	162.0
		P-547-PH1	JAN1998	OREGON GARDENS	6	PVC	158.0
		S-084	AUG1940	DIGERNESS ST	6	PVC	146.0
		S-381	SEP1981	E MAIN ST	6	PVC	50.0
		P-551	MAR1995	SILVER ST	6	PVC	47.0
		P-678	SEP2001	WELCH ST	6	PVC	42.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		MAP22-SH	JAN2000	OREGON GARDENS	6	PVC	0.0
		S-705	AUG2004	ENSTAD LN	6	PVC	0.0
		P-541	JUL1994	RESERVE ST	6	PVC	0.0
		MAP12-SH	JAN2003	McCLAINE ST	6	PVC	0.0
		MAP18-SH	JAN2004	WESLEY ST	6	PVC	0.0

City of Silverton
Wastewater Facility Masterplan
Summary of Recommended Sanitary Sewer Segments for Condition Assessment
Sorted by Priority

US_MH	DS_MH	SOURCE	DATE_	LOCATION	DIA (in.)	MATERIAL	LENGTH (ft.)
		P-541	JUL1994	RESERVE ST	6	PVC	0.0
		P-482	JUN1991	MILL ST	6	PVC	0.0
		P-482	JUN1991	MILL ST	6	PVC	0.0
		P-482	JUN1991	MILL ST	6	PVC	0.0
		P-482	JUN1991	MILL ST	6	PVC	0.0
		P-482	JUN1991	MILL ST	6	PVC	0.0
		P-406	SEP1985	4TH ST	6	PVC	0.0
		MAP17-SH	UNKNOWN	PHELPS ST	6	PVC	0.0
		MAP17-SH	UNKNOWN	PHELPS ST	6	PVC	0.0
		MAP17-SH	UNKNOWN	PHELPS ST	6	PVC	0.0
		P-631	FEB2000	APRIL LANE NE	4	PVC	360.0
		S-494	APR1993	COOLIDGE ST	4	PVC	31.0
		S-704	MAR2003	DIGERNESS ST	4	PVC	25.0
		S-494	APR1993	COOLIDGE ST	4	PVC	0.0
		S-494	APR1993	COOLIDGE ST	4	PVC	0.0
		S-494	APR1993	COOLIDGE ST	4	PVC	0.0
		S-494	APR1993	COOLIDGE ST	4	PVC	0.0
		P-553	JUN1995	N 1ST ST	4	PVC	0.0
		P-553	JUN1995	N 1ST ST	3	PVC	0.0
		P-646	NOV1998	WESTFIELD ST	2	PVC	458.0
		P-661	DEC2002	CRAIG ST	2	PVC	136.0
		P-661	DEC2002	CRAIG ST	2	PVC	136.0
		P-661	DEC2002	CRAIG ST	2	PVC	69.0
		P-553	JUN1995	N 1ST ST	2	PVC	0.0
		MAP12-SH	UNKNOWN	TRIX ST	2	PVC	0.0
		MAP12-SH	UNKNOWN	TRIX ST	2	PVC	0.0
						Mat. Total	**52081**

Total Pipe Length **148293**



Legend

Gravity Main
- 6 inches
- 8 inches
- 10 inches
- 12 inches
- 15 inches
- 18 inches
- 21 inches
- 30 inches

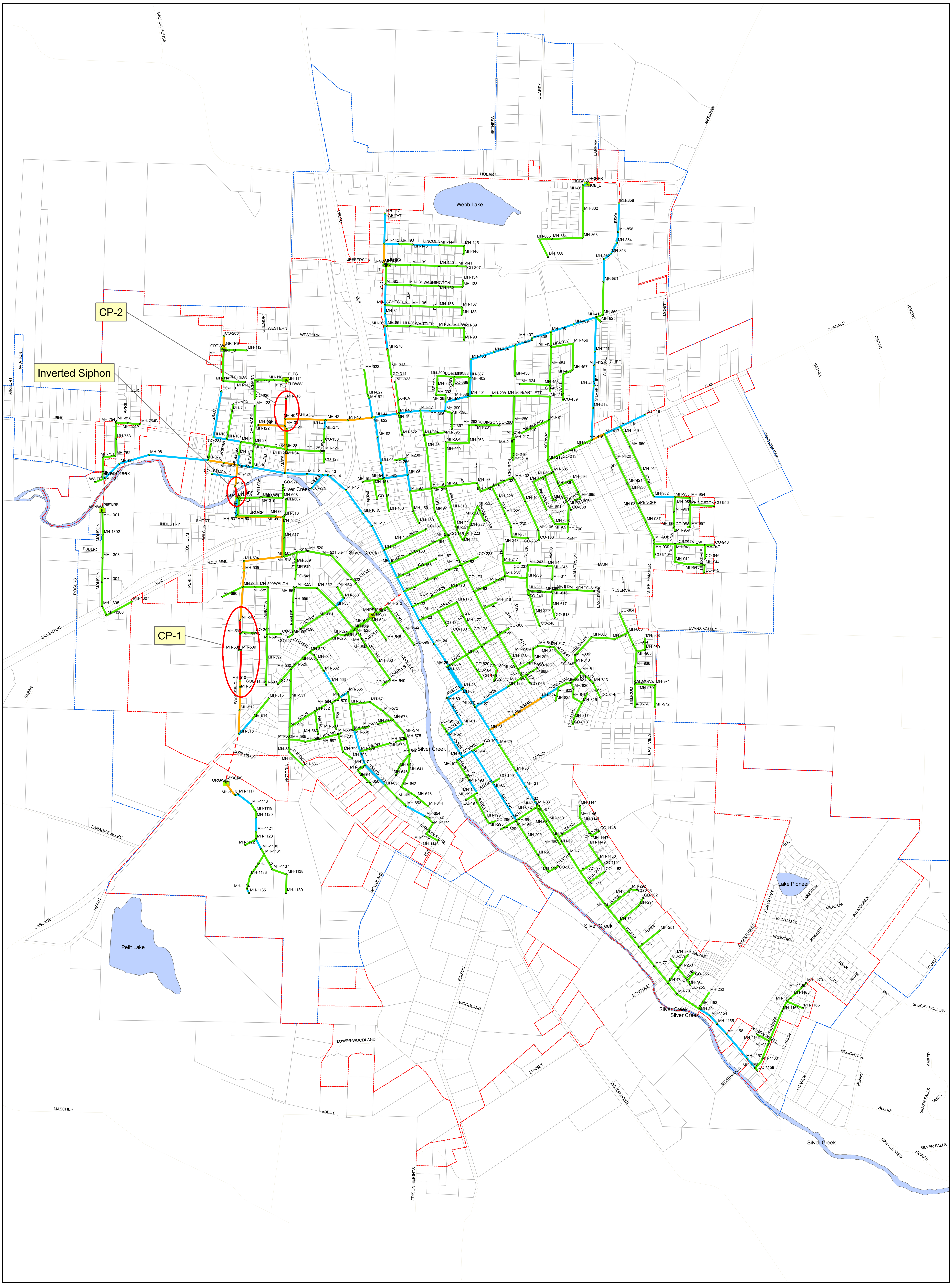
Pressure Main
- 4 inches
- 6 inches
- 8 inches
- 10 inches

Future Pipes
- 8 inches
- 10 inches
- 12 inches
- 18 inches

- Manhole
- Future Manhole
- Pump Station
- Future Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

Figure 8-3
Flow Monitoring and Point Loading Locations

HDR ONE COMPANY *Many Solutions*℠



Legend

- - Force Main

Utilized Pipe Capacity

- < 25%
- 25% to 50%
- 50% to 75%
- > 75%

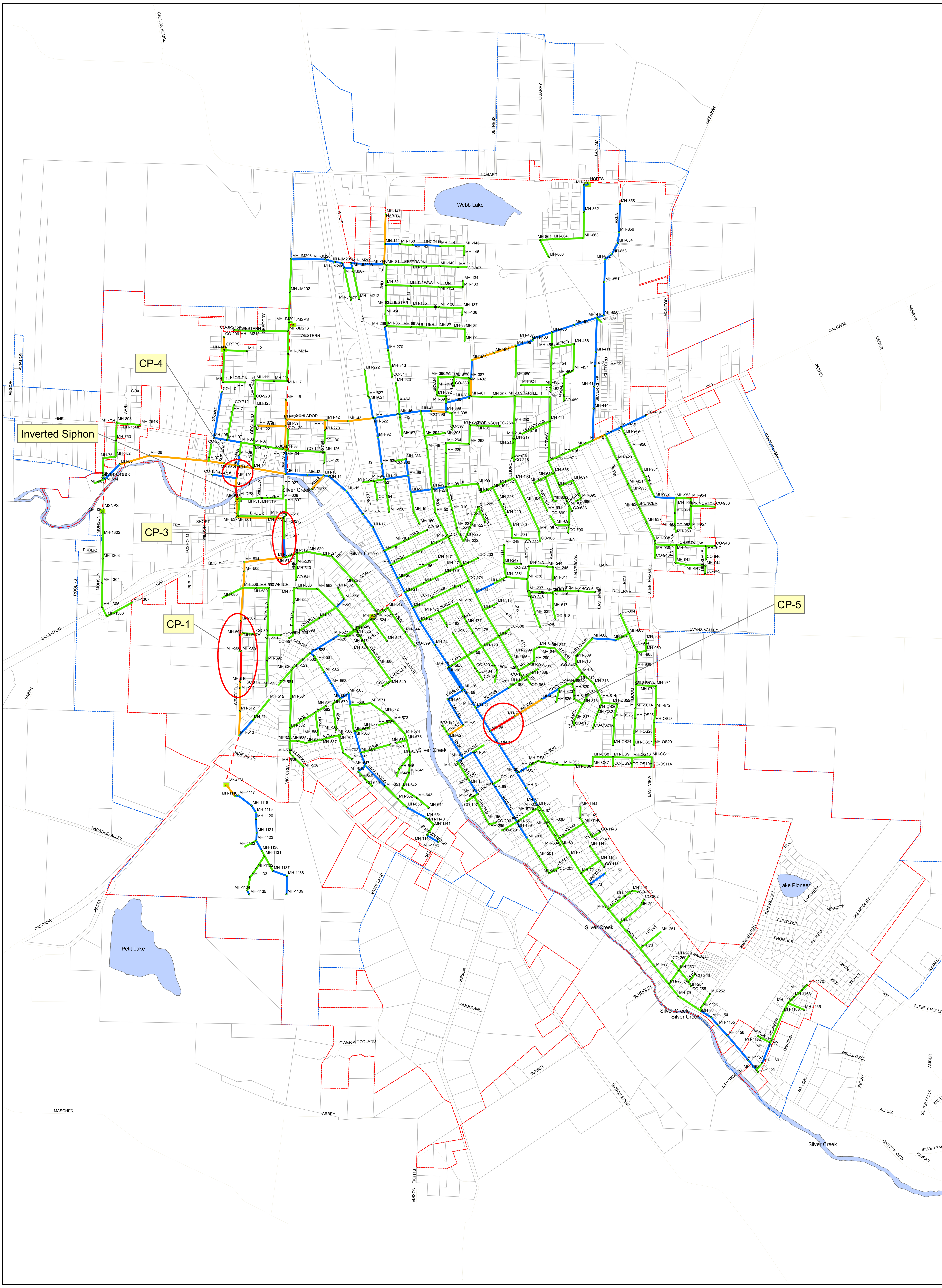
- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

N W E S

0 0.1 0.2 0.3 0.4 0.5 Miles

Figure 8-7
2006 System Capacity

HDR ONE COMPANY *Many Solutions*SM



Legend

- Force Main
- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

Utilized Pipe Capacity

- < 25%
- 25% to 50%
- 50% to 75%
- > 75%

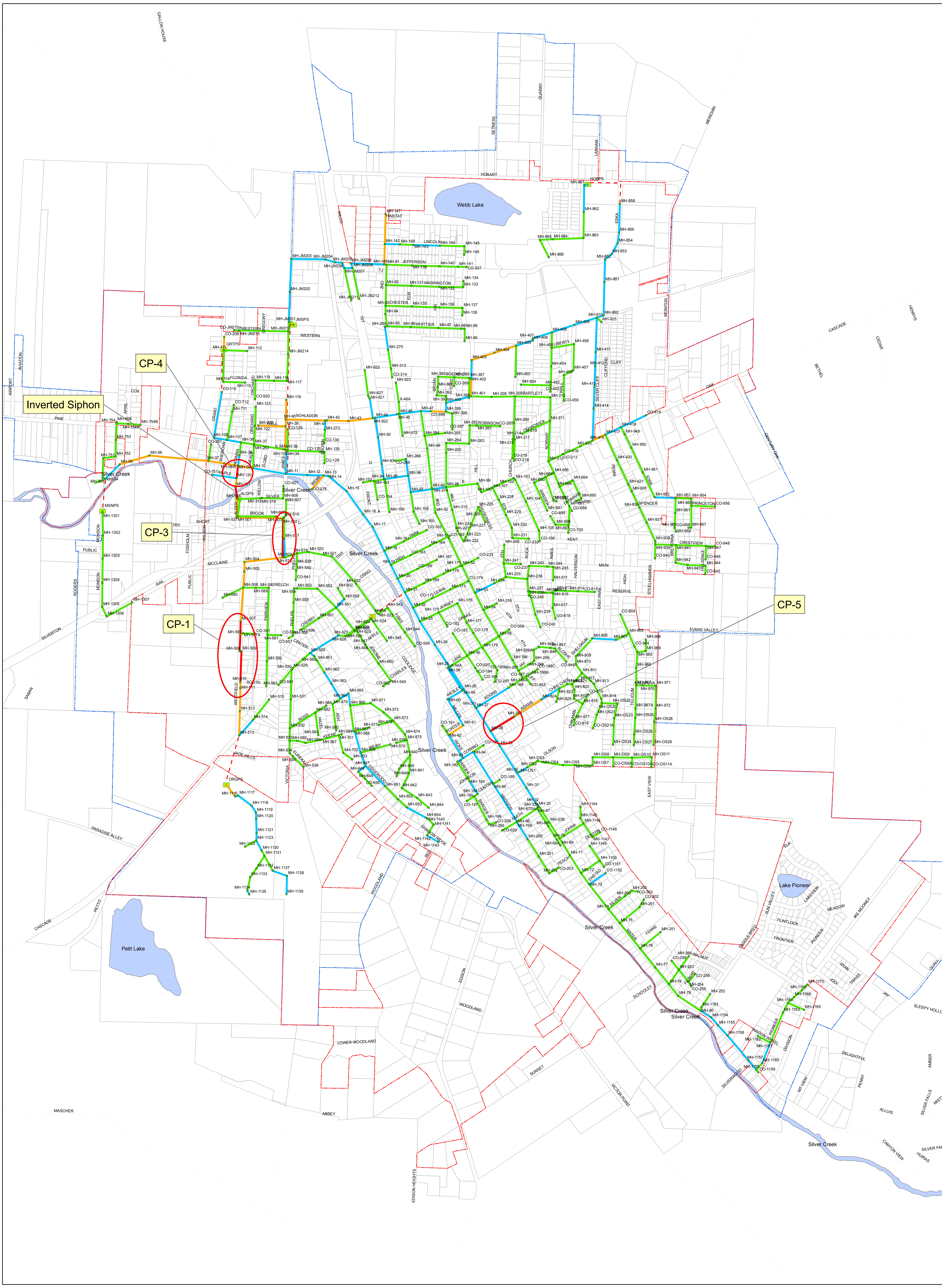
N E S W

0 0.1 0.2 0.3 0.4 0.5

Miles

Figure 8-8
2030 System Capacity
Industrial Reserve - Site A

HDR | ONE COMPANY *Many Solutions*℠



Legend

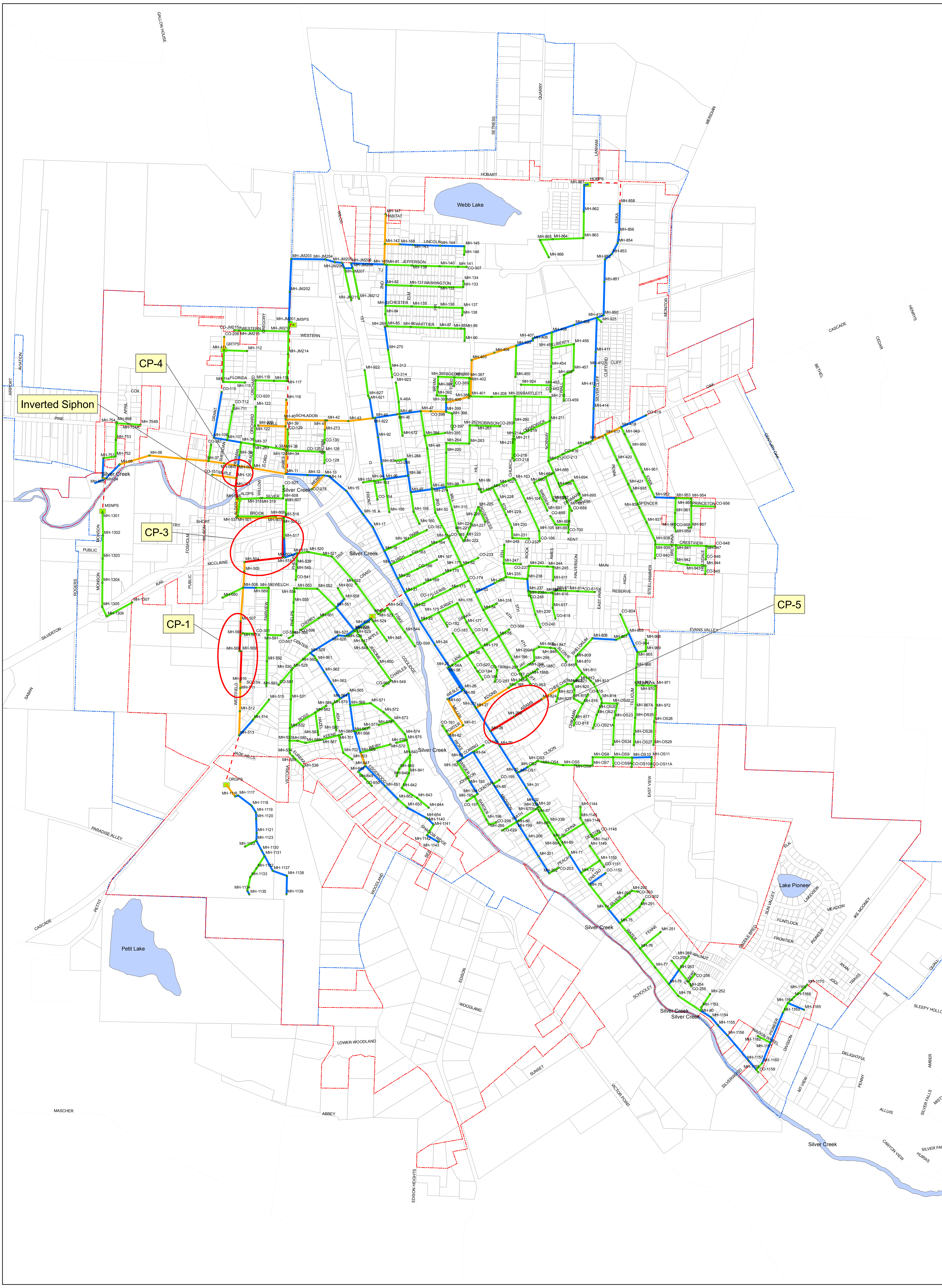
- Force Main
- Utilized Pipe Capacity
  - < 25%
  - 25% to 50%
  - 50% to 75%
  - > 75%
- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

N E S W

0 0.1 0.2 0.3 0.4 0.5
Miles

Figure 8-9
2030 System Capacity
Industrial Reserve - Site B

HDR | ONE COMPANY *Many Solutions*℠



Legend

- Force Main
- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

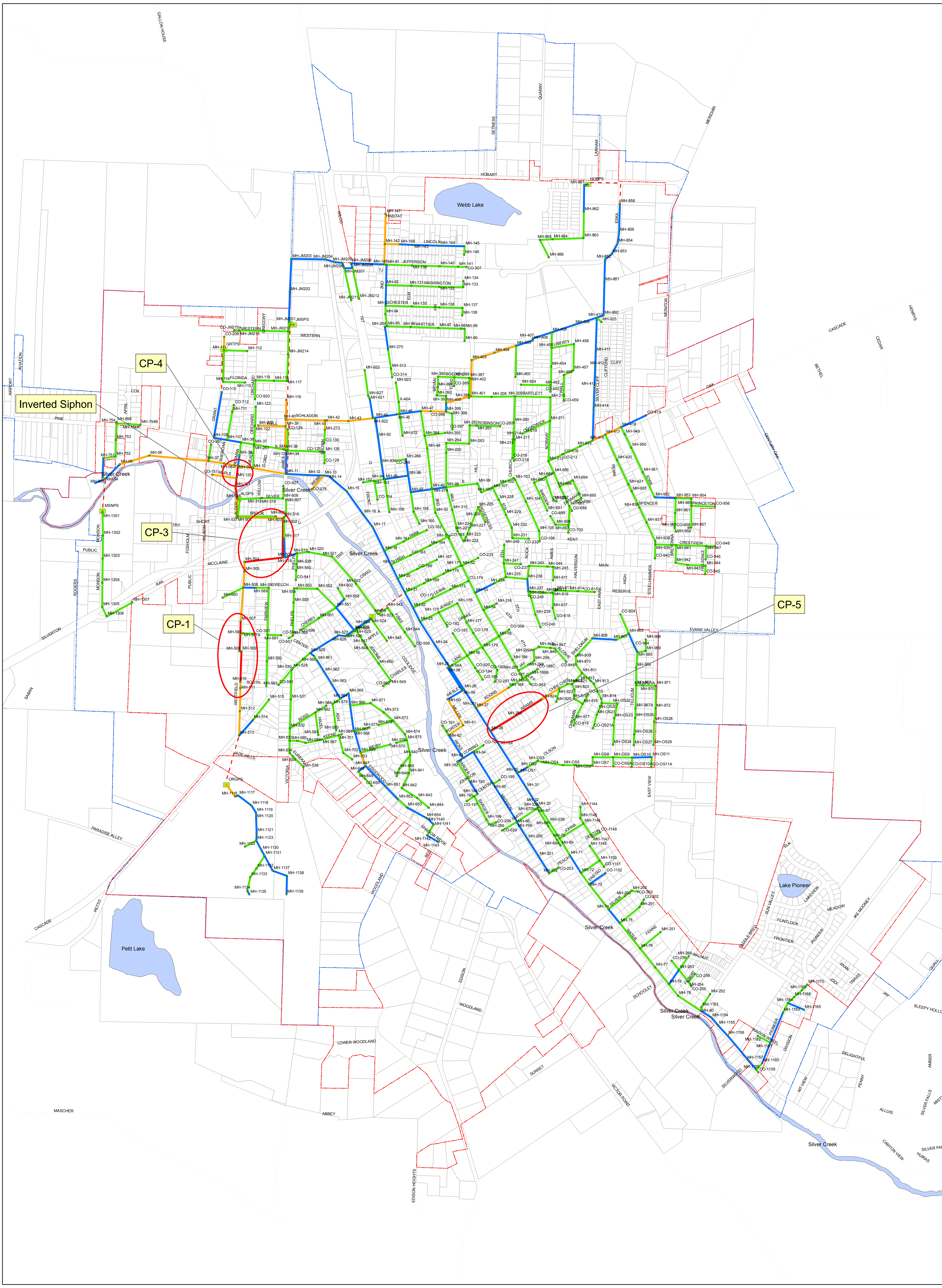
Utilized Pipe Capacity
- < 25%
- 25% to 50%
- 50% to 75%
- > 75%

N / W / E / S

0 0.1 0.2 0.3 0.4 0.5
Miles

Figure 8-10
Ultimate Build-Out Capacity
Industrial Reserve - Site A

HDR | ONE COMPANY *Many Solutions* SM



Legend

- Force Main
- Utilized Pipe Capacity
  - < 25%
  - 25% to 50%
  - 50% to 75%
  - > 75%
- Manhole
- Pump Station
- City Limits
- Urban Growth Boundary
- Water Bodies

N W E S

0 0.1 0.2 0.3 0.4 0.5
Miles

Figure 8-11
Ultimate Build-Out Capacity
Industrial Reserve - Site B

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