City of Sweet Home

Wastewater Facilities Plan

December 2016

Brown AND Caldwell

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Kate Brown, Governor

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December 29, 2016

Mr. Michael J. Adams, Public Works Director
City of Sweet Home
1140 12th Avenue
Sweet Home, Oregon 97386

RE: City of Sweet Home
File # 86840
Linn County
Sweet Home Wastewater Facilities Plan Approval

Dear Mr. Adams:

On December 12, 2016 we received the revised Sweet Home Wastewater Facilities Plan prepared by Brown and Caldwell. Last revisions were submitted December 22, 2016.

The revised document addresses all our comments as per our letter of August 16, 2016 and the Facilities Plan is approved. For the record, the document does not address environmental concerns required by potential State and Federal funding agencies.

The next step is to start pre-design work. To avoid extra work and cost overruns, the City should not authorize final design until a pre-design report is reviewed and agreed by City staff and DEQ.

Should the implementation of the proposed alternative in the facilities plan lapse over five years, we strongly recommend that you consult with DEQ staff to ensure that the proposed plan and issues are still relevant. It is sometimes possible that preparation of a new document may be warranted after five years.

Please feel free to call me at (541) 687-7341 should you have any questions or comments.

Sincerely,

Jaime Isaza

Jaime Isaza, Project Officer

ec: Jonathan Holland - Brown and Caldwell
Michael Kucinski, Tim Caire, Bob Dicksa, Tim McFetridge - DEQ

ji: Sweet Home FP 2016 app.docx

Wastewater Facilities Plan

Prepared for the
City of Sweet Home, Oregon
December 2016

Brown AND Caldwell

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Table of Contents

List of Figures

(indicates figure immediately follows page listed)*

List of Tables

List of Abbreviations

°C	degree(s) Celsius
°F	degree(s) Fahrenheit
AACEI	Association of Cost Engineering International
AAGR	annual average growth rate
ADWF	average dry weather flow
ATS	automatic transfer switch
AWWF	average wet weather flow
BASIN 4.1	Better Assessment Science Integrated Point and Nonpoint Sources
BC	Brown and Caldwell
BFP	belt filter press
BOD	biochemical oxygen demand
BLM	Biotic Ligand Model
Blue Book	Rental Rate Blue Book for Construction Equipment
CBOD	carbonaceous biochemical oxygen demand
CCT	chlorine contact tank
cfs	cubic foot/feet per second
City	City of Sweet Home
Cl_2	chlorine
CMU	concrete masonry unit
CT	contact tank
DEQ	Oregon Department of Environmental Quality
DMR	discharge monitoring report
DT	dry ton(s)
ENR	*Engineering News-Record*
EPA	U.S. Environmental Protection Agency
ft	foot/feet
ft^2	square foot/feet
ft^3	cubic foot/feet
FTE	full-time equivalent
FY	fiscal year
gal	gallon(s)
gpm	gallon(s) per minute
hp	horsepower
HRC	high-rate clarification
I/I	inflow and infiltration
in.	inch(es)
IPS	influent pump station
kW	kilowatt(s)
lb	pound(s)
LF	linear foot/feet
MAO	Mutual Agreement and Order
MBR	membrane bioreactor
MCA	Mechanical Contractors Association
MCL	maximum contaminant level
MG	million gallons
mgd	million gallons per day
mg/kg	milligram(s) per kilogram
mg/L	milligram(s) per liter
mL	milliliter(s)
MLSS	mixed liquor suspended solids
MMDWF	maximum-month dry weather flow
MMWWF	maximum-month wet weather flow
MOR	Monthly Operating Report
mph	mile(s) per hour
N/A	not applicable
ND	non-detectable
NECA	National Electrical Contractors Association
NH_3	ammonia
NPDES	National Pollutant Discharge Elimination System
NWEA	Northwest Environmental Advocates
O&M	operation and maintenance
OAR	Oregon Administrative Rules
OEA	Office of Economic Analysis
OWRD	Oregon Water Resources Department
PAA	peracetic acid
PDF	peak day flow

PIF	peak instantaneous flow
ppd	pound(s) per day
ppcd	pound(s) per capita per day
PREDATOR	PREDictive Assessment Tool for ORegon
psi	pound(s) per square inch
PWWF	peak wet weather flow
RAS	return activated sludge
RDII	rainfall-derived inflow and infiltration
RMZ	regulated mixing zone
R&R	rehabilitation and replacement
SCADA	supervisory control and data acquisition
scfm	standard cubic foot/feet per minute
SDC	system development charge
SRT	solids retention time
SSO	sanitary system overflow
TBD	to be determined
TDH	total dynamic head
TMDL	total maximum daily load
TSS	total suspended solids
UGB	urban growth boundary
USGS	U.S. Geological Survey
UV	ultraviolet
VFD	variable-frequency drive
WAS	waste activated sludge
WLA	waste load allocation
WPCF	water pollution control facility
WW	wet weather
WWTP	wastewater treatment plant
yd^3	cubic yard(s)
ZID	zone of immediate dilution

Executive Summary

This Executive Summary provides an overview of this Facility Plan, including the need for upgrades to the City of Sweet Home's (City's) wastewater system; alternatives considered; selected alternative; and cost, schedule, and revenue requirements for the recommended improvements.

Background and Purpose

The prior *Wastewater Facility Plan* (Brown and Caldwell [BC], 2002) evaluated inflow and infiltration (I/I) reduction, peak flow storage, and wastewater treatment plant (WWTP) expansion to address peak wet weather flows (PWWF) that were more than three times higher than the WWTP capacity of 7.0 million gallons per day (mgd). This capacity deficiency resulted in occasional sanitary system overflows (SSOs), in violation of the City's National Pollutant Discharge Elimination System (NPDES) Permit (Appendix A). The City elected to pursue I/I reduction, despite its higher price tag, to gain the added benefit of restoring the structural integrity of its sewer system.

Addressing this capacity deficiency was the objective of the 1999 Mutual Agreement and Order (MAO, Appendix B) with the Oregon Department of Environmental Quality (DEQ). The MAO remained in effect until May 2015 when DEQ notified the City that it was in compliance as a result of the City's successful, $15M sewer rehabilitation program between 2003 and 2012. The program resulted in a 50 percent reduction in the 5-year recurrence, peak-hour flow.

However, a follow-up *Inflow and Infiltration Update Report* (BC, 2013, Appendix C) estimates current 5-year recurrence, peak-hour flows at 12 mgd, and future flows up to 13 mgd. The WWTP capacity remains at 7 mgd. There were no major wet weather events between 2012 and November 2015, so no overflows occurred during that period. However, heavy rains in December 2015 culminated in an overflow on December 17. The collection system response to this event is estimated at approximately a 2-year recurrence. The overflow was deemed by DEQ in its February 18, 2016, letter (Appendix D) to the City as "beyond the reasonable control of the permittee," so no enforcement action was taken.

As expected, with each phase of sewer rehabilitation work, the cost per gallon of I/I removed increased. This trend is projected to continue if further I/I work were undertaken. Given these realities, along with aging equipment and facilities at the WWTP, this planning update focuses on WWTP improvements as the best path to compliance.

The purpose of this Facility Plan is to determine the recommended improvements, including phasing, costs, and revenue needs, to most effectively and affordably address system capacity, reliability, and performance concerns.

The same alternatives previously examined of additional I/I reduction, peak flow storage, and WWTP expansion through conventional and high-rate processes are re-evaluated with updated flow, performance, and cost information.

Alternatives Evaluation and Selection

Four alternatives were evaluated, including:

- Additional reduction of I/I in the collection system to limit wet weather flows
- Temporary storage of flows exceeding existing treatment capacity
- New membrane bioreactor (MBR) and new wet weather treatment process
- Improvements to existing treatment facilities and with new wet weather treatment process

General

The first two alternatives mitigate peak flows being conveyed to the WWTP during high flow events, but neither alternative on its own improves existing treatment infrastructure or provides additional treatment capacity required for existing and future conditions. Additionally, because these two options are not mutually exclusive, some level of repair and/or upgrades at the WWTP is required.

A new MBR-based secondary treatment process for flows up to 7 mgd in conjunction with a wet weather treatment process does offer advantages in terms of reducing the need for downstream filtration and achieves comparable effluent quality. However, the MBR-based alternative does not make the best use of the existing infrastructure, thus significantly increasing overall cost.

Improvements to the existing secondary treatment process coupled with a wet weather treatment installation does not provide as much new infrastructure; however, it makes the best use of the existing infrastructure. The use of existing infrastructure has the significant added benefit of providing phasing opportunities to increase affordability.

Cost Comparison for Liquid Stream Alternatives

Planning-level costs presented in Table ES-1 are an estimate of the cost to construct or modify each of the affected processes. The costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Biosolids handling improvement costs are also not included. Each construction cost estimate was developed using standard cost estimating procedures including layouts, equipment quotations, and unit costs based on a November 2015 *Engineering News-Record* (ENR) index. These construction costs are intended to provide a reference point for comparison for the possible alternatives.

The lowest-cost alternative is 3C, which improves the existing facility including the addition of a high-rate clarification (HRC) process. Total project costs for Alternative 3C are presented in Table ES-3.

Table ES-1. Comparative Construction Cost Summary for Liquid Stream Alternatives[a]

Alternative	Description	Collection System Costs, ($)	WWTP Costs to Treat 7 mgd, ($)	WWTP Added Costs to Treat Peak Flows up to 13 mgd, ($)	Total
1	Further reduce I/I	28M	10M	0	38M
2	Flow equalization/storage	28M	10M	0	38M
3a	Parallel secondary process	0	N/A	N/A	N/A
3b	MBR with HRC	0	20.1M	6.7M	27M
3c	Upgrade existing and add HRC	0	12.2M	6.7M	19M

a. *Costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Biosolids improvements are also not included here.*

Phasing

The alternatives were evaluated on their ability to incorporate phasing into their construction sequencing. Phasing over time is critical to affordability given the City's current revenue and debt service. Phasing potential for each alternative is assessed below:

- I/I reduction can be phased basin by basin.
- Temporary storage cannot be phased.
- The MBR-based alternative has limited phasing opportunity; the first phase would be a comprehensive WWTP upgrade with large capital costs.
- Upgrading the existing system allows for multiple phasing options.

Operations and Maintenance

The operations and maintenance (O&M) costs for the proposed alternatives are generally based on the existing O&M costs (baseline), plus or minus for the new or excluded components:

- I/I reduction and storage alternatives generally do not increase the baseline O&M costs. Additional oversight and pumping energy costs are assumed for the storage of peak flows.
- O&M costs for the MBR-based alternative were not developed in detail, as there would need to be a large reduction in O&M costs to demonstrate long-term financial benefits of this option. On the contrary, MBR systems are energy-intensive and would certainly add costs to the baseline.
- The upgrade-based alternative would be very similar to baseline O&M costs with the addition of infrequently used energy and chemical costs for the new HRC.

Seismic Considerations

Based on the age of the existing WWTP structures, and past history of similar structures of this type, the WWTP would likely suffer varying levels of failure during a future significant seismic event. The recent, much-publicized research on the Cascadia Subduction Zone identifies the risk of a major earthquake as higher than previously understood. Seismic resilience of each alternative is briefly assessed below:

- I/I reduction and storage alternatives do not improve the seismic reliability of the WWTP at any phase of their implementation.
- The MBR-based alternative allows for a comprehensive upgrade of the WWTP, thus allowing the WWTP to be designed to new reliability standards.
- The upgrade-based alternative allows for seismic components to be added to the WWTP for future treatment during/after a seismic event. The addition of new seismic rated influent pump station (IPS), HRC, and disinfection upgrades will allow for significant treatment while repairs/replacement of damaged structures are completed.

During predesign, a seismic assessment of the existing WWTP structures should be conducted to identify weaknesses and how they can be addressed and to plan for future, incremental upgrades to provide enhanced resiliency.

Selected Liquid Stream Alternative

The recommended alternative is 3C, which upgrades the existing secondary treatment system and constructs a parallel wet weather treatment system using HRC. It is the lowest-cost, most affordable option for the City because of its use of existing WWTP facilities and opportunities for phasing. O&M cost increases beyond present costs are minimal, and the HRC system will provide the most effective treatment for flows beyond 7 mgd. This alternative also allows phased-in seismic enhancements as new facilities can be designed to current standards.

Permit Compliance

The WWTP is generally able to meet permit limits today and will perform even better as a result of the proposed process improvements and implementation of a HRC process for wet weather flows beyond 7 mgd. However, mass limits for total suspended solids (TSS) have occasionally been exceeded during wet weather and will continue to be in the future without adjustment. Mass limits for carbonaceous biochemical oxygen demand (CBOD) may similarly be exceeded during wet weather. This is because as flows increase during wet weather, the current permit mass limits effectively require substantially lower effluent concentration limits for TSS and CBOD. This problem is even worse during May, the wettest month of the dry season, when mass limits are lower, driving the required effluent concentration limits down to 5 milligrams per liter (mg/L) for both TSS and CBOD.

The City and DEQ will have an opportunity to modify these restrictive mass load effluent limits during the next NPDES permit renewal process by following the process prescribed in Oregon Administrative Rules (OAR) 340-041-0004(9) for exceptions to DEQ's Antidegradation Policy. This process may require additional studies beyond the scope of this Facility Plan.

Recommended Improvements

Improvements associated with the recommended alternative are a new IPS and preliminary treatment improvements, secondary treatment improvements, a parallel system for conveying and treating flows greater than 7 mgd, disinfection improvements, hydraulic improvements including a new outfall pipe and diffuser, and improvements to solids handling facilities. The improvements composing the recommended alternative and their functions are listed in Table ES-2 and shown in Figure ES-1.

Table ES-2. Recommended Alternative Elements

Improvement	Function
New IPS	Increase pumping capacity to convey peak flows
Mechanical bar screening facilities	Remove rags and debris and prolong equipment life
Grit removal facilities	Reduce maintenance and prolong equipment life
Third aeration basin and improvements for existing basins	Increase secondary treatment capacity
Clarifier upgrades	Improve secondary effluent quality
New tertiary filters	Improve final effluent quality
Disinfection improvements	Treat higher flows
Outfall upgrades	Increase peak conveyance capacity
Biosolids storage improvements	Reduce odors, improve performance, and reduce maintenance
Biosolids dewatering improvements	Reduce hauling and solids disposal costs

Capital Costs and Phasing

The project improvements were prioritized and three phasing options (A, B, and C) were prepared for evaluation. Option A assumes all elements of the recommended alternative are built as a single project with construction midpoint of 2020. Option B is a three-phase approach with construction midpoints between 2020 and 2030. And Option C is a four-phase plan that makes compliance-related upgrades in the first two phases, with midpoints of 2018 and 2025, and other improvements in two later phases dependent on O&M needs, growth, and future permit limits with midpoints of 2035 and 2045.

Alternative 3C

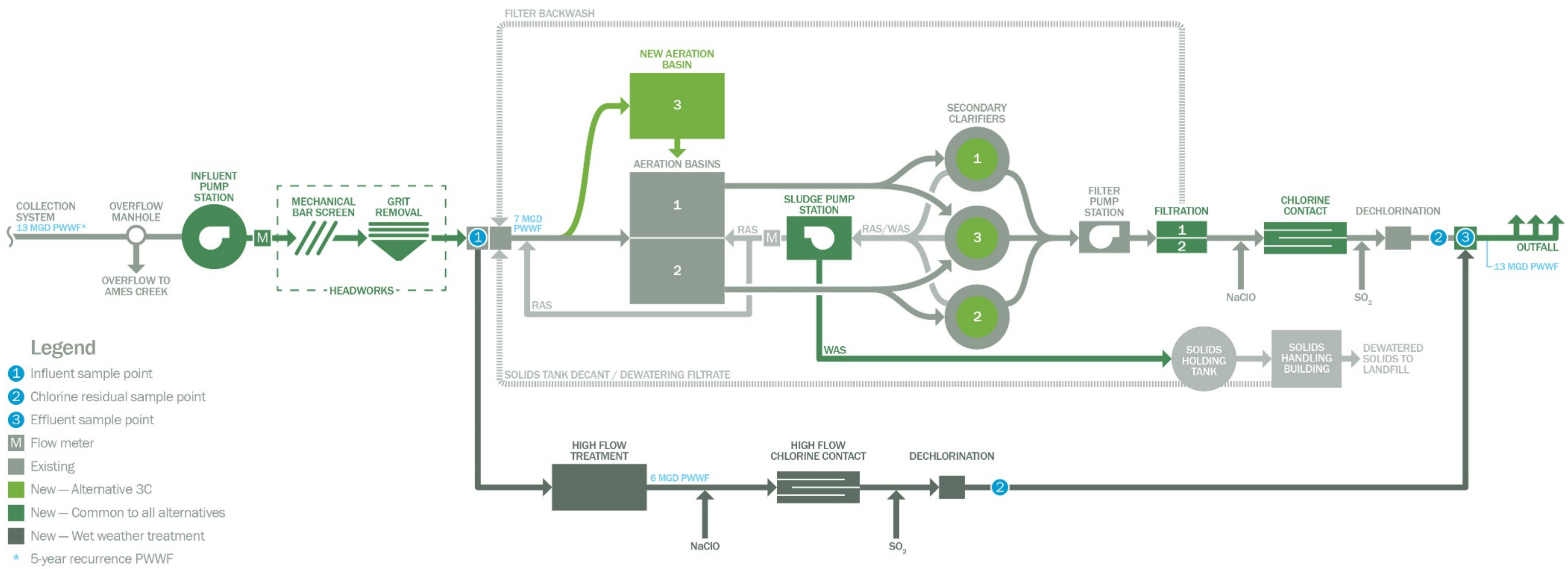

Figure ES-1. Recommended alternative process schematic

Phasing Option C was selected as it was based on considerations of affordability and project element need and benefit. The four project phases and capital costs (including allowances for engineering, administration, and construction management as well as escalation to the midpoint of construction) for each phase are shown in Table ES-3.

Table ES-3. Estimated Capital Cost and Timing for Option C

Item	Description [a]	Cost ($)
	Phase 1: midpoint of construction, July 2018	
ST-1	Aeration improvements for existing basins	626,000
OI-1	Outfall improvements	362,000
BS-1	Biosolids handling	1,187,000
ST-2	Secondary clarifier improvements	447,000
M-1	Miscellaneous improvements	479,000
	Subtotal project cost including escalation [b]	**$4,200,000**
	Phase 2: midpoint of construction, July 2025	
IP-1	Influent pumping	3,294,000
PT-1	Mechanical bar screen facility (one screen)	492,000
PT-3	Flow diversion pipe and structure	113,000
WWT-1	Wet weather treatment (HRC)	3,520,000
D-1	Existing CCT and disinfection improvements	99,000
D-2	Wet weather disinfection facility	408,000
SG-1	Standby generator	260,000
CS-1	Civil site work	265,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**$14,200,000**
	Phase 3: midpoint of construction, July 2035	
IP-2	Influent pumping capacity expansion	430,000
ST-3	New aeration basin	2,229,000
PT-2	Additional mechanical bar screen	285,000
CS-1	Civil site work	95,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**$6,900,000**
	Phase 4: midpoint of construction, July 2045	
TT-1	Tertiary filtration	3,333,000
PT-4	Grit removal	1,994,000
CS-1	Civil site work	172,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**16,700,000**
	Total project cost for Option C	**$42,000,000**

a. *Individual line item costs are presented as the construction costs.*

b. *The sub total and total costs include 15% for engineering and administration, 10% for construction management, and 3% annual escalation to the midpoint of construction.*

Figure ES-2 graphically shows the phasing plan superimposed on the WWTP site plan.

Figure ES-2. Prioritized, phased improvements result in affordable program, regulatory compliance

Revenue Requirements

Figure ES-3 shows the monthly revenue required from an average single-family residence for a period of 30 years for Phasing Option C.

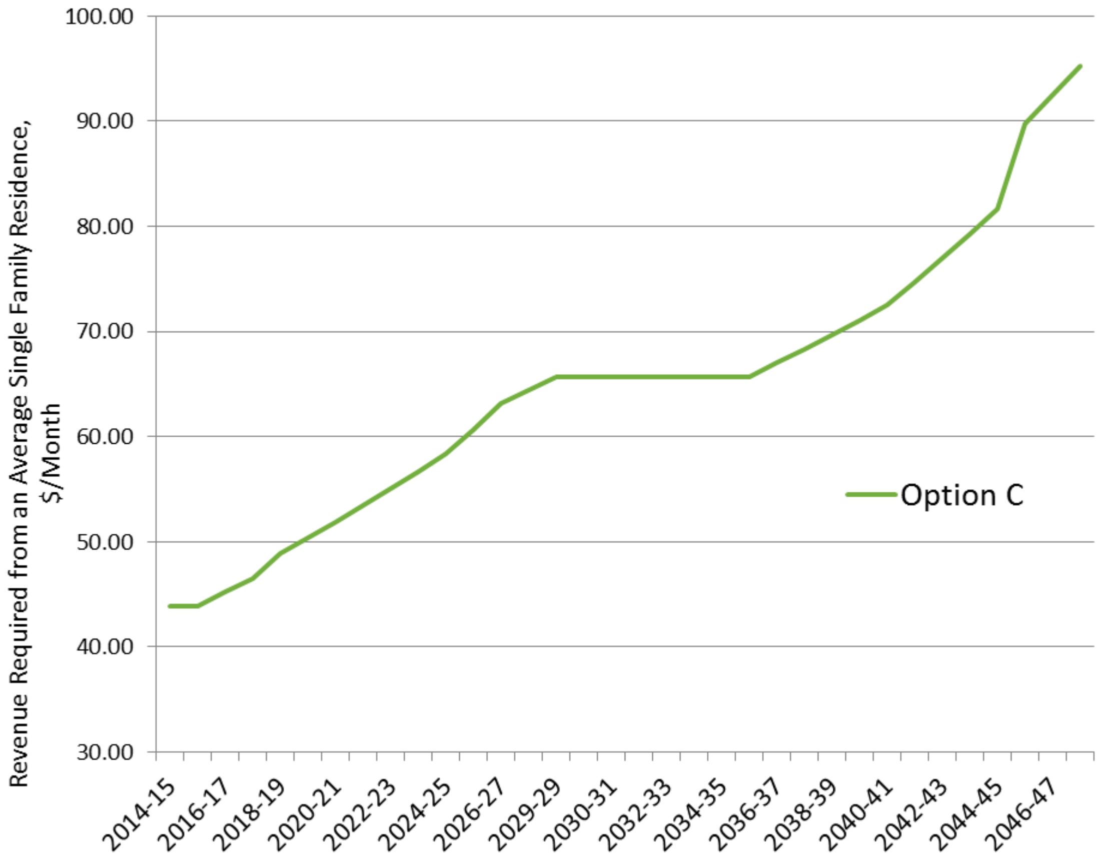

Figure ES-3. Monthly revenue required from an average single-family dwelling

The revenue estimates start at the 2015 estimated average monthly user fee of approximately $44 per month. Revenue requirements increase each year based on the escalation of planned costs plus applicable inflation rates. Steep increases shown are due to the debt service required to support borrowing for capital improvements.

Section 1

Introduction

This section provides summary information related to the background and purpose of this Facility Plan.

1.1 Background

The City of Sweet Home's (City's) wastewater collection and treatment systems serve essentially the entire population of its 9,170 residents within the 6.5-square-mile city limits and urban growth boundary (UGB). The wastewater treatment plant (WWTP), located at 1357 Pleasant Valley Road, has been in service since 1947. Major upgrades were completed in 1974 and 1994. Treated effluent is discharged to the South Santiam River at river mile 31.5 under National Pollutant Discharge Elimination System (NPDES) Permit 101657, included as Appendix A.

The most recent planning efforts preceding this report produced two documents:

- *Wastewater Facility Plan* (Brown and Caldwell [BC], 2002)
- *Inflow and Infiltration Update Report* (BC, 2013)

The 2002 Facility Plan accurately quantified the wet weather capacity deficiency that was the topic of a 1999 Mutual Agreement and Order (MAO), included as Appendix B, with the Oregon Department of Environmental Quality (DEQ). The peak hourly flow in a 5-year recurrence event at that time was estimated at 22 million gallons per day (mgd) compared to the plant's capacity of 7 mgd. By 2027, peak flows were expected to reach 25 mgd.

The 2002 Facility Plan evaluated alternatives of inflow and infiltration (I/I) reduction, peak flow storage, WWTP expansion using conventional technology, and WWTP expansion using high-rate clarification (HRC) technology. Though it was the most expensive option, the City elected to pursue I/I reduction as it had the dual benefit of also restoring structural integrity to deteriorated portions of the collection system.

The 2013 *Inflow and Infiltration Update Report,* included as Appendix C, documents the City's investment of more than $15M, from 2003 through 2012, in four separate phases of sewer rehabilitation as well as the resultant 50 percent reduction in 5-year recurrence, peak-hour flows. However, as expected, with each phase of sewer work, the return on investment decreased in terms of cost per gallon of I/I removed.

The MAO remained in effect until May 2015 when DEQ notified the City that it was in compliance as a result of the City's successful, $15M sewer rehabilitation program between 2003 and 2012.

1.2 Purpose

The diminishing returns of further major sewer rehabilitation projects, combined with aging equipment/facilities at the WWTP, led to this planning effort. The same alternatives, previously examined, of additional I/I reduction, peak flow storage, and WWTP expansion through conventional and high-rate processes, are reevaluated with updated flow, performance, and cost information. The purpose of this Facility Plan is to determine the recommended improvements, including phasing, costs, and rate impacts, to most effectively and economically address remaining system capacity deficiencies as well as reliability and performance concerns.

This document is generally organized as recommended in *Preparing Wastewater Planning Documents and Environmental Reports* (DEQ, 2013) to meet the requirements of various potential funding agencies.

Section 2

Study Area

Developing a long-range project plan requires consideration of natural and socioeconomic factors. Environmental characteristics such as topography, geology, soils, climate, and water resources affect the design and operation of wastewater conveyance systems. Socioeconomic factors such as land use and population projections affect wastewater system capacity. In this section, the city of Sweet Home and its characteristics are examined.

2.1 Location

Sweet Home is located along the west slope of the Cascade Mountains at the edge of the Willamette Valley, and is bordered to the north by the South Santiam River just below the Foster Reservoir. The study area for the Sweet Home Facilities Plan encompasses approximately 6.5 square miles within the UGB, which is parallel with the city limits. An aerial view of the city with the UGB delineated is shown in Figure 2-1.

2.2 Socioeconomic Environment

The U.S. Census Bureau reports socioeconomic data for each city within the nation. The data are presented on a 5-year basis from 2008 to 2012. Information on local industries, employment, median household income level, vulnerable populations, and poverty levels can influence future capital planning decisions. Pertinent census data are listed in Table 2-1.

Table 2-1. Sweet Home Census Bureau Data

Parameter	Year	Data
Total number of business firms	2007	378
Top three employment fields: • Education service • Health care • Social assistance	2008–12	21%
Retail trade	2008–12	19%
Manufacturing	2008–12	17%
Unemployment rate (percent in the labor force)	2008–12	9%
Median household income	2008–12	$36,200
Persons below poverty level	2008–12	22%

Source: http://factfinder2.census.gov/faces/nav/jsf/pages/community_facts.xhtml

While various unemployment percentages for different categories are presented in the American FactFinder U.S. Census Bureau data, the overall unemployment percentage of Sweet Home is reported at 9.4 percent. This closely matches more recent data provided for Sweet Home on the City-data website.

2.3 Environmental Resources Present

The physical environment includes the topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items as they relate to the sewerage planning program.

2.3.1 Topography, Geology, and Soils

The topography, geology, and soils of a region can have a significant effect on the design and construction requirements of sewage works. Topography can determine the route and slope of sewer lines as well as the need for and location of pump stations. The geology and soil conditions in an area can affect construction costs for pipelines and treatment units as well as I/I through joints and cracks in the sewer system.

Topography. Sweet Home is in the mid-Willamette Valley area with the majority of the development built on relatively flat terrain. Terrain slope varies from 2 percent at the river to 10 percent at the UGB. These slopes allow the entire sewer system to flow by gravity to the WWTP. City elevations vary from 510 feet above sea level at the WWTP to 1,280 feet on the hilltops at the southern edge of town. The topography and city limits are shown in Figure 2-2.

Geology and Soils. The Sweet Home area is underlain by predominantly volcanic rock dating back as far as the Eocene-Paleocene epoch. Its formations remain as a testament to some of the earliest and most extensive lava flows experienced in Oregon. Although volcanic rock is the dominant outcropping throughout vast areas in the western Cascade Range, it is overlain by more recent deposits in the Sweet Home area.[1]

The most recent deposits in the Sweet Home area are alluvium deposits along the banks of the South Santiam River, Ames Creek, and Wiley Creek from the Holocene Epoch that consist of well-drained, sandy to silty clay loams. These deposits overlay more ancient alluvial deposits that consist primarily of poorly drained gravelly silt loams and gravelly silty clay loams. Because the deposits are compressible, precautions must be taken in foundation design for heavy structures. Additional geotechnical studies are required to determine the extent of compressibility and resulting foundation requirements. Unconsolidated sand is particularly unstable during earthquakes; liquefaction and major additional settling can occur.[2]

2.3.2 Climate

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of sewerage facilities. Climate conditions directly influence the feasibility of reusing treated wastewater effluent for irrigation. Rainfall is especially significant because it can directly or indirectly cause flows with high peaking factors in sewage collection systems. For example, stormwater runoff may directly enter the sewers at manholes or through illicitly connected roof drains, area drains, and foundation drains. Accumulated rainfall may raise groundwater levels in many areas, leading to infiltration through cracks and poor-quality joints in the sewer system.

Other climatic factors can also affect wastewater processes. Biological treatment processes depend on air and water temperature. Temperature, cloud cover, and the rate of evaporation are important factors to be considered in design of sludge drying beds, composting facilities, and sludge lagoons.

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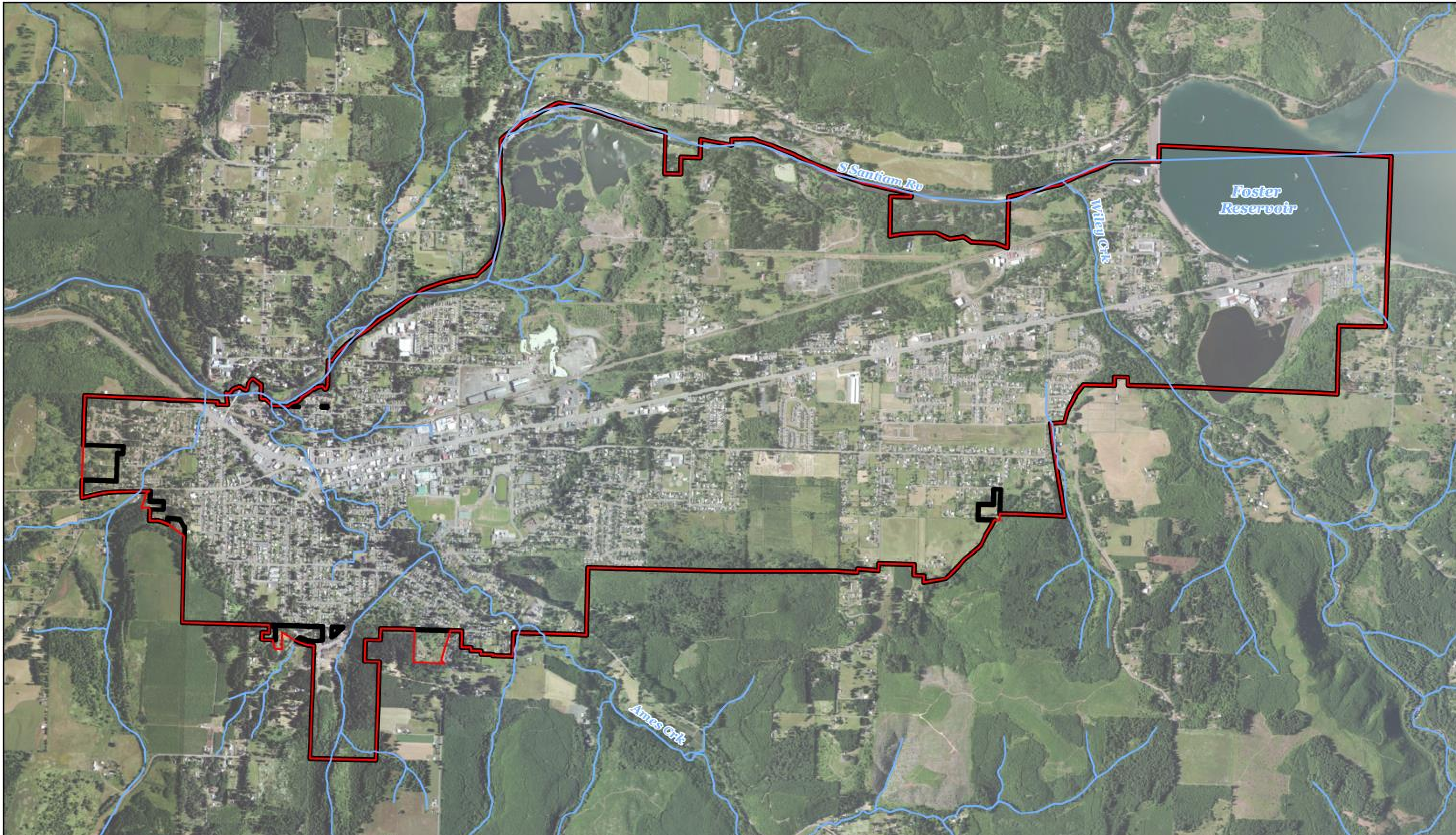
[1] Allan, Stuart, A.R. Buckley, J. E. Meacham. 2001. *Atlas of Oregon*, Second Edition. University of Oregon Press.

[2] U.S. Department of Agriculture. 1987. *Soil Conservation Service. Soil Survey of Linn County Area, Oregon.* July.



Rivers and Streams
Urban Growth Boundary
City Limits

0 0.25 0.5 1 Miles

Figure 2-1.
UGB and environmental resource map

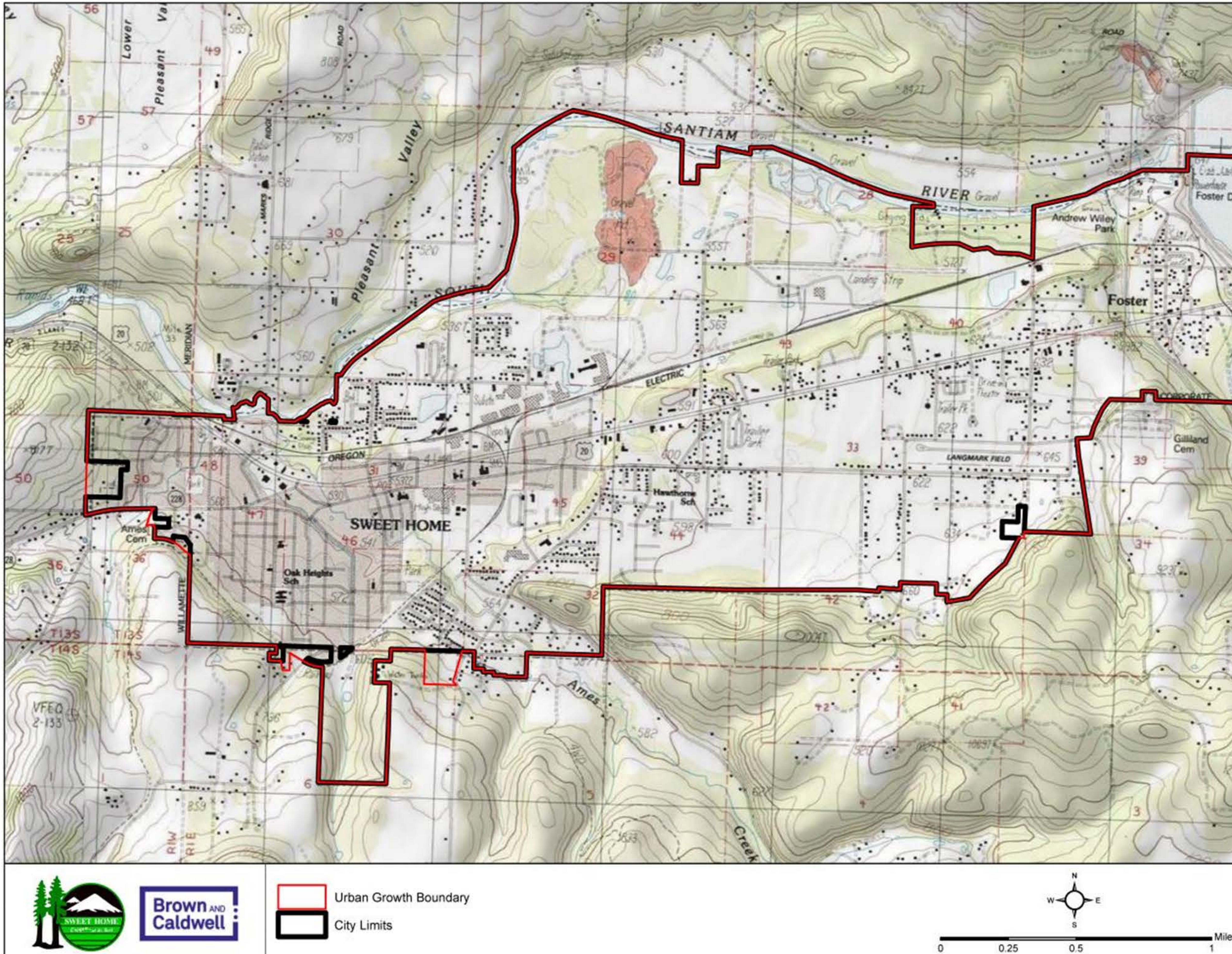


Figure 2-2. Topographical map

General Climatic Conditions. Sweet Home generally has a moderate climate, characterized by warm, dry summers and cool winters with abundant rain and some snow. There are brief periods in the summer when temperatures exceed 80 degrees Fahrenheit (°F), as well as brief periods in the winter when temperatures drop below freezing. The coastal mountain range generally breaks the prevailing Pacific storm fronts, causing the mid-Willamette region to receive roughly half of the annual rainfall experienced in the coastal areas. All climate data reported are from the Western Regional Climate Center. The Foster Dam weather station provides data for the region near Sweet Home. Current reported data span 1969 to 2012.

Precipitation. The average annual precipitation at Sweet Home is 54.41 inches. Essentially all of the precipitation is in the form of rain; snow rarely exceeds minor flurries. The average annual snowfall is 1.2 inches. About 65 percent of the rainfall occurs from November through March.

Temperature. The mean and extreme temperatures recorded at Foster Dam, located at the eastern end of Sweet Home, are summarized in Table 2-2. Maximum and minimum temperatures are fairly mild, although winter temperatures occasionally fall well below freezing. Freezing temperatures have been experienced from September through May. Although subfreezing temperatures may persist long enough to freeze water in aboveground facilities, they do not last long enough to be of concern for buried facilities. The highest summer temperature recorded during the period of record was 106°F.

Table 2-2. Sweet Home Area Temperature Summary

Month	Maximum temperature (°F)	Minimum temperature (°F)	Average temperature (°F)
January	67	0	40.8
February	71	2	43.4
March	79	22	46.4
April	85	22	49.8
May	96	28	55.0
June	102	35	60.2
July	106	39	65.7
August	105	36	65.6
September	102	32	61.1
October	93	20	53.1
November	75	16	45.5
December	69	0	40.7
Annual	106	0	52.3

Note: *Temperature data from Foster Dam, 1969–2012. Western Regional Climate Center http://www.wrcc.dri.edu/cgi-bin/cliMAIN.pl?or3047*

Other Climatic Factors. Wind speed and direction are not measured and recorded for the Sweet Home area. The nearest location with wind data is Salem. At that location, the prevailing wind in the winter is southerly at 7.7 miles per hour (mph). The summertime prevailing wind is northwesterly at 6.4 mph. Discussions with City WWTP staff indicate that the winds are similar in Sweet Home to those in Salem. The wind patterns blow odors from the WWTP over the South Santiam River during the winter and over the town center during the summer.

Evaporation data for the area are not available. If lagoons and effluent irrigation facilities are ever proposed, site-specific, and crop-specific data would be needed. If sludge drying beds or sludge lagoons are proposed in the future, pan evaporation data should be collected.

2.4 Land Use

The city and surrounding unincorporated rural area, locally known as the Sweet Home Valley, encompass approximately 18 square miles (11,520 acres). Land use and development is governed largely by the local topography. The city is bordered by hills, resulting in the town layout occurring in an east–west orientation.

Approximately 15 percent of the vacant land within the city is unsuitable for development and the majority of the undeveloped land has been designated for urban residential development. The commercial district extends along U.S. Highway 20 and is concentrated in downtown Sweet Home between 18th and 9th avenues. This commercial district is bordered by high- and medium-density residential areas. Industrial land uses are concentrated along the highways through the center of town, but outside of the commercial areas.[3]

A map dated 2003 for the August 2010 Comprehensive Plan outlining the land uses is included in Appendix E.

2.5 Community Engagement

Two public meetings will be facilitated during the facility planning process. One will occur early in the project to share information on the needs for the project and the types of alternatives that will be evaluated. A tour of the facility will be provided to interested parties. The second public meeting will occur toward the end of the project to share the results and request public comment on the draft facility plan.

3 City of Sweet Home. 2003. *City of Sweet Home Comprehensive Plan.* September.

Section 3

Basis of Planning

This section identifies the parameters that will serve as the basis of planning for the recommended wastewater treatment facilities. Existing wastewater characteristics are evaluated and discussed, future wastewater characteristics are projected, and assumptions used for planning-level cost estimating are defined.

3.1 Regulatory Authority

Standards for protection of water quality are set by DEQ through Chapter 340 of the Oregon Administrative Rules (OAR), subject to the approval of the U.S. Environmental Protection Agency (EPA). The general policy followed in these rules is one of anti-degradation of surface waters. Discharges from WWTPs are regulated through the NPDES. The criteria in the NPDES permit are based on protecting designated beneficial uses of the receiving waters, existing water quality in the receiving stream, water quality standards, anti-degradation policy, and minimum design standards for waste treatment.

3.2 Planning Period

In accordance with DEQ guidelines, the planning period for the Sweet Home Facilities Plan is 20 years. The anticipated population, wastewater flows and loads, and effluent limits at the end of this period are developed as the basis for planning the future facilities. The period begins in 2020 when the initially proposed project is placed into service. The corresponding 20-year planning period would therefore conclude in 2040.

3.3 Service Area Population and Reasonable Growth

Forecasts for populations within the Sweet Home service area were made by methodology approved by the Oregon Department of Administrative Service's Office of Economic Analysis (OEA). In 2014, the OEA adopted Division Rule 32 and the population-estimating methodology defined in OAR 660-032-0040 for counties that have not prepared a population forecast for at least 10 years. The last population forecast adopted by Linn County occurred in 1999.

3.3.1 Existing Population

In accordance with OAR 660-032-0040(7), Portland State University certified the 2014 population for the city at 9,060, and concluded that the city accounted for 7.57 percent of Linn County's total population. The 2015 OEA population estimate for Linn County is 121,142. The corresponding Sweet Home population for 2015 is therefore 9,170.

3.3.2 Population Trends

In accordance with OAR 660-032-0040(6), the annual average growth rate (AAGR) predicted for Linn County is 1.168 percent for the period 2015 to 2040. Application of this growth rate to the city 2015 population estimate results in the projections shown on Table 3-1.

Table 3-1. Existing and Projected Population Data		
Year	Total population	Increase from 2015
2015	9,170	0
2020	9,718	548
2025	10,299	1,129
2030	10,915	1,745
2035	11,567	2,397
2040	12,259	3,089

Historical population data for the city from 1990 to 2014 were obtained from the Portland State University Population Research Center. The data are shown graphically on Figure 3-1 along with the OEA population projection for the years covering 2015 to 2040.

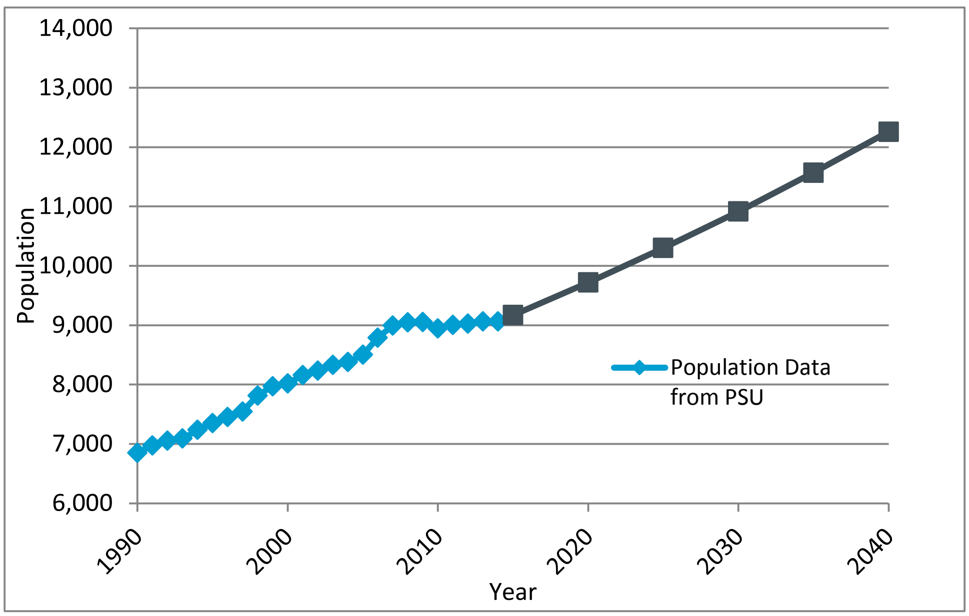


Figure 3-1. Sweet Home historical population with OEA projection

3.3.3 Reasonable Growth

Reasonable growth for purposes of this Facility Plan is defined as the projected growth in the service area over the 20-year planning period in accordance with OEA's Division Rule 32, and the population-estimating methodology defined in OAR 660-032-0040.

3.4 WWTP Flows

Base flows attributable to customers in the service area were determined from WWTP records.

In lieu of the statistical methodology described in the DEQ facility planning guidance document for development of peak flow projections, the estimate of current and projected flow (including peak flow) were developed using the comprehensive collection system model developed by BC as part of the City's recent I/I reduction projects. Reliable peak flow data are essential for adoption of the DEQ method but not available from the WWTP record because only flow that can be conveyed through the WWTP is metered. The modeling methodology used represents the best available and most accurate method for evaluating both existing flow and future flows as it takes into account the recently rehabilitated collection system's response to variable rainfall patterns under a range of antecedent conditions, and uses historical rainfall patterns to predict future rainfall patterns. The basic elements are described below.

The City's collection system was delineated into 20 sub-basins. Flow meters were installed at multiple locations within the collection system to measure each basin's response to rainfall. Using the flow meter data and concurrent rainfall data, the model was calibrated and I/I rates were assigned to each basin. Once calibrated, 40 years of historical rainfall data plus base flow attributable to sewer customers were input to the model and a long-term simulation run. The results were used to develop a statistical characterization of both dry weather and wet weather (WW) flow conveyed to the WWTP as even most dry weather flows are influenced by I/I to some degree. This methodology for service area flow characterization provides a useful tool for evaluating existing flows and for predicting future flows because it is based on the collection system's actual response to rainfall duration and intensity. This methodology also takes into account groundwater influence, and the results are based on historical rainfall patterns, which are the best predictors of future precipitation patterns.

The flow projections are based on the assumption that the existing I/I rates remain consistent over the planning period. If I/I rates increase during the planning period, the City will need to increase rehabilitation and replacement (R&R) of public main lines and service laterals on private property sufficient to bring I/I back within the capacity of the WWTP. The "no-increase" in I/I assumption over the planning period is based on this R&R work offsetting any increases associated with future degradation of laterals and sewer mains. Though at a much reduced level compared to the 2003-2012 R&R program, efforts must continue at a level that make this assumption valid. Additional information regarding the collection system I/I reduction and the flow modeling effort are provided in Appendix C.

Flow modeling results for existing and future conditions are presented in Table 3-2.

Table 3-2. 2040 Projected Flows

Flow (mgd)	2015	2040
ADWF	1.40	1.80
MMDWF	2.68	3.08
AWWF	2.49	2.99
MMWWF	4.03	4.57
PDF	9.86	11.29
PIF	11.66	13.28

DEQ staff were consulted and approved use of this alternate flow projection approach for Sweet Home.

3.5 Wastewater Loads

Six years of WWTP Monthly Operating Reports (MORs) were tabulated and reviewed to evaluate and characterize loading to the WWTP. Development of load projections is described in this section.

The influent loading data provided in the MORs show an inexplicable and sustained increase in biochemical oxygen demand (BOD) and total suspended solids (TSS) loading starting in May 2011. Sampling data compiled beyond May 2011 indicate that BOD loads essentially doubled while the TSS loads increased on average over 250 percent when compared to earlier data. The WWTP relies on an effluent flow meter, which is calibrated annually, as there is no influent flow meter presently. Figure 3-2 provides a graphical representation of the influent loading record.

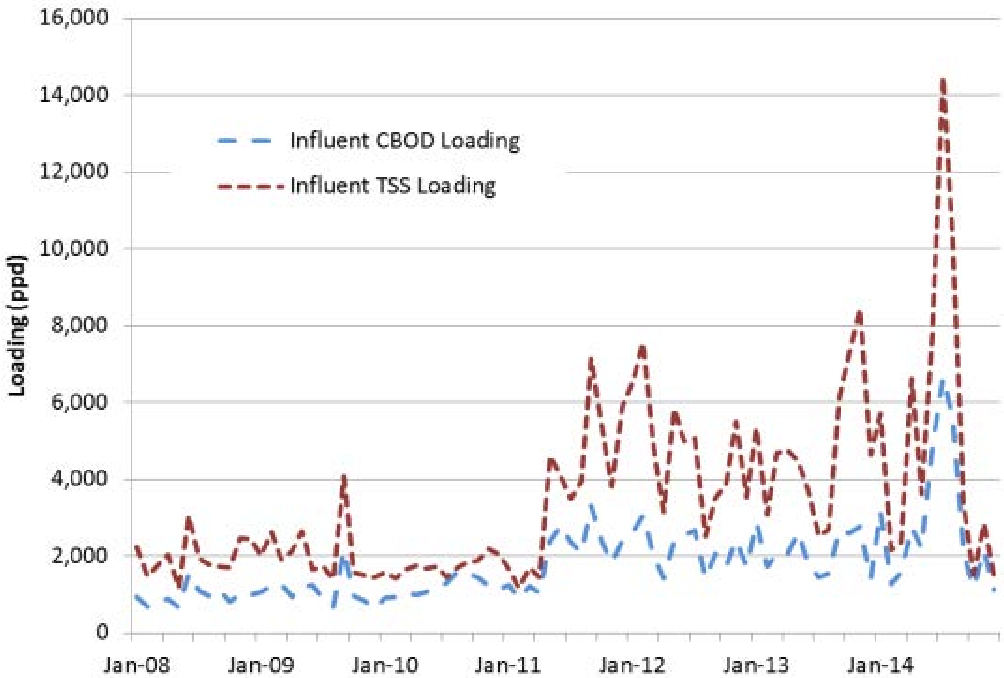


Figure 3-2. Influent loading record (ppd)

Examination of the solids hauling record for the same period indicates that annual solids production at the WWTP was consistent. The apparent variation between 2013 and 2014 is primarily due to maintenance activities causing an interruption in the normal dewatering and hauling schedule in 2013. Hauling was increased in 2014 as a result; the average for 2013/2014 was 203 dry tons (DT), which is consistent with the preceding years. Figure 3-3 shows the DT of solids hauled from 2009 to 2014.

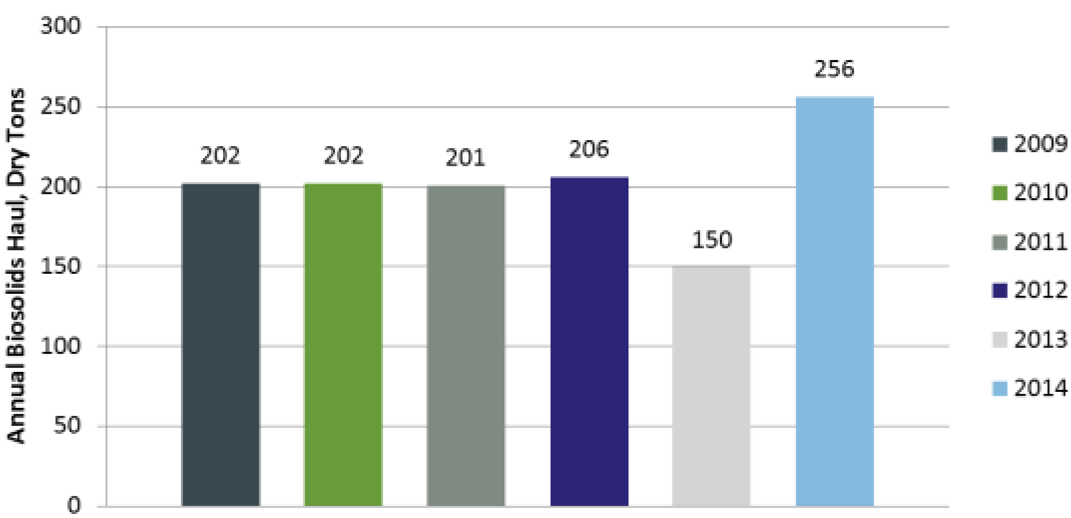


Figure 3-3. DT of biosolids hauled

The WWTP does not have an in-plant pump station so recycle streams including filter backwash, dewatering filtrate, and decant from the solids holding tank are recycled through the influent pump station (IPS) and, depending on timing of composite sampling, is potentially accounted for as additional influent BOD and TSS. However, resampling of process streams was practiced before the apparent increases in May 2008, so this does not account for the sudden and sustained increase in loads indicated by records.

No identifiable changes have occurred in the influent sampling methodology or in laboratory procedure used to analyze influent loadings during the period examined, nor have any identifiable changes occurred in the customer base or activities causing the apparent increase in loadings. It was concluded that the annual consistency in solids production indicates that influent loading to the WWTP was also consistent over this period despite the apparent increase indicated by the records.

Considering the large variation in the influent sampling record, the solids hauling data were used to estimate representative BOD and TSS loadings; the results were then compared to the sampling data. Based on the analyses of the solids hauling data, per capita BOD and TSS values were established at 0.20 pound per capita per day (ppcd) and 0.22 ppcd, respectively. These values fall within the expected range for BOD and TSS loadings for a city the size of Sweet Home comprising a typical mix of residential, commercial, and industrial customers.

The average monthly BOD and TSS values for the sampling record before May 2011 are 0.12 ppcd and 0.20 ppcd, respectively. The average monthly BOD and TSS values for the sampling record after May 2011 is 0.24 ppcd and 0.52 ppcd, respectively. Influent BOD samples taken after May 2011 have a better correlation with the results obtained from the solids hauling record, whereas influent TSS records taken before May 2011 have a better correlation with the solids hauling data. For purposes of facility planning, the annual average influent loadings are assumed to be 0.20 ppcd BOD and 0.22 ppcd TSS. Other design loading conditions are derived by scaling up from the annual average based on ratios determined through review of historical data. The resulting influent loading projections are listed in Table 3-3.

Table 3-3. Existing and Projected Loads

Parameter	BOD (ppd)[a]	TSS (ppd)
Existing design loading		
Annual average	1,834	2,017
Average dry weather	1,981	2,381
Max month dry weather	2,549	2,703
Max month wet weather	2,568	2,824
Peak day	4,365	5,846
2040 design loading		
Annual average	2,452	2,697
Average dry weather	2,648	3,182
Max month dry weather	3,408	3,614
Max month wet weather	3,433	3,776
Peak day	5,835	7,816

a. Mass loads were calculated by applying WWTP specific peaking factors to the baseline per capita values.

Effluent data were reviewed to confirm per capita projections utilizing hauled solids data were not affected by solids discharged from the plant. The effluent data shows a consistent discharge with seasonal variation but no rapid increase during the period of 2011 through 2014. The average CBOD and TSS pounds per day discharge over the entire sampling period was approximately 50 ppd and 80 ppd, respectively. This is equivalent to approximately 0.005 and 0.009 ppcd, respectively, and considered negligible when establishing the baseline from which future loads at the plant were projected.

The historical record shows that influent BOD and TSS loads are not highly correlated to season or flow rate. Therefore, influent is expected to be more dilute as flows increase with wet weather and more concentrated in dry weather when flows are lower. The resulting concentrations, based on the average dry weather flow (ADWF) and the average dry weather load, are 170 milligrams per liter (mg/L) and 204 mg/L for BOD and TSS, respectively. The resulting concentrations, assuming the average wet weather flow and the peak day load, are 210 mg/L and 282 mg/L, respectively.

Influent ammonia (NH_3) sampling is not a permit requirement so data are not available. The average ammonia loading was assumed at 30 mg/L at the average annual daily flow. Influent ammonia for untreated domestic wastewater normally ranges between 15 mg/L and 40 mg/L.

3.6 Cost Estimating

Development of the engineer's opinion of probable construction cost is discussed in this section. A summary of the cost estimate for the recommended alternative is presented in Appendix F.

3.6.1 Association of Cost Engineering International Estimate Classification

The costs developed for the Sweet Home Facilities Plan were prepared in accordance with the Association of Cost Engineering International (AACEI) criteria for a Class 4 estimate. A Class 4 estimate is defined as a conceptual-level or project viability estimate. Typically, engineering design work is 1 to 15 percent complete. Class 4 estimates are used to prepare planning-level cost scopes, evaluate alternative schemes, or develop long-range capital outlay plans. Class 4 estimates can also be used as the basis of a Class 3 estimate for project budgets and funding.

3.6.2 Accuracy

Expected accuracy of Class 4 estimates typically range from -30 percent to +50 percent, depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. In unusual circumstances, ranges could exceed the ranges stated.

3.6.3 Estimating Methodology

The estimate was prepared using take-offs of excavation and concrete work, vendor quotes, and equipment pricing furnished by either the product team or the estimator. The estimate includes direct labor costs and anticipated productivity adjustments to labor and equipment.

Construction labor crew and equipment hours were calculated from production rates contained in documents and electronic databases published by R.S. Means, Mechanical Contractors Association (MCA), National Electrical Contractors Association (NECA), and Rental Rate Blue Book for Construction Equipment (Blue Book).

The estimate was prepared using BC's estimating system, which consists of a Windows-based commercial estimating software engine using BC's material and labor database, historical project data, the latest vendor and material cost information, and other costs specific to the project local.

3.6.4 Engineering and Administrative Cost, Contingencies

The cost of engineering services for projects of this nature typically covers a standard geotechnical investigation, survey, and preparation of contract documents. Construction management activities, including startup services, the preparation of operation and maintenance (O&M) manuals, and performance certifications, were applied as a separate line item. Depending on the size and type of project, engineering, administration, and construction management costs may range from 20 to 30 percent of the construction contract cost. The lower end of the range applies to large projects without complicated mechanical systems. The higher end of the range applies to more complicated projects or projects that involve extensive remodeling of existing plants. The City has its own administrative costs associated with any construction project. These include internal planning and budgeting, the administration of engineering and construction contracts, legal services, and liaison with regulatory and funding agencies. The City's administrative costs are assumed to be approximately 3 percent of the construction contract cost. A 35 percent contingency is typical of an AACEI Class 4 estimate.

Section 4

Existing Facilities

This section describes the existing City collection system and WWTP facilities.

4.1 Collection System

The City's wastewater collection system comprises approximately 275,000 linear feet (LF) of sewer pipe. Construction of the collection system began as early as 1910. The sewer pipe ranges from 6 to 24 inches in diameter with over 80 percent of the pipe sized at 8 inches. Most pipes are constructed of non-reinforced concrete in 3½-foot sections. Other pipe materials include reinforced concrete, cast iron, and polyvinyl chloride. No pump stations are associated with the collection system; all wastewater is conveyed to the WWTP by gravity. Figure 4-1 shows an overview of the City's collection system and location of the WWTP.

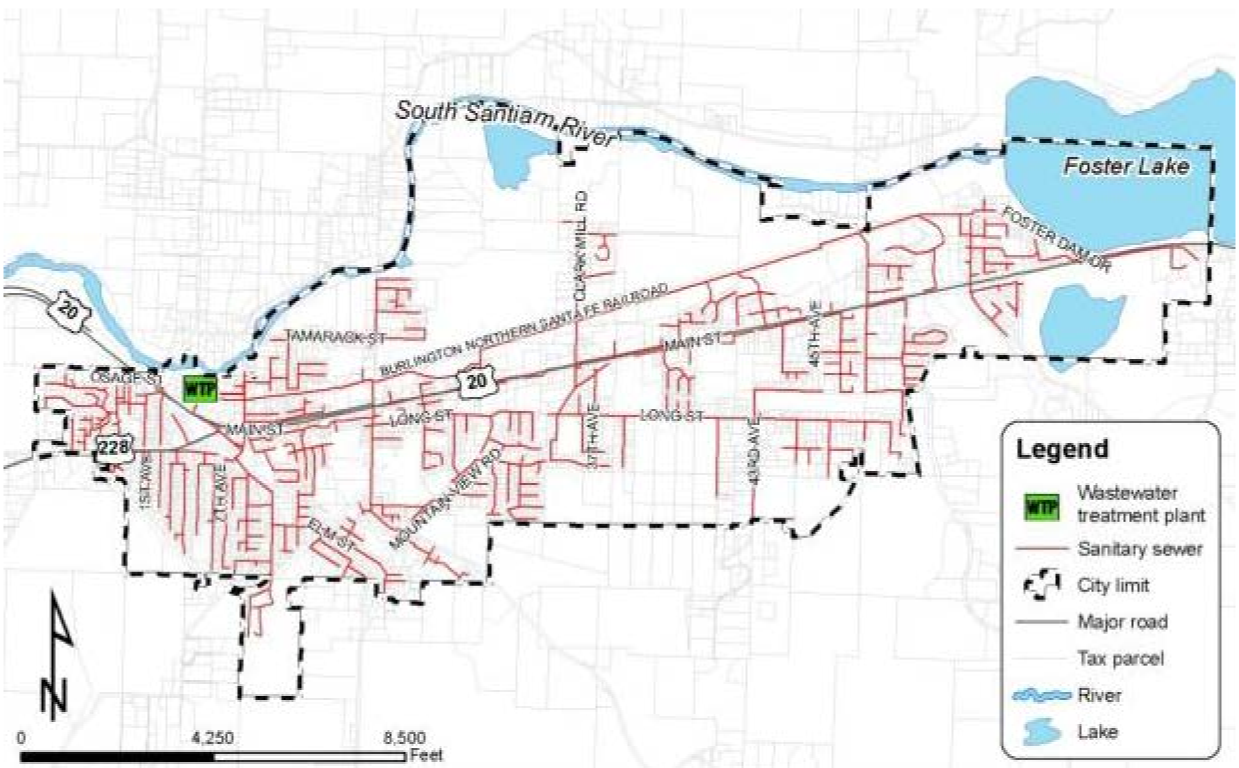


Figure 4-1. Overview of the Sweet Home sanitary collection system

The City's collection system comprises 27 sanitary drainage basins, 19 of which have residents within their boundaries connected to the public sewer system. Figure 4-2 shows the extents of the 27 sanitary drainage basins.

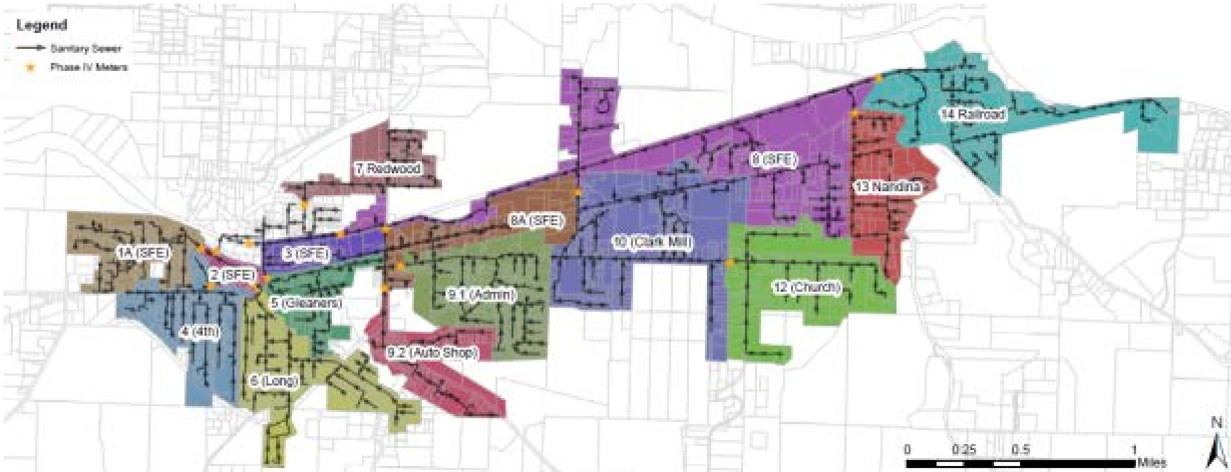


Figure 4-2. Sweet Home sanitary collection drainage basins

4.1.1 Collection System Rehabilitation

Historically, the City's collection system has been subject to high levels of rainfall-derived inflow and infiltration (RDII). The resulting flows were overwhelming the collection system's capacity, resulting in sanitary system overflows (SSOs) that are prohibited by the City's NPDES permit. In February 1996, the City and DEQ entered into an MAO recognizing that the City would continue to experience SSOs while agreed-upon investigations were conducted, and corrections to the collection system were made. In accordance with OAR 341-041-0120, the MAO requires the elimination of SSOs caused by less than the 1-in-5-year flow recurrence from November 1 through May 21, and by less than the 1-in-10-year flow recurrence from May 22 to October 31. The City requested an extension of the MAO In January 2001, acknowledging that it had not yet been able to make sufficient progress on collection system improvements to achieve the mandates set forth in the MAO.

An investigation conducted in 2001 concluded that in order to meet the no-SSO mandate, the City would need to either reduce I/I within the collection system, at an estimated cost of $30M, or increase the capacity of the WWTP, at an estimated cost of $17M. Despite the higher costs associated with I/I reduction, the City recognized that its collection system was in need of structural repairs and that not addressing the deterioration would lead to potential failures and additional I/I. Based on this reasoning, the City embarked upon an aggressive I/I abatement program in 2002.

In 2003 and 2004, the first two phases of the I/I reduction programs were initiated. These projects, referred to as Phases 1 and 2, were undertaken in the seven drainage basins showing the highest degree of I/I. These projects demonstrated that holistic, basin-wide rehabilitation addressing manholes, sewer mains, and laterals was the most effective method of removing I/I. Phases 1 and 2 cost approximately $3M and concluded that rehabilitation of laterals on private property was an essential component of the overall I/I reduction program. Table 4-1 lists the results of the Phase 1 and 2 projects.

Table 4-1. Post-Phase 1 and 2 Rehabilitation Effectiveness Summary

I/I reduction method	Effectiveness at reducing I/I
Sewer mains and manholes	11%–16%
Laterals only	7%–11%
Sewer mains, manholes, and laterals to building	60%–88%

Phase 3 of the abatement program, undertaken in 2007, completed the holistic rehabilitation of basins initiated under Phases 1 and 2 and holistic rehabilitation in another previously unaddressed basin. The Phase 3 project was completed at a cost of approximately $3M. Post-rehabilitation flow monitoring and modeling were conducted to measure results and to target areas for future rehabilitation. Combined, the first three phases addressed 36,000 LF of sewer main, or approximately 15 percent of the sewers in the city. Approximately 700 laterals comprising about 20 percent of the total were either rehabilitated or replaced. Figure 4-3 shows the extent of rehabilitation for the first three phases of the rehabilitation work.

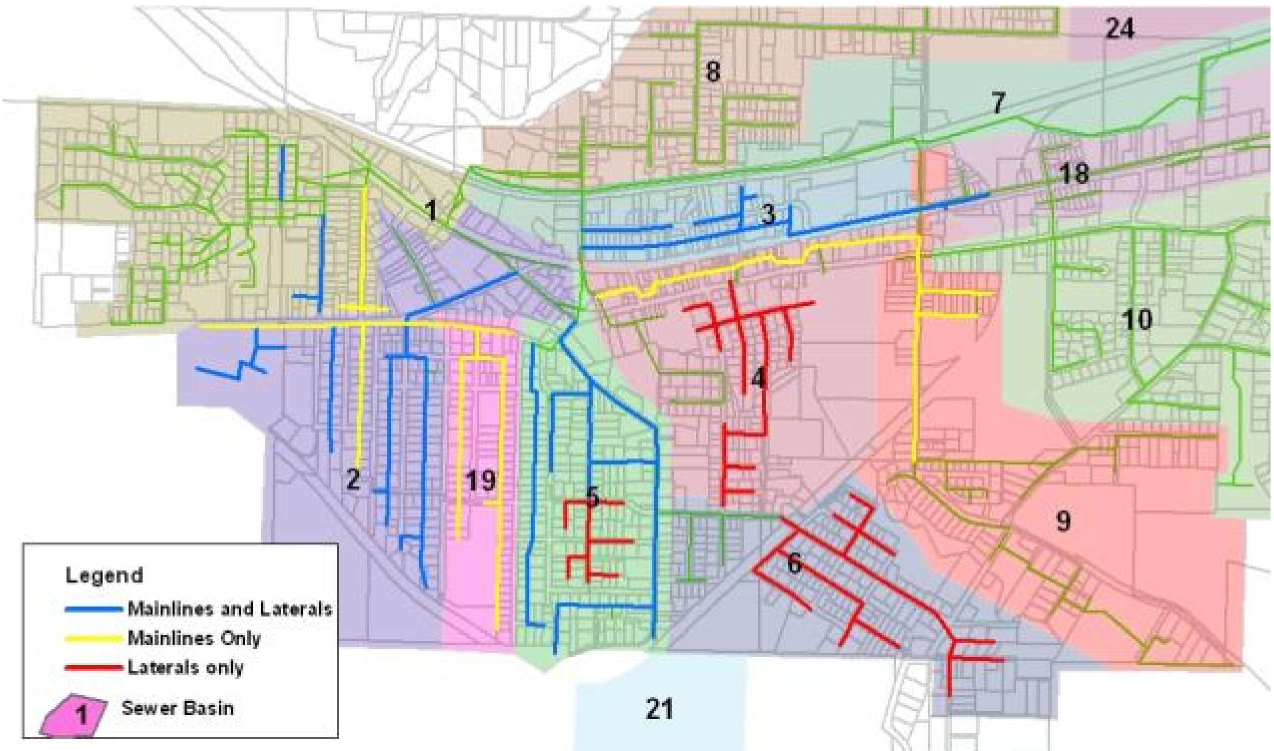


Figure 4-3. Phases 1, 2, and 3 rehabilitation work

Phase 4, the largest of the four I/I abatement projects, was completed in 2012. The $6M project involved 11 basins and rehabilitated or reconstructed 51,500 LF of sewer and 700 laterals.

In total, the City's I/I abatement program has addressed 92,500 LF of sewer main, or approximately 35 percent of the sewers in the city. A total of 1,250 laterals have been rehabilitated or replaced, which accounts for approximately 30 percent of the laterals in the city. Figure 4-4 shows the extent of rehabilitation for all four phases of the rehabilitation work.

Figure 4-5 shows the peak hour flows before and after the projects along with the peak flow capacity of the WWTP.

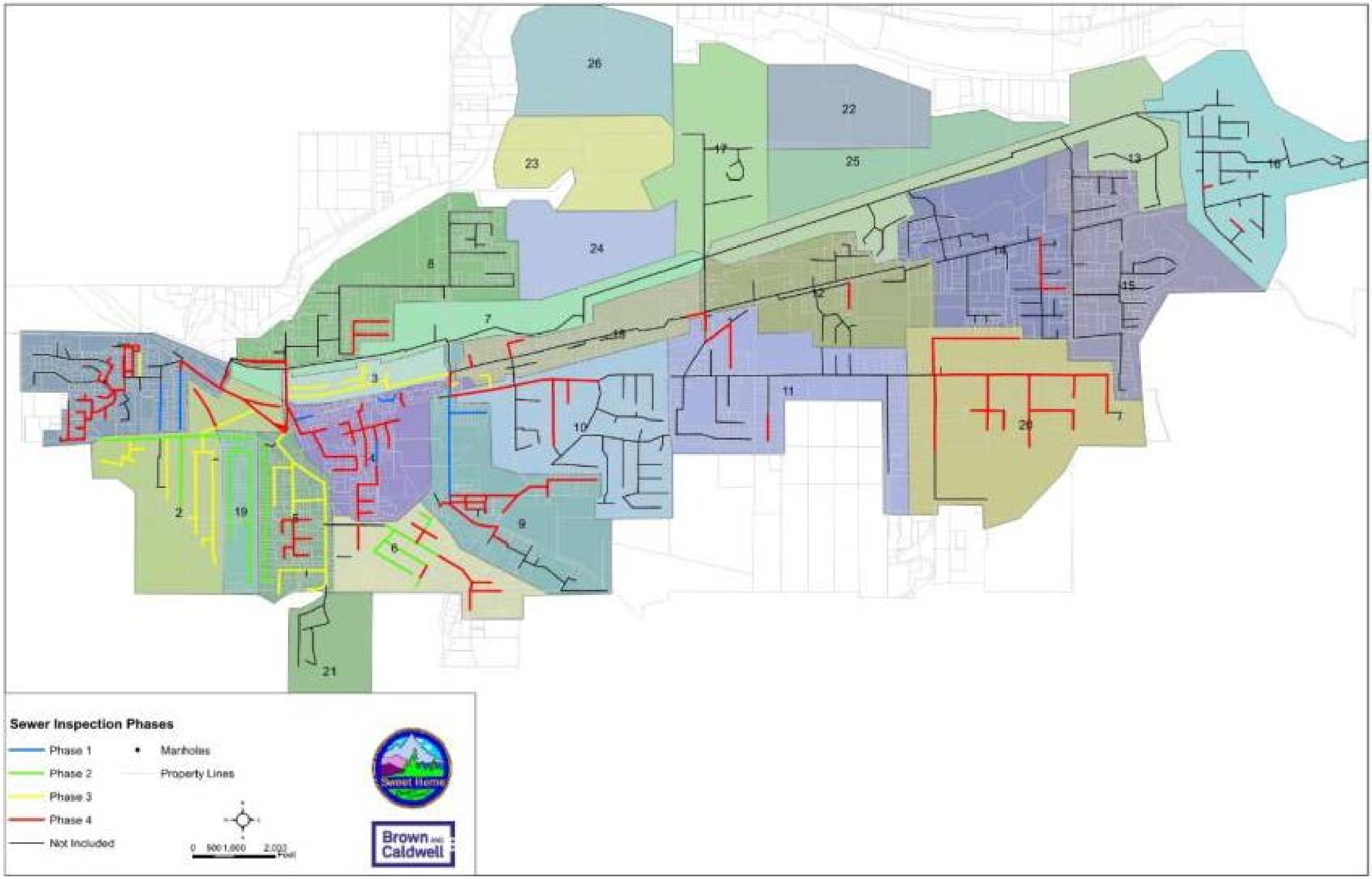


Figure 4-4. Phases 1, 2, 3, and 4 rehabilitation work

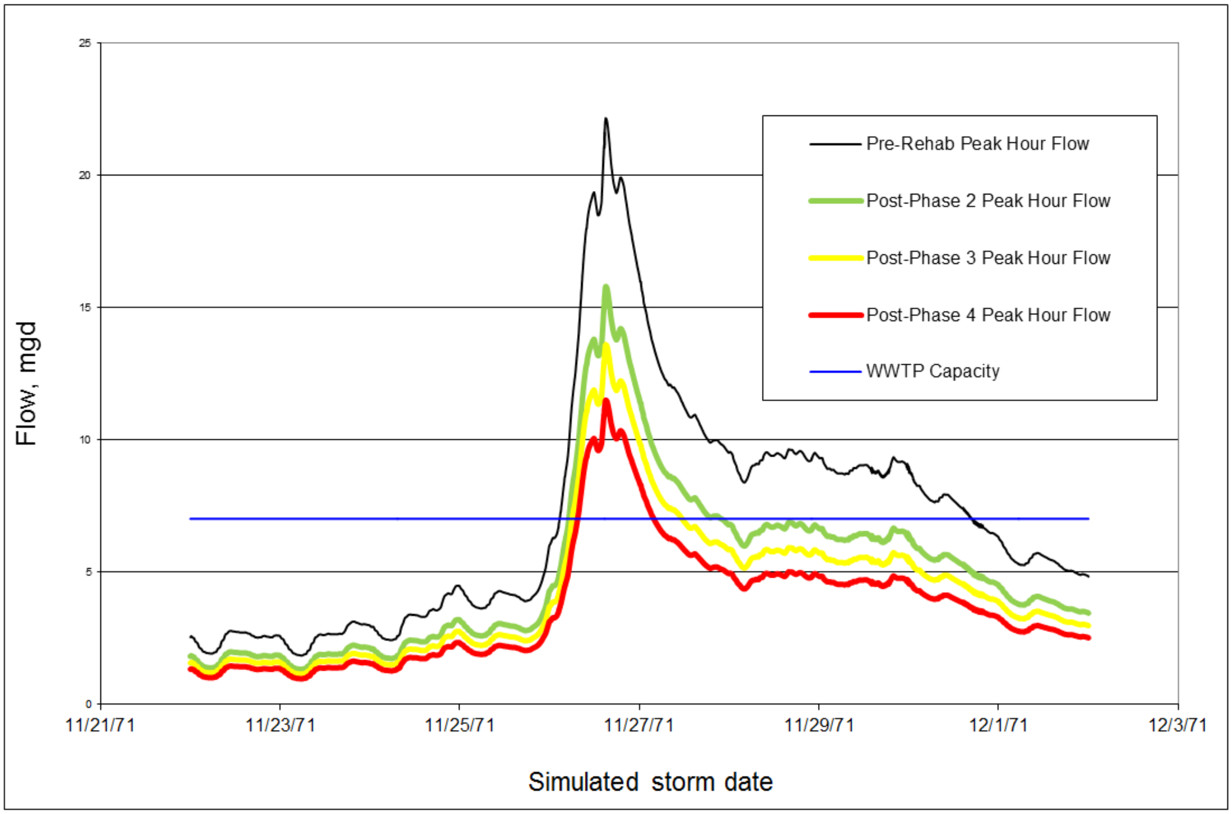


Figure 4-5. Peak hour flows and WWTP capacity before and after rehabilitation

4.1.2 Collection System Rehabilitation Summary

Since 2002, the City has invested more than $15M on planning, design, and construction of four collection system R&R projects within the service area. The construction costs for each phase are listed in Table 4-2.

Table 4-2. Summary of R&R Costs by Phase

Construction phase	Construction cost ($)
Phase 1	1.3M
Phase 2	1.7M
Phase 3	3.1M
Phase 4	6.0M

These projects have rehabilitated roughly 35 percent of the main line sewers and 30 percent of the laterals, and removed approximately 50 percent of the total I/I in the service area. The reduction in I/I has reduced existing flows to the WWTP from approximately 22 mgd to 12 mgd. This equates to approximately $1.10 spent for every 1 gallon per day of I/I removed. The areas incorporated into the four projects were carefully chosen to maximize cost-effectiveness of improvements and reduce peak flows to the WWTP most effectively.

Based on knowledge of the existing basins, additional projects to remove I/I would not be so cost-effective. For instance, sanitary basins 18 and 9 would be the next most cost-effective areas to rehabilitate, at an estimated cost of $3 per gallon of I/I removed. Reduction of I/I in other basins will have a declining return on investment. Basin 8 for example, would require an expenditure of approximately $10 per gallon of I/I removed.

There were no major wet weather events between 2012 and November 2015, so no overflows occurred during that period. However, heavy rains in December 2015 culminated in an overflow on December 17th. The collection system response to this event is estimated at approximately a 2-year recurrence. The overflow was deemed by DEQ in their February 18, 2016 letter to the City as "beyond the reasonable control of the permittee," so no enforcement action was taken.

4.2 Wastewater Treatment Plant

The WWTP is located on the south bank of the South Santiam River adjacent to Ames Creek. The original Imhoff tank and trickling-filter-based WWTP was constructed at the current site in 1947 and upgraded to an activated sludge facility in 1974. Additional improvements were made in 1993 to accommodate a growing population and higher flows resulting mainly from increasing collection system I/I over time.

The existing WWTP incorporates influent pumping, a conventional activated sludge process and secondary clarification, tertiary filtration, and final effluent disinfection and dechlorination. Residual solids are stored temporarily and then dewatered and hauled offsite for landfill disposal. The existing unit processes are described in the following section.

4.2.1 Plant Layout

The WWTP is bordered on the north side by the South Santiam River and on the south side by the Burlington Northern railroad tracks. The City's maintenance yard is located to the east of the WWTP on the same parcel of property. An aerial view of the existing WWTP is shown in Figure 4-6. A flow schematic showing the existing unit processes is provided as Figure 4-7.

Figure 4-6. Existing WWTP aerial view

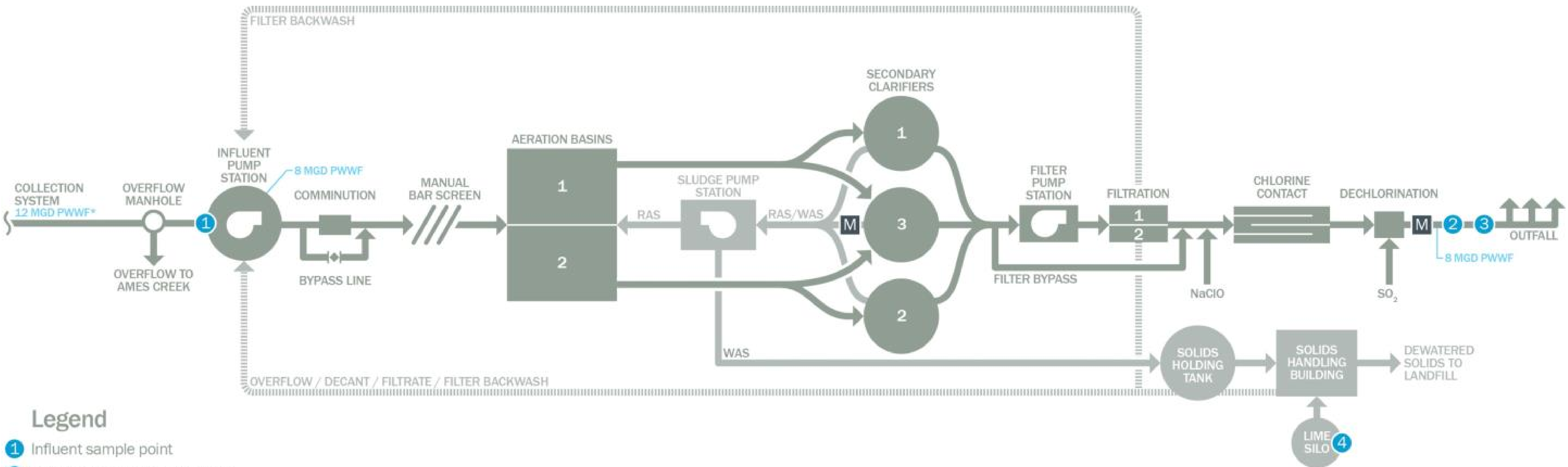


Figure 4-7. Existing WWTP process flow schematic

4.2.2 Influent Pumping

WWTP influent is conveyed to the IPS by gravity and pumped to the headworks. The IPS uses a round, cast-in-place structure that was constructed as part of the original WWTP. New, larger-capacity drywell pumps were installed in 1975. In 1993, the station was converted from a drywell pump configuration to one using one small and two large submersible pumps. Firm pumping capacity of the pump station was increased to 6 mgd.

A 16-inch-diameter, parallel force main was also constructed as a part of the 1993 submersible pump conversion project. The original 12-inch-diameter force main is dedicated to the smaller pump; the two larger pumps discharge to the 16-inch-diameter force main. The dual force mains allow the smaller pump to be operated in conjunction with the larger pumps, and all three pumps can produce approximately 9 mgd when pumping simultaneously. The physical size and configuration of the IPS will not allow for additional capacity expansion. Figure 4-8 shows exterior and interior photos of the IPS.

Figure 4-8. IPS exterior and dual force mains (left), and interior above-grade discharge piping (right)

4.2.3 Influent Flow Sampling

A refrigerated, 24-hour composite sampler is used for influent flow sampling. The sample is taken from the IPS wetwell. Figure 4-9 shows a photo of the flow sampler.

Figure 4-9. Influent flow sampler at IPS wetwell hatch

4.2.4 Preliminary Treatment

The IPS discharges to a small headworks structure integrated within the upstream portion of the aeration basins; it was originally equipped with twin mechanical comminutors. In 1993, the two comminutors were removed and manual bar screens were installed at their locations. In-line grinder stations were added to both force mains upstream of the headworks. The in-line grinders were intended to replace the comminutors; however, they are no longer used because of excessive maintenance requirements and generally poor performance. Currently, preliminary treatment consists exclusively of the manually cleaned bar screens, which is inadequate for a WWTP of this size.

Return activated sludge (RAS) is introduced at the downstream end of the headworks structure, where it mixes with raw influent prior to entering the aeration basins. There are no grit removal provisions. Figure 4-10 shows the influent grinder station associated with the 16-inch-diameter force main and the manual bar screens at the headworks.

Figure 4-10. Influent grinder station (left) and manual bar screens (right)

4.2.5 Secondary Treatment

Secondary treatment is provided in twin parallel aeration basins, each equipped with two platform-mounted surface aerators. The aeration basins, constructed in 1975, are completely mixed rectangular tanks with no baffles or dividing walls. Secondary effluent is settled in three circular clarifiers. Two of the clarifiers, clarifiers 1 and 2, were constructed in 1975. Clarifier 3 was constructed as part of the plant upgrade in 1993.

RAS from clarifiers 1 and 2 is introduced at V-notch weirs at the headworks. RAS from clarifier 3 is introduced downstream of the headworks. RAS from clarifier 3 can be conveyed to either or both basins, whereas aeration basins 1 and 2 are configured to be operated with clarifiers 1 and 2, respectively. Flow distribution of secondary effluent is controlled by four weir boxes at the downstream end of the aeration basins. Effluent from either aeration basin can be conveyed to clarifier 3; flow to clarifiers 1 and 2 must be conveyed from aeration basins 1 and 2, respectively.

Clarifier 3 has a larger diameter and is deeper than the original clarifiers to provide additional capacity and promote better performance. Clarifier 3 incorporates a rapid-rate sludge collection mechanism and energy-dissipating inlet. All three clarifiers use peripherally mounted effluent launders with V-notch weirs. Figure 4-11 shows one of four surface aerators in the aeration basins and the newer, larger secondary clarifier.


Figure 4-11. Aeration basin 1 (left) and secondary clarifier 3 (right)

4.2.6 RAS and WAS Pumping

RAS pumping is required to draw down the continuously forming sludge blankets in the secondary clarifiers and recirculate biological solids back to the aeration basins. Waste activated sludge (WAS) pumps are used to control the concentration and mass of solids in the secondary treatment system by periodically removing biological solids. Scum accumulating on the clarifier surface is collected in a scum box and pumped with the WAS to the solids holding tank.

The RAS and WAS pumping facilities are housed in the sludge pump building located at the down-stream end of the aeration basins. The original sludge pumping facilities, constructed in 1975, con-sisted of three RAS pumps and two WAS pumps to serve clarifiers 1 and 2. The footprint of the sludge pump building was enlarged by 20 percent in 1993 to accommodate two additional RAS pumps and one additional WAS pump serving clarifier 3.

All five RAS pumps are vertical centrifugal-type and equipped with variable-speed drives, that are manually set for normal operations. The drives are limited to an operating band of 48-60 Hz. Flow based control is only available for the RAS pump dedicated to Clarifier 3. The two original WAS pumps are self-priming types, and the newer WAS pump is a piston-type pump. Figure 4-12 shows the RAS and WAS pumping facilities.


Figure 4-12. RAS pumps (left) and WAS pumps (right)

4.2.7 Filtration

Tertiary treatment is provided in twin parallel, traveling-bridge sand filters constructed in 1993. The sand filters replaced a pressure filtration system installed in 1975. The two sand filters have a combined peak capacity of approximately 4 mgd and are normally used year-round to improve final effluent quality. Secondary effluent is initially pumped to the upstream distribution channel and then conveyed by gravity through the filters. Filter effluent is conveyed by gravity to the chlorine contact tank (CCT).

The filter pump station is equipped with one 7.5-horsepower (hp) pump and one 30 hp pump to convey the range of flows up to approximately 4 mgd. The filters are automatically backwashed by the traveling-bridge mechanisms; backwash is conveyed by gravity to the IPS and recycled through the WWTP (there is no influent flow meter to be affected by this stream). The existing sand filters and filter pumping station are shown in Figure 4-13.


Figure 4-13. Traveling-bridge sand filters (left) and filter pump station (right)

4.2.8 Disinfection and Dechlorination

Disinfection and dechlorination of final effluent is accomplished with liquid sodium hypochlorite and gaseous sulfur dioxide, respectively. Contact time is provided in the CCT consisting of twin parallel basins. The primary disinfection chemical feed point is at the upstream end of the CCT, but sodium hypochlorite can also be added in the effluent launders of the secondary clarifiers. Introduction of sodium hypochlorite is paced to effluent flow rate and chlorine residual by a primary feed pump. A second hypochlorite pump is a backup unit and is flow-paced. Sulfur dioxide is introduced at the effluent structure located at the downstream end of the CCT. Axial flow mixers enhance mixing of both chemicals with effluent. The CCT is equipped with permanent baffles to provide a serpentine flow pattern, thus minimizing short circuiting and maximizing contact time. The two tanks have a combined volume of 50,000 gallons, which provides approximately 36 minutes of detention time at the average dry weather design flow, and approximately 10 minutes of detention time at the peak day design flow rate. The length to width ratio of the tank (17.3 : 1) is well below the current design criterion.

Sodium hypochlorite solution used for disinfection is stored at approximately 13 percent active concentration. Daily chlorine (Cl_2) usage is typically between 35 and 40 pounds (lb). One 500-gallon storage tank and the two feed pumps for sodium hypochlorite are located in the administration building in the location previously used for storage of chlorine gas. The storage tank is located within a secondary spill containment area constructed of concrete masonry unit (CMU) block.

Introduction of sulfur dioxide for dechlorination is paced to effluent flow rate and chlorine residual. The final effluent structure incorporates an axial flow mixer for rapid dechlorination. A final effluent sample is continuously pumped to the chlorine residual analyzer located in the administration building. Chemicals and feed equipment for dechlorination are housed in the administration building. Sulfur dioxide is delivered and stored in 150 lb cylinders. Figure 4-14 shows the CCT and final effluent structure.


Figure 4-14. CCT (left) and effluent structure with chemical mixer (right)

4.2.9 Flow Metering and Effluent Sampling

Flow metering for the WWTP is provided at the final effluent structure constructed adjacent to the CCT. The structure is equipped with a static weir, ultrasonic level sensor, and signal transmitter. Flow rate is estimated by monitoring the height of the water surface above the weir. Flows beyond 7 mgd will flood the weir and diminish flow metering accuracy.

A refrigerated 24-hour composite sampler is used for effluent flow sampling. The sample is taken at the final effluent structure. Figure 4-15 shows the flow metering and sampling locations.


Figure 4-15. Effluent flow metering and sampling

4.2.10 Outfall

Disinfected and dechlorinated effluent is conveyed from the effluent structure to the South Santiam River through a 14-inch-diameter ductile iron pipe. Approximately 60' of the discharge pipe was replaced and realigned in 1993 to facilitate construction of the sand filters.

The submerged portion of the pipe was damaged and repaired in 2000. The repairs consisted of adding a wye to the existing pipe and installing a parallel pipeline and diffuser. The diffuser, incorporating eight 7-inch-diameter holes in the submerged portion of the outfall, was originally intended to enhance mixing, but the end is currently open to maximize outfall capacity. The end of the diffuser extends about 15 feet from the bank.

The outfall pipe and associated flow control structure have a peak capacity of approximately 8.5 mgd. Higher flows will back up water surface elevations in upstream structures. Figure 4-16 shows the outfall vicinity and outfall pipe at the point where it enters the river.

Figure 4-16. Outfall vicinity (left) and outfall pipes (right) (lower pipe abandoned)

4.2.11 Standby Generator Building

Standby power is provided by a 150-kilowatt (kW) diesel generator located in the standby generator building. According to plant staff, the generator has capacity to power the influent pumps, automatic valves at the headworks, motorized gates at the junction box, select lighting throughout the plant site, and disinfection equipment. There is insufficient standby power capacity for SCADA functionality, aeration, clarification equipment, RAS and WAS pumping, or for solids holding tank operation. Thus, when operating on standby power, treatment is limited to conveyance, settling, and disinfection. The influent sampler is powered during an outage but the effluent sampler requires manual resetting. Alarms rely on battery backup.

The existing building, generator, and appurtenances are all in good condition. Variable-frequency drives (VFDs) for the three influent pumps are located in the building. The exterior and interior of the standby generator building are shown in Figure 4-17.

Figure 4-17. Standby generator building (left) and equipment (right)

4.2.12 Administration Building

Operations and personnel facilities are located in the administration building, constructed in 1975. The total area of the building is approximately 1,650 square feet (ft^2). The building incorporates a laboratory facility; disinfection and dechlorination equipment rooms; maintenance shop; and wash-room including lavatory, water closet, and shower.

The laboratory occupies about 440 ft^2. The shelf and counter space in the laboratory are generally adequate for normal analytical activities and there is a single, light-duty fume hood.

The sodium hypochlorite feed equipment and storage tank are located in the area originally used to house chlorine gas cylinders. The dechlorination room houses the chlorine residual analyzer, com-pound loop controller, sulfonator, and several 150 lb sulfur dioxide gas cylinders. An equipment maintenance garage is located on the north side of the building. The original pressurized filtration system equipment room has been converted to personnel and meeting space. Figure 4-18 shows the exterior of the administration building and laboratory space.

Figure 4-18. Administration building (left) and laboratory (right)

4.3 Solids Handling

The City produces about 200 DT of solids per year requiring disposal. Solids management practices currently include temporary storage of WAS, dewatering, and landfilling of dewatered solids at the Wasco County Landfill located near the city of The Dalles, OR.

The City previously administered a Class B land-application program using lime stabilization. The stabilized biosolids were applied to a 600-acre, privately owned agricultural site in rural Linn County. Seasonal restrictions on land application and a lack of dewatered solids storage capacity at the WWTP made land application of solids problematic. Funding for process upgrades was unavailable and the decision was made to go with landfill disposal instead.

4.3.1 Solids Holding Tank

WAS is pumped from the secondary clarifiers to the solids holding tank. The holding tank, which was originally constructed to provide aerobic digestion, has a capacity of 231,000 gallons, providing approximately 15 to 20 days of WAS storage at current production rates. The holding tank was retrofitted with a cover, blowers, and coarse-bubble diffusers to contain odors and keep solids in suspension. Figure 4-19 shows the solids holding tank cover and mixing blowers.

Figure 4-19. Covered solids holding tank (left) and mixing blowers (right)

4.3.2 Solids Handling Building

Solids stored in the solids holding tank are processed in the sludge handling building. The 800 ft^2 building, constructed in 1975, houses a solids feed pump, in-line grinder, and 0.7-meter belt filter press (BFP) in the larger room. Polymer feed equipment and the plant's compressed-air equipment is housed in the adjacent room. The standby generator equipment was originally housed in this building but relocated to a dedicated building in 1993. Ventilation of the building is passive in that there are no central ventilation provisions. Solids are fed to the dewatering equipment at a concentration normally ranging between 1.5 and 3 percent. The BFP has a capacity of approximately 35 gallons per minute (gpm) and dewaters the feed material to about 13 percent solids concentration on average.

Figure 4-20 shows the dewatering room containing the in-line grinder, pneumatic sludge pump, BFP, load-out conveyor, and 20-cubic-yard (yd^3) container at the exterior of the building. Figure 4-21 shows the polymer conditioning and feed equipment and the plant air compressor located in the room adjacent to the dewatering equipment.


Figure 4-20. Dewatering room interior (left) and exterior (right) with 20 yd^3 container on rails


Figure 4-21. Polymer feed equipment (left) and plant compressor (right)

Dewatered solids are conveyed from the BFP to a 20 yd^3 container on an open-air belt conveyor. It typically takes 4 to 5 days to fill the container, at which point the container is hauled offsite and replaced with an empty container. The container must be moved as it fills because the drop point from the conveyor is stationary. There are no provisions for containment of odors emitted during conveyance and storage of solids, although the storage container is mostly covered.

4.3.3 Solids Stabilization

Residual solids are not currently stabilized. Solids generated at the WWTP were originally digested aerobically in the tank currently used for solids storage. More recently, solids were stabilized to Class B requirements with lime stored in a silo located adjacent to the solids handling building. The mixing auger previously used to mix lime with dewatered solids has been removed. Control of lime dust and odor generation was problematic during the lime stabilization process. There are no provisions for storage of stabilized solids if weather does not permit land application. Because of the challenges associated with lime addition, a lack of storage facilities, and the favorable economics associated with landfilling solids, solids stabilization and the land application program have been discontinued. Figure 4-22 shows the existing lime silo located adjacent to the solids handling building.

Figure 4-22. Lime silo adjacent to solids handling building

4.3.4 Odor Control

Odors generated within the solids storage tank are treated by conveying the air through a compost/woodchip biofilter; a small blower is installed for this purpose. Odors generated within the dewatering room and the storage container are not treated. There are no other provisions for odor containment or odor control at the WWTP. Figure 4-23 shows elements of the WWTP odor control system.

Figure 4-23. Odor control blower (left) and odor control biofilter (right)

4.3.5 Solids Disposal

Solids disposal currently consists of landfilling dewatered cake at Wasco County Landfill near the city of The Dalles, OR. The City contracts with a local hauler to transport 20 yd³ containers from the WWTP to the landfill. The hauler makes 12 trips to the landfill per month on average.

4.3.6 Solids Quality

Biosolids produced at the WWTP meet EPA and DEQ limits for metals concentrations. Table 4-3 lists the results the City's solids quality analyses. Because solids are unstabilized, they do not meet pathogen and vector attraction reduction requirements for beneficial use.

Table 4-3. Biosolids Analysis

Parameter	Units	Average	Maximum	Minimum	MCL
Total solids	%	23.68	33	19.6	N/A
Volatile solids	%	46.39	58.3	29.7	N/A
Total nitrogen	%	3.6	4.72	2.73	N/A
Nitrate, nitrogen	%	0.01	ND @ 0.01	ND @ 0.01	N/A
NH_3, nitrogen	%	0.2	0.27	0.09	N/A
Phosphorus	%	0.84	1.07	0.58	N/A
Potassium	%	0.26	0.34	0.2	N/A
Arsenic	mg/kg	7.09	9	ND @ 5.0	41
Cadmium	mg/kg	1.18	2	ND @ 1.0	39
Chromium	mg/kg	12.62	16.6	ND @ 10.0	1,200
Copper	mg/kg	97.08	148	76.3	1,500
Lead	mg/kg	20.6	28.3	ND @ 10.0	300
Mercury	mg/kg	0.65	0.8	ND @ 0.4	17
Molybdenum	mg/kg	5.15	6.8	4	18
Nickel	mg/kg	9.57	12.7	ND @ 10.0	420
Selenium	mg/kg	8.53	ND @ 10.0	ND @ 5.0	36
Zinc	mg/kg	448.13	781	271	2,800

Notes: mg/kg = milligrams per kilogram.
ND = non-detectable.
MCL = maximum contaminant level.

4.3.7 Solids Production

The annual average of solids hauled from 2009 to 2014 was 203 DT. Operation issues and maintenance activities in the solids holding tank accounted for the reduced quantity of solids hauled in 2013. The amount hauled in 2014 included 38 DT of solids removed from the solids holding tank when the tank was drawn down and cleaned, and the diffusers were repaired.

Figure 4-24 shows the annual quantity of solids hauled to offsite disposal for the period 2009 to 2014.

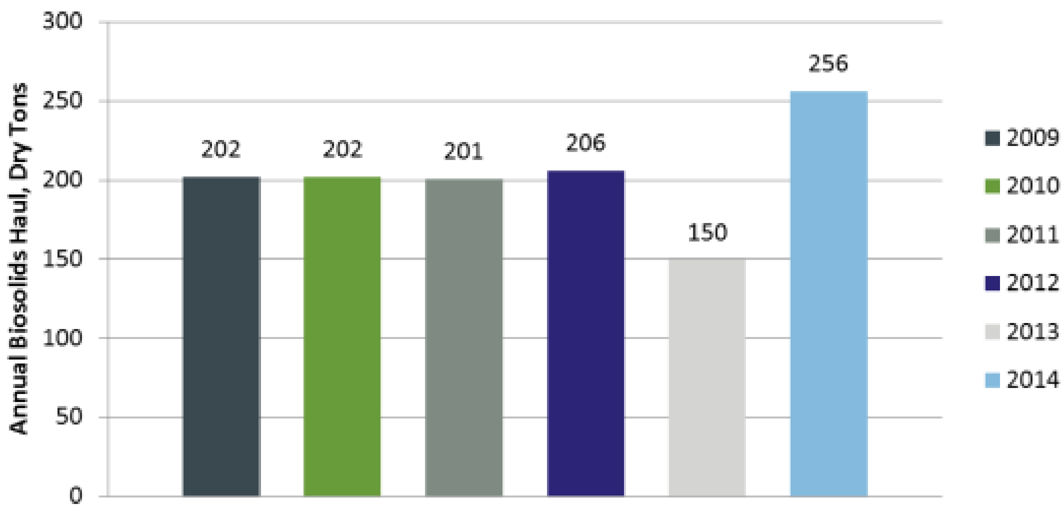


Figure 4-24. Solids hauled on annual basis

4.4 Design Data

Design data for the existing WWTP are shown on Table 4-4.

Table 4-4. Design Data Summary, Existing Facilities	
System	Data/Type
IPS	
Pump 1 (gpm/TDH)	3,500 @ 50 ft TDH
Pump 2 (gpm/TDH)	700 @ 40 ft TDH
Pump 3 (gpm/TDH)	3,500 @ 50 ft TDH
Peak firm pump capacity (mgd)	6
Preliminary	
Sewage grinders (decommissioned)	
Type	In-line
Number	2
Width (in.)	12/18
Capacity/unit (mgd)	1.70/5.76
Total capacity (mgd)	7.4
Bypass channel (mgd)	8.5
Horsepower	5
Bar screen	
Type	Manual
Number	2
Opening size (in.)	2
Horsepower	N/A
Secondary treatment	
Aeration basin	
Number	2
Length, width, depth (ft)	64/30/12
Volume/basin (gal)	172,000

Table 4-4. Design Data Summary, Existing Facilities	
Aeration	
Type	Surface
Multi-speed	Yes, 2-speed
Number (per basin)	2
Horsepower, low speed/high speed	8.6/15
Wet weather treatment	
Type	N/A
Capacity (mgd)	N/A
Horsepower (total)	N/A
Secondary clarifiers	
Type	Circular
Number	3
Diameter (ft)	45/45/60
Avg. depth (ft)	12/12/15
Collection mechanism type	Scraper
Overflow rate (all clarifiers in service)	
ADWF (gal/ft^2/day)	248
PWWF (gal/ft^2/day)	1,990
AWWF (gal/ft^2/day)	411
Sludge pumps	
WAS pumps	
Type	Self-priming
Capacity (gpm)	3 @ 200
RAS pumps	
Type	Vertical centrifugal
Capacity (gpm)	3 @ 685
	2 @ 650
Biosolids holding tank	
Capacity (gal)	231,000
Solids processing	
Type	Belt filter press
Number	1
Size, meter	0.7
Capacity (gpm)	35
Normal cake (%)	13.5
Lime auger (removed)	N/A
Lime storage silo capacity (tons)	20
Tertiary treatment	
Type	Traveling-bridge sand filter
Number	2
Bed depth (in.)	11
Peak capacity (mgd)	4.0 (combined)
Design flow (mgd)	2.0
Peak loading rate (gpm/ft^2)	4
Average loading rate (gpm/ft^2)	2

Table 4-4. Design Data Summary, Existing Facilities	
Disinfection	
Chlorine gas, online (lb)	1 @ 2,000
Chlorine gas, storage (lb)	2 @ 2,000
Contact tank (gal)	50,000
L:W ratio	17.3:1
Detention	
ADWF (minutes)	38
PWWF (minutes)	2.9
Chlorination capacity (ppd)	200
Mixer	
Horsepower	2
Velocity gradient, s^{-1}	500
Dechlorination: sulfonator	
Tank volume (gal)	1,184
Sulfur dioxide cylinders	
Online (lb)	2 @ 150
Storage (lb)	2 @ 150
Sulfonator capacity (ppd)	100
Mixer	
Horsepower	2
Velocity gradient, s-1	573
Standby generator	
Rated capacity (kW)	150

Note: *Data adapted from WWTP Expansion drawings by KCM, Inc., 1992, and from information from plant operations staff.*

4.5 Plant Performance

WWTP performance from a regulatory perspective can be measured in terms of effluent quality, and compliance with permit limits. Historical plant performance is documented in this section.

To gauge past plant performance, effluent carbonaceous biochemical oxygen demand (CBOD) and TSS records for the period January 2009 through December 2014 are summarized in Table 4-5.

Table 4-5. Plant Effluent Summary

Month	CBOD		TSS	
	mg/L	Discharged (ppd)	mg/L	Discharged (ppd)
Annual average	3.4	50.9	5.5	95.7
Dry weather average	3.5	33.5	5.0	54.2
Wet weather average	3.3	68.3	6.1	141.7

Note: These values represent the average of monthly averages for the period covering 2008 through 2014.

Both the dry weather and wet weather averages for concentration and mass load are well below permit limits indicating that ordinarily, the plant is operating well.

The monthly variation in effluent quality and plant flow is shown in Figure 4-25. The graph illustrates that during periods of high flow, the effluent quality is typically degraded. This would be expected as plant performance will be optimized when flows and hydraulic loading rates are lower.

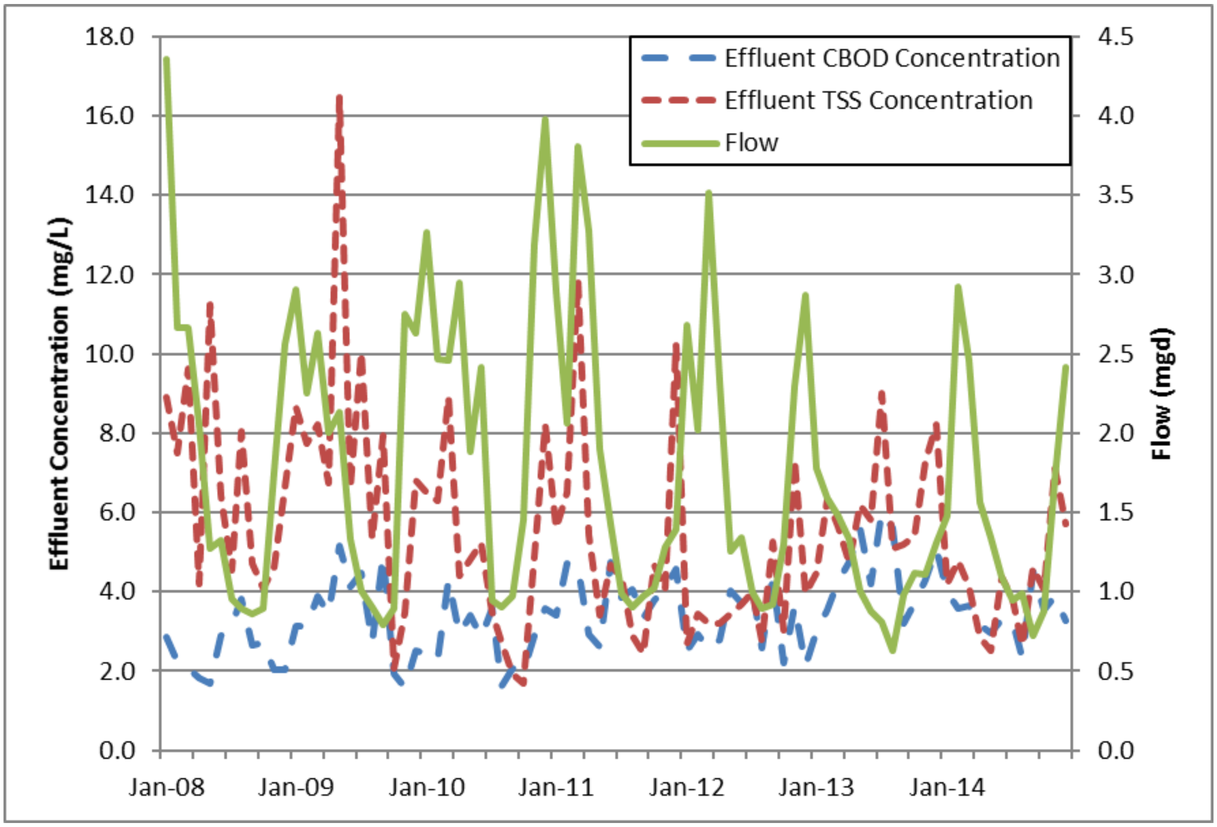


Figure 4-25. Monthly average flows and monthly average effluent quality

The occurrences of NPDES permit exceedances during the period 2008 through 2014 are shown on Table 4-6. The occurrences of MAO exceedances for the same period are shown on Table 4-7. The WWTP was operating under the MAO interim limits during this period.

Table 4-6. Number of Permit Exceedances between 2006 and 2014

Parameter	2007	2008	2009	2010	2011	2012	2013	2014
Overflow	4	2	1	2	2	3	0	1
Temperature	0	0	0	0	0	0	0	0
TSS	1	1	2	5	5	1	0	0
Cl_2	2	0	4	0	0	2	0	0
E. coli	0	3	0	2	1	1	0	0
NH_3	0	2	2	0	0	0	1	0
CBOD	0	0	1	0	0	0	0	0

Table 4-7. Number of MAO Exceedances between 2006 and 2014								
Parameter	2007	2008	2009	2010	2011	2012	2013	2014
Overflow	0	2	1	0	1	0	0	1
Temperature	0	0	0	0	0	0	0	0
TSS	1	1	2	0	1	0	0	0
Cl_2	2	0	5	0	0	2	0	0
E. coli	0	3	0	2	1	1	0	0
NH_3	0	2	3	0	0	0	1	0
CBOD	0	0	1	0	0	0	0	0

Mass limits for TSS account for more permit excursions than any other parameter because of the high flows the plant treats. This issue is discussed further in Section 5.

Monthly percent removal of CBOD and TSS data are shown in Figure 4-26.

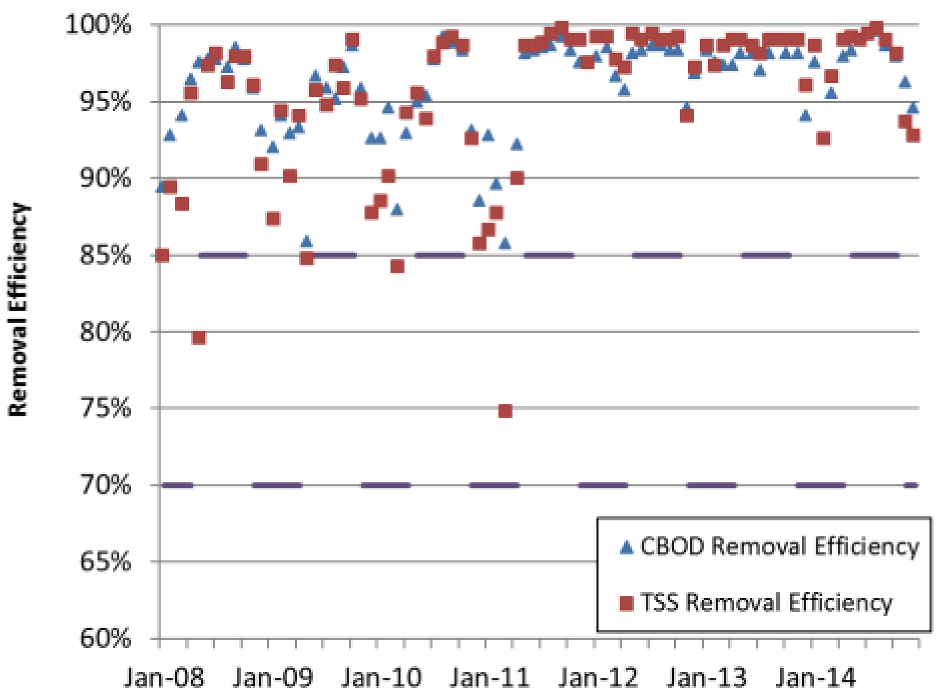


Figure 4-26. Monthly percent removal of CBOD and TSS

The percent removals are shown to be higher starting in May 2011. This corresponds with the increase in recorded influent loads shown on Figure 3-2. Actual influent loads are thought to be consistent over the period 2008 through 2014 based on solids hauled. Solids hauled for disposal have averaged 202 DT per year since 2008.

Section 5

Need for Project

The project identified in this Facility Plan is intended primarily to address insufficient hydraulic capacity at the WWTP to convey projected peak flows; provide additional treatment capacity as required for treating peak flows and for regulatory compliance; and upgrade some existing equipment for improved performance, increased reliability, and reduced maintenance.

This section provides a discussion on relevant regulations impacting treatment requirements, existing infrastructure, and its ability to meet current and future effluent limits.

5.1 Regulatory Background

This section presents information on the regulatory framework for the South Santiam River and wastewater treatment requirements for the City's WWTP. Because the amount of river flow available for dilution is a key factor, river flow information is also included.

5.2 South Santiam River Flow

Flow data for the South Santiam River are available from the U.S. Geological Survey (USGS) Water Data Reports for its Foster Dam (river mile 37) monitoring station. Flow data are available from 1973 to the present for the Foster site. Between 1968 and 1973, the gauge was 1/2 mile upstream and the data are not comparable. The years prior to 1968 are influenced by the construction of upstream dams at Foster Reservoir and Green Peter Reservoir. Table 5-1 summarizes the monthly average, maximum, and minimum river flows for the Foster monitoring station between 1973 and 2013.

Table 5-1. Monthly Flow Data for the Foster Monitoring Station, 1973–2013

Month	Average flow (cfs)	Maximum flow (cfs)	Minimum flow (cfs)
January	5,450	12,400	730
February	3,110	10,460	590
March	2,970	6,670	780
April	3,220	6,220	1,180
May	2,420	5,290	760
June	1,720	4,530	630
July	840	1,510	560
August	750	1,170	580
September	1,240	2,080	610
October	1,860	3,230	610
November	4,380	9,270	820
December	6,280	12,770	1,140

Note: Flow data obtained from USGS River Station 14187200 near Foster Dam.

The WWTP discharges to the South Santiam River at river mile 31.5. Discharge data from the USGS Foster site were analyzed using DFLOW, which is part of EPA's Better Assessment Science Integrated Point and Nonpoint Sources (BASIN 4.1) environmental analysis system. The following river flow statistics for low river flow were calculated, as well as the harmonic mean:

- 1Q10 = 528 cubic feet per second (cfs)
- 7Q10 = 549 cfs
- 30Q5 = 613 cfs
- Harmonic mean = 1,450 cfs

River flow varies seasonally and Figure 5-1 shows discharge curve from USGS Station 14187200.

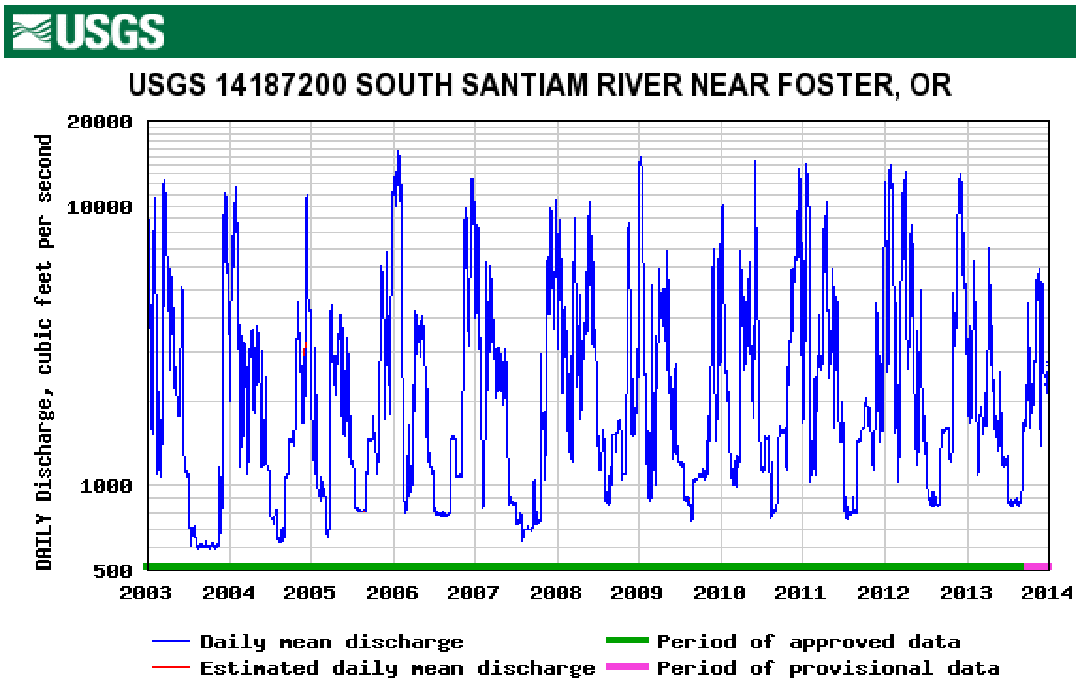


Figure 5-1. South Santiam River flow at Foster Dam

5.3 Regulatory Framework

The regulatory environment surrounding water quality protection in Oregon is relatively complex, requiring interaction and cooperation among a number of federal and state regulators. The first step in the process is to assign beneficial uses to the water body. This task is the responsibility of the Oregon Water Resources Department (OWRD). A water body's beneficial uses depend on characteristics such as its size and location. The following are the designated beneficial uses for the South Santiam River (OAR 340-041-0340):

- Public domestic water supply
- Private domestic water supply
- Industrial water supply
- Irrigation
- Livestock watering
- Fish and aquatic life

- Wildlife and hunting
- Fishing
- Boating
- Water contact recreation
- Aesthetic quality
- Hydropower

DEQ is responsible for establishing and enforcing water quality and waste treatment standards that ensure that the river's beneficial uses are preserved. DEQ's general policy is one of anti-degradation of surface water quality. Discharges from WWTPs are regulated through the NPDES. All discharges of treated wastewater to a receiving stream must comply with the conditions of an NPDES permit. EPA oversees state regulatory agencies, and can intervene if the state agencies do not successfully protect water quality.

This section summarizes the regulatory requirements pertinent to wastewater facilities planning for the WWTP.

5.3.1 OARs for Wastewater Treatment

The State surface water quality and waste treatment standards for the South Santiam River are detailed in the following sections of the OARs:

- OAR 340-041-0004 lists policies and guidelines applicable to all basins. DEQ's policy of anti-degradation of surface waters is set forth in this section.
- OAR 340-041-0007 through 340-041-0036 describe the standards that are applicable to all basins.
- OAR 340-041-0340 through 340-041-0345 contain requirements specific to the Willamette River Basin including beneficial uses, approved total maximum daily loads (TMDLs) in the basin, water quality standards, and the minimum design criteria for waste treatment.

The surface water quality and waste treatment standards in the OARs are viewed as minimum requirements. Additional, more stringent limits developed though the TMDL process are intended to bring the water body into compliance with the basin standards.

5.3.2 Clean Water Act Section 303(d) List

Every 2 years, DEQ is required to assess water quality and report findings to EPA including an identification of waters that do not meet applicable water quality standards. DEQ prepared the 2012 integrated report and submitted it to EPA in November 2014. EPA is currently reviewing the 2012 report. The single parameter assigned to a Category 5 (Water Quality Limited, TMDL needed) is the biological criteria. This listing is based on a single assessment at the City's outfall completed in August 2005. Oregon's biocriteria are contained in OAR 340-041-0011 and states as shown below:

> *Waters of the State must be of sufficient quality to support aquatic species without detrimental changes in the resident biological communities.*

DEQ developed a PREDictive Assessment Tool for Oregon (PREDATOR) to assess macroinvertebrate communities. This model analyzes data from reference sites and a specific sample is compared to the reference assemblage. If the measured taxa are more than 15 percent less than the reference, the site is listed as not meeting criteria.

In addition to the biocriteria, the following parameters are on the 2012 list as Category 3 status where no action is proposed:

- Alkalinity
- Copper
- Lead
- Manganese

Low alkalinity is a natural condition and a by-product of excellent source water. It is very unlikely that this will be an issue in the future. Wastewater discharged to the river increases the alkalinity.

EPA disapproved Oregon's copper criterion and the State is revising the Oregon water quality standard for copper. Based on EPA's recommendations and the feedback from the services, it is likely that the new standard will be premised on the use of the Biotic Ligand Model (BLM). Consequently, no action is anticipated related to copper until the new standard is in place.

For lead, only 1 of 11 samples and for manganese, 1 of 36 samples exceeded the criterion, and these were at river mile 7. No action is planned by DEQ.

5.3.3 Temperature TMDL

DEQ prepared the TMDL for temperature in the Willamette Basin in 2006, which was approved by EPA on September 29, 2006. Subsequently, Northwest Environmental Advocates (NWEA) sued EPA in federal court and prevailed in that litigation, which held that EPA should not have approved the TMDLs because the inclusion of a "natural-conditions criterion" was not legal. EPA has since disapproved the natural-conditions criterion contained in Oregon's water quality standards. NWEA has also sued EPA, asking that all of the TMDLs approved by EPA that contained a temperature natural-conditions criterion be disapproved. This litigation is ongoing and a decision is not expected before 2016. The status of the temperature TMDL for the Willamette River is uncertain because a natural-condition criterion was used by DEQ for the development of the TMDL.

The South Santiam River at Sweet Home is not water-quality-limited for temperature, but the river is listed as such in downstream sections and therefore DEQ provided a waste load allocation (WLA) to the City that is shown in Figure 5-2.

SWEET HOME WWTP
South Santiam River Mile 31.5
NPDES WQ File Number 86840
USGS Flow Gage 14187500

JUNE 16 - AUG 31 Core Cold-Water Habitat

7Q10	T_{RC}	T_{PS}	Q_{PS}	Q_{DF}
(cfs)	(Celsius)	(Celsius)	(cfs)	(cfs)
523	16.0	20.0	1.86	3.2

Q_R	T_{RC}	HUA	WLA
River Flow greater than (cfs)	River Temperature Criteria (Celsius)	Allowed Temperature Increase (Celsius)	Excess Thermal load (Million Kcals/Day)
0	16.0	0.0243	31

No flow based WLA formula is provided. Facility design flow (Q_{DF}) is limiting. Waste Load Allocation is calculated using facility design flows multiplied by 1.5.

SEPT 1 - JUNE 15 Salmon & Steelhead Spawning Use

7Q10	T_{RC}	T_{PS}	Q_{PS}	Q_{DF}
(cfs)	(Celsius)	(Celsius)	(cfs)	(cfs)
550	13.0	17.0	1.86	13.92

Q_R	T_{RC}	HUA	WLA	HUA	WLA
River Flow greater than (cfs)	River Temperature Criteria (Celsius)	Dry Weather Allowed Temperature Increase (Celsius)	Dry Weather Excess Thermal load (Million Kcals/Day)	Wet Weather Allowed Temperature Increase (Celsius)	Wet Weather Excess Thermal load (Million Kcals/Day)
0	13.0	0.0405	55	0.0987	136

Q_{DF} during Dry Weather period is 3.2 cfs. Q_{DF} during Wet Weather Period is 13.92 cfs.

Figure 5-2. DEQ temperature WLA to Sweet Home

The City collected temperature data for several years and these data are included in the City discharge monitoring report (DMR) submitted to DEQ. Figure 5-3 shows the 7-day moving average of the daily maximum temperature from 2008 through 2013.

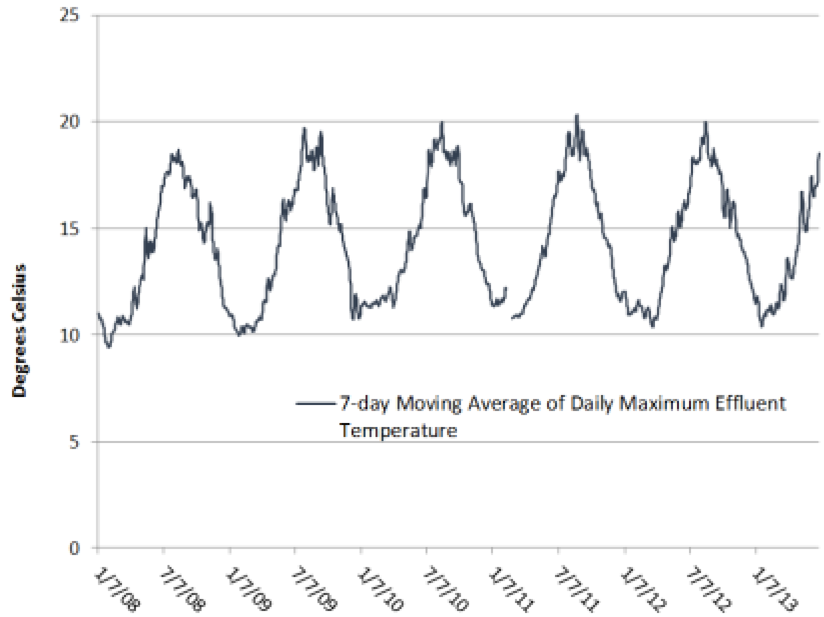


Figure 5-3 Effluent temperature

Source: City WWTP DMRs

Based on the data evaluated, it is unlikely that the City would violate the allocations shown in Figure 5-2. In the June 16 to August 31 period, the City's maximum excess thermal load is typically below 12 million kilocalories, well below the limit of 31 million kilocalories. During the remainder of the year when the dry weather limit is 55 million kilocalories, the maximum excess thermal load is below 33 million kilocalories. The wet weather excess thermal load allocation of 136 million kilocalories will not be approached because the effluent temperature is generally below the standard.

Should the temperature TMDL be vacated in its entirety, the plant effluent may not be allowed to increase the temperature by more than 0.3 degree Celsius (°C) above the applicable criterion if mixed with 25 percent of the river flow or at the regulated mixing zone (RMZ). The specific rule states as follows:

> *(A) Prior to the completion of a temperature TMDL or other cumulative effects analysis, no single NPDES point source that discharges into a temperature water quality limited water may cause the temperature of the water body to increase more than 0.3 degrees Celsius (0.5 Fahrenheit) above the applicable criteria after mixing with either twenty five (25) percent of the stream flow, or the temperature mixing zone, whichever is more restrictive;*

For a low-flow year, the plant would typically not raise the river temperature by more than 0.1°C above the applicable standard, as shown in Figure 5-4. Effluent temperatures for 2010 and 2011 were the warmest for the records examined. The greatest impact will typically be in early November when plant flows increase but the effluent temperature is still somewhat elevated.

Given this information, it is likely that the City will not face a temperature limit that could not be met with the existing discharge. However, the final outcome of the temperature litigation and the setting of new standards does cause uncertainty related to future requirements.

Figure 5-4. Effluent temperature impact at 7Q10 flow for 25 percent of River

5.3.4 Cold Water Protection and Thermal Plumes

In addition to the thermal load limits, water quality criteria have been set to protect fish including spawning habitat. The South Santiam River at Sweet Home is designated as spawning habitat and is subject to the requirements of the rule related to protection of cold water. The following additional limits are relevant to the WWTP discharge:

- If the rolling 60-day average maximum ambient water temperature, between the dates of spawning, is 10°C to 12.8°C , the allowable increase is 0.5°C above the ambient.
- If the rolling 60-day average maximum ambient water temperature, between the dates of spawning, is less than 10°C, the allowable increase is 1.0°C above the ambient.
- Impairment of an active salmonid spawning area where spawning redds are located or likely to be located is prevented or minimized by limiting potential fish exposure to temperatures of 13°C.
- Acute impairment or instantaneous lethality is prevented or minimized by limiting potential fish exposure to temperatures of 32.0°C.
- Thermal shock caused by a sudden increase in water temperature is prevented or minimized by limiting potential fish exposure to temperatures of 25.0°C (77.0°F) or more to less than 5 percent of the cross-section of 100 percent of the 7Q10 low flow of the water body.
- Unless the ambient temperature is 21.0°C or greater, migration blockage is prevented or minimized by limiting potential fish exposure to temperatures of 21.0°C (69.8°F) or more to less than 25 percent of the cross-section of 100 percent of the 7Q10 low flow of the water body.

Based on analyses of the data collected from the last 5 years, the discharge will not cause any thermal plume impacts identified above.

5.3.5 Toxics

WWTP effluent is discharged to the South Santiam River through a 14-inch-diameter outfall pipe and diffuser. The diffuser consists of a 48-inch-long spool piece at the end of the outfall pipe with eight 7-inch-diameter openings. However, the end of the diffuser was removed by City staff to reduce head loss at high flows. The resulting configuration is considered an open-ended outfall (i.e., no diffuser ports) by DEQ.

The current NPDES permit provides for an RMZ that extends from 10 feet upstream of the outfall to 100 feet downstream. The zone of immediate dilution (ZID) is within 10 feet of the outfall. DEQ conducted a mixing zone study of the outfall in August 2005 and presented an analysis of the mixing in a memorandum dated September 12, 2007. For the plant design dry weather design flow of 1.4 mgd, DEQ determined the following mixing values at 7Q10 river flows:

- Edge of the ZID: 4:1
- Edge of the RMZ: 27:1

The mixing values established by DEQ were used for evaluating reasonable potential for exceedance of water quality criteria.

Ammonia. DEQ has adopted new ammonia water quality standards based on the *Aquatic Life Ambient Water Quality Criteria for Ammonia—Freshwater 2013,* published by EPA. EPA approved this standard on August 4, 2015. Based on an analysis of water quality conditions in the summer, there will be no reasonable potential to violate the new ammonia water quality standard when mixing is considered.

Metals. Metals data have been obtained by the City based on the requirements of the permit. Based on these data, there is a potential for violating the aquatic life water quality standards for both copper and zinc, as shown in Figure 5-5.

Determine Monitoring Reqs.		Identify Pollutants of Concern					Determine In-Stream Conc.			Determine Reasonable Potential			
Pollutant Parameter	Evaluation Required?	# of Samples	Highest Effluent Conc.	Coefficent of Variation	Estimated Max Eff. Conc.	RP at end of pipe?	Ambient Conc.	Max Total Conc. at ZID	Max Total Conc. at RMZ	WQ CRITERIA 1 Hour (CMC)	WQ CRITERIA 4 Day (CCC)	Is there Reasonable Potential to Exceed? (Y/N) Acute	Is there Reasonable Potential to Exceed? (Y/N) Chronic
	(Y/N)		µg/l	Default=0.6	µg/l	(Y/N)	µg/l	µg/l	µg/l	µg/l	µg/l	Acute	Chronic
Table 2: Metals (total recoverable), cyanide and totoal phenols													
Arsenic (Total)	*	*	*	No Water Quality Criteria			*						
ARSENIC III	*	*	*	0.60	--	--	*	--	--	360.0	190.0	--	--
Cadmium	Yes	3	0.00	0.60	0.00	No	*	--	--	0.7	0.1	--	--
Chromium (total)	*	*	*	No Water Quality Criteria			*						
Chromium III	*	*	*	0.60	--	--	*	--	--	160.3	16.5	--	--
Chromium VI	*	*	*	0.60	--	--	*	--	--	15.7	10.6	--	--
Copper	Yes	3	166.00	0.60	647.40	Yes	0.00	161.85	23.98	4.1	2.5	YES	YES
Iron	*	*	*	0.60	--	--	*	--	--	--	####	--	--
Lead	Yes	3	0.00	0.60	0.00	No	*	--	--	11.6	0.3	--	--
Mercury	Yes	3	0.02	0.60	0.07	Yes	0.01	0.02	0.01	2.4	0.0	NO	NO
Nickel	*	*	*	0.60	--	--	*	--	--	126.3	11.0	--	--
Selenium	*	*	*	0.60	--	--	*	--	--	260.0	35.0	--	--
Silver	Yes	3	0.00	0.60	0.00	No	*	--	--	0.2	0.1	--	--
Zinc	Yes	3	51.00	0.60	198.90	Yes	0.00	49.73	7.37	31.5	24.9	YES	NO
Cyanide (Free)	*	3	9.00	0.60	35.10	Yes	0.00	8.78	1.30	22.0	5.2	NO	NO
Cyanide (Total)	*	3	9.00	No Water Quality Criteria			0.00						

Figure 5-5. Reasonable potential analysis for metals

Source: EPA

Sampling for the data shown in Figure 5-5 was not conducted using an ultra-clean technique. Experience has shown that when grab samples are collected and analyzed using ultra-clean techniques, results are typically lower than conventional procedures.

Because the water quality standard for copper will be changed by DEQ, initial action related to copper should be limited to additional data collection in anticipation of the revised standard.

For zinc, additional ultra-clean sampling may be warranted. If improved sampling does not mitigate the acute toxicity at the ZID, outfall diffuser improvements could be considered to improve the mixing at the ZID.

Human-Health Criteria. Based on the initial sampling of the Sweet Home effluent, there is no reasonable potential for violating any of the human-health criteria. However, because a measurable amount of total mercury has been reported, a mercury management plan (MMP)_will be required. The only organic parameter that has been detected in the effluent is chloroform, which is orders of magnitude below the human-health criteria.

5.4 Permit Limits

The existing NPDES permit discharge limits and requirements are summarized in Table 5-2.

Table 5-2. Existing Discharge Requirements

Parameter	Average effluent concentration (mg/L)		Mass load limits (ppd)		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
May 1–October 31					
CBOD$_5$	10	15	120	180	240
TSS	10	15	120	180	240
November 1–April 30					
CBOD$_5$	15	23	290	460	630
TSS	20	30	350	520	690
Other parameters (year-round unless otherwise noted)					
pH	Shall be within the range of 6.3–9.0.				
Ammonia-N (May–October)	Shall not exceed a monthly average concentration of 5.1 mg/L and a daily maximum concentration of 11 mg/L.				
CBOD$_5$ and TSS removal efficiency, May–October	Shall not be less than 85% monthly average.				
CBOD$_5$ and TSS removal efficiency, November–April	Shall not be less than 70% monthly average.				
Total chlorine residual	Shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum concentration of 0.05 mg/L.				
E. coli bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.				

5.4.1 Seasonal Percent Removal

The removal efficiency standard of 70 percent from November through April is vitally important to the City's ability to maintain compliance. Though substantial I/I reduction has been achieved, significant non-excessive I/I remains (see Section 7 for a comparison of the cost of treatment alternatives versus further I/I reduction). For example, using the 2015 MMWWF of 4.03 mgd and the annual average loads of 1,834 ppd of CBOD and 2,017 ppd of TSS (see Table 3-3), influent concentrations of 54 mg/L for CBOD and 60 mg/L for TSS result. Achieving 85 percent removal year-round would require effluent limits of 8 mg/L for CBOD and 9 mg/L for TSS.

5.4.2 Mass Load Limits

As shown in Table 4-6, mass loads discharged by the WWTP are at times higher than permit limits. This is because the WWTP treats flows much higher than the dry weather design capacity. This fact was recognized in the MAO, which contained the following exception to mass limits: "On any day that the daily flow to the treatment facility exceeds 2.76 MGD (twice the average dry weather design flow of 1.38 MGD), the daily mass load limit shall not apply…."

While the City has made substantial progress in reducing peak flows (as documented in Section 4 and Appendix C), further I/I reduction is not practical. Therefore, sustained high flows, well beyond twice the average dry weather design flow, will continue at the WWTP and, on occasion, the WWTP will not be able to comply with the mass effluent limits. Table 5-3 demonstrates the resulting restrictive treatment impact of the sustained high flows by showing the effluent discharge concentrations the WWTP would need to meet from November through April to stay within the permitted mass limits. These concentrations are based on the flows presented in Table 3-2.

Table 5-3. Required Effluent Concentrations to Meet Mass Load Limits at 2015 Flows

Parameter	Concentrations, mg/L			
	AWWF	MMWWF	Max week	PDF
$CBOD_5$	14	9	9	8
TSS	17	10	10	8

Even at average wet weather flow, the resulting 14/17 CBOD/TSS effluent quality is more restrictive than the permitted concentration limits of 15/20. However, the effective required concentration limit to meet permitted mass limits ratchets down substantially at higher flows. It culminates in an effective limit of 9/10 compared to the permit limit of 23/30 for maximum week flows and an effective limit of 8/8 for the peak day flow.

As the city grows, the concentrations required to meet mass loads will become even more stringent. Table 5-4 shows the required effluent concentrations based on projected 2040 flows.

Table 5-4. Effluent Concentrations to Meet Mass Load Limits at 2040 Flows

Parameter	Concentrations, mg/L			
	AWWF	MMWWF	Max week	PDF
$CBOD_5$	12	8	8	7
TSS	14	9	9	7

The WWTP will not be able to meet these limits consistently, especially during extended high flow conditions.

The following summary of prior informal correspondence with DEQ is provided as background for future discussions with DEQ during permit renewal on revising mass load limits from November through April. As a part of that process, DEQ has indicated that a demonstration must be made that any proposed adjustment satisfies DEQ's Antidegradation Policy described in OAR 340-041-0004(9):

- This is a modification that will require completion of an anti-degradation review worksheet for a proposed individual NPDES discharge .
- A "Treatment Capabilities Report" may be required.

- Modifications to the treatment plant biological process could result in the plant being deemed a "new facility" such that new loads could be calculated for future permit limits.
- Effluent concentration limits are not expected to change with future permit modifications.
- Wet season flows should be based on 2-year recurrence event. The daily 2-year recurrence flow is approximately 8.5 mgd as determined for the December 17, 2015 storm and overflow event. The monthly 2-year recurrence flow can be approximated as the MMWWF from Table 3-2. And the weekly 2-year recurrence flow is roughly interpolated between the two at about 6 mgd.
- Table 5-5 shows potential mass load limits based on wet weather concentration limits and the estimated flows above.
- Mass load limits should be reviewed for potential adjustments during the wet-weather treatment modifications at the plant.

Table 5-5. Proposed Mass Load Discharge Requirements for November through April

Parameter	Average effluent concentration (mg/L)		Mass load limits (ppd)		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
Flow for calculating proposed limits (mgd)			4.03	6	8.5
November 1–April 30 (existing/ ***proposed***)					
$CBOD_5$	15	23	290/ ***504***	460/ ***1,150***	630/ ***1,063***
TSS	20	30	350/ ***664***	520/ ***1,001***	690/ ***1,418***

A second important fact recognized in the MAO that is missing in the permit is that high flows should be expected in May. Using the maximum month dry weather flow of 2.68 mgd from Table 3-2, and the lower mass limits in the permit in effect from May through October, the resulting effective required effluent concentration is 5/5. Likewise, the WWTP will not be able to consistently meet this limit.

Similar to the above discussion for wet season mass limits, the City and DEQ will have an opportunity to modify this restrictive mass load effluent limit for May during the next NPDES permit renewal process. As a part of that process, DEQ has indicated that a demonstration must be made that any proposed adjustment satisfies DEQ's Antidegradation Policy described in OAR 340-041-0004(9).

The permit renewal process may require additional studies beyond the scope of this Facility Plan.

Again, the following summary of prior informal correspondence with DEQ is provided as background for future discussions with DEQ during permit renewal on including May in the wet season mass limits:

- This is a modification that will require completion of an anti-degradation review worksheet for a proposed individual NPDES discharge.
- Historical climate data for the month of May should be collected and presented to show how May is typically a more seasonably wet month than summer months.

Precipitation data for the months of May through October for the last 47 years in Sweet Home are shown in Table 5-6. Monthly average and standard deviation are presented. From this data, it is apparent that May is much wetter on average with a wider range than any other "dry season" month except October. But May is very different than October as it follows the wet season when the ground is still saturated and groundwater levels are higher, resulting in higher WWTP flows than October, which follows the driest months.

Table 5-6. Sweet Home (Foster Dam) Monthly Rainfall Records, 1969-2016		
Month	Average monthly rainfall, inches	Standard deviation, inches
May	3.74	2.15
June	2.51	1.43
July	0.70	0.82
August	0.97	1.13
September	1.96	1.61
October	4.27	2.45

5.4.3 Ammonia

Based on information presented in a letter from DEQ on February 18, 2016 (Appendix D), the ammonia-N limit has been removed without permit modification as a result of the new (August 4, 2015) standard adoption. During the NPDES permit renewal process, the City will be required to complete a reasonable potential analysis for ammonia toxicity using the new ammonia criteria. If the analysis indicates toxicity above the standard, ammonia limits may be required in the renewal permit.

5.5 Aging Infrastructure and Deficiencies

This section addresses aging infrastructure and provides details of unit process condition, performance, and remaining useful life. Details regarding the existing system's ability to meet current and future effluent limits and other regulatory requirements are provided. Improvements needed to meet existing and future needs are summarized.

5.5.1 Existing IPS

Erosion and cracking of the above-grade structure can be observed at the walls and roof. The exterior walls appear weathered and aggregate is exposed at multiple locations. The underside of the roof slab leaks as evidenced by moisture stains permeating through cracks. Figure 5-6 shows photos of the IPS interior.

Figure 5-6. Existing IPS interior

Because of poor inlet conditions, sand, grit, and debris tend to accumulate in the wetwell and there is no means to isolate a portion of the wetwell for cleaning.

The firm capacity of the existing IPS is approximately 6 mgd. Capacity of approximately 9 mgd is possible with all three pumps in service. A firm capacity of 13.5 mgd is required for meeting future conditions.

The physical size and configuration of the pump station will not allow for a significant capacity expansion. The existing 12- and 16-inch-diameter force mains do not provide capacity for conveyance of the regulatory peak flow at reasonable velocities. The existing pumps were installed as part of the 1993 upgrade with impeller upgrades made more recently. There is no influent flow meter.

The existing IPS is insufficient for conveyance of peak flows through the WWTP. Because of its age and condition, a new IPS is recommended.

5.5.2 Influent Flow Sampling

Influent sampling is done at the IPS wetwell using an automated, 24-hour composite sampler with sampling interval set on a timer (in the absence of an influent flow meter). Process return flows including filter backwash, solids holding tank decant, and dewatering filtrate are all returned to the IPS, which potentially results in double counting of these process streams depending on timing of the sample. Figure 5-7 shows the sampler unit used in the IPS wetwell.

Figure 5-7. Composite sampler at IPS wetwell

A new sampling location should be integrated with a new IPS that allows for influent sampling upstream of in-plant recycle streams. This will require a small in-plant pump station for conveyance of recycle process streams.

5.5.3 Preliminary Treatment

The existing manual bar screens do not effectively remove debris and rags that can pass through the relatively wide bar spacing. The screenings must be manually removed, requiring several checks per day by plant staff. Plastics and rags tend to collect in downstream processes, causing damage to equipment and requiring expensive maintenance activities. There are no grit removal provisions allowing grit to accumulate in the aeration basins and solids holding tank. There are no provisions for splitting flow to a parallel treatment process proposed for treatment of flows beyond 7 mgd. Figure 5-8 shows the existing headworks and routine cleaning of the manual bar screen.

Figure 5-8. Routine cleaning of headworks bar screens

A new headworks facility with a mechanical bar screen, manually cleaned bypass channel, and a screenings washer/compactor is recommended to effectively remove material that accumulates in downstream structures and results in significant additional O&M costs.

5.5.4 Aeration Basins

The existing aeration basins exhibit minor structural deficiencies that could be remedied. Structural deficiencies include erosion of interior surfaces consistent with age, minor cracking at fillets, and evidence indicating possible rebar corrosion in select areas close to the concrete surface. Aeration basin 2 has a 1-inch gap in the upper portion of one wall at the location of a construction joint but no apparent leakage.

The platform-mounted surface aerators are in fair shape commensurate with their age. Two of the aerators are original and two were salvaged from a neighboring plant. The aerators splash to the extents of the tank and are inefficient in terms of both oxygen transfer and pounds of oxygen delivered per installed horsepower. The aerators currently do not have adequate capacity to maintain the recommended oxygen residual in the tanks under moderate to high influent loading conditions.

Flow split from the aeration basins to the secondary clarifiers is controlled by weir boxes in the downstream ends of the tank. The weir boxes, installed as part of the 1993 upgrades, are apparently set at different elevations, resulting in the inability to effectively split flow between the three secondary clarifiers. As a result, a disproportionate share of flow is conveyed to the older and smaller clarifiers rather than to the newer, larger, and more effective third clarifier.

The existing aeration basins do not have feed or flow management provisions for conserving solids under high flow conditions. These types of provisions would include multiple feed points for raw influent and RAS and partitions to allow concentration and conservation of mixed liquor suspended solids (MLSS). The basins also lack adequate volume needed to effectively provide longer solids retention time (SRT) in dry weather to enhance ammonia removal. Figure 5-9 shows photos of the aeration basin interior.

Figure 5-9. Interior of aeration basin

Improvements to the existing basins should include resetting the hydraulic control points, a fine-bubble air diffuser system, blowers and piping appurtenances for aeration, and rehabilitation of concrete defects.

5.5.5 Secondary Clarifiers

The concrete in both of the original secondary clarifiers shows the expected signs of aging but is generally in good or repairable condition. Both original clarifier mechanisms and perimeter weirs are in need of replacement. Scum and floating debris tend to quickly collect on the center feed tub of the original clarifiers, requiring frequent manual removal. Scum and floatable solids tend to accumulate on the surface of the older clarifiers.

The original clarifiers are shallow by modern design standards and are not as well suited for treatment of high flows compared to clarifiers with deeper side water depth. The third clarifier, constructed as part of the 1993 upgrades, is larger and deeper than the two original clarifiers and generally in good operating condition. Figure 5-10 shows one of the original clarifiers and the newer, larger clarifier 3.


Figure 5-10. Original secondary clarifier 1 (left) and newer clarifier 3 (right)

Currently, the ability to split flow between the three clarifiers is limited by the configuration and elevations of the effluent structures and the tendency is to hydraulically overload the two smaller clarifiers. Improvements to the flow control structures at the downstream ends of the aeration basins, described later in this section, would provide the ability to balance flow between the three clarifiers.

Figure 5-11 shows the condition of the weirs and effluent trough and the drive gear of one of the older clarifiers.

Improvements for the two older clarifiers would include new clarifier mechanisms and appurtenances, new overflow weirs, and rehabilitation of concrete defects. No improvements are recommended for the newer clarifier.


Figure 5-11. Original secondary clarifier weir (left) and drive mechanism (right)

5.5.6 RAS and WAS Pumping Facilities

There is considerable leakage from the original piping and valves resulting from leaky gaskets and advancing age. The original RAS pumps serving clarifiers 1 and 2 are not metered. The three-way plug valves used to distribute RAS between the pumps leaks, which can airlock the pumps. There are no provisions for automatically pacing RAS pumping to plant flow or to measure the depth of the sludge blankets in the secondary clarifiers. Peak RAS capacity should be increased to more effectively manage the sludge blanket depth in the secondary clarifiers and avoid washout of solids under high flow and load conditions.

The WAS pumps are not variable-speed and pump at too high a rate to maintain a consistent MLSS concentration. There are no provisions for WAS metering or automatic control of wasting through supervisory control and data acquisition (SCADA). A check valve associated with the piston-type WAS pump tends to stick open, which allows mixed liquor to siphon from the clarifiers to the solids holding tank if the line is inadvertently not isolated between pumping cycles. Figure 5-12 shows the existing RAS and WAS pumps.

Figure 5-12. RAS (left) and WAS (right) pumping equipment

Improvements needed for solids pumping include increasing RAS pumping capacity, flow metering, optimization and automation of WAS pumping, and piping and valve improvements.

5.5.7 Secondary Effluent Distribution

Distribution of secondary effluent is facilitated at a flow distribution structure constructed as a part of the 1993 upgrades. The distribution structure receives flow from the three secondary clarifiers and conveys it to the filters, the CCT, or to a combination of both.

Conveyance of flow to the filters is possible from any combination of the three clarifiers, but effective flow split is not possible under high flow conditions. When flows exceed 4 mgd, secondary effluent from newer clarifier 3 will be conveyed directly to disinfection thus bypassing filtration. Flows above 4 mgd conveyed through the two smaller clarifiers can bypass the filters through a gated passage in the flow distribution structure. The three slide gates in the distribution structure are motor-actuated. Despite limited flexibility, no modifications are recommended to the secondary effluent distribution structure.

5.5.8 Final Effluent Distribution

Tertiary effluent along with secondary effluent beyond 4 mgd not sent to the filters is conveyed to the CCT through a flow-splitting box that can direct flow to either of the CCTs or to both tanks simultaneously. The structure is undersized for peak flows and is a hydraulic pinch point, causing the secondary clarifiers to back up under high flow conditions. The hydraulics of this structure and possible improvements should be investigated during predesign of plant improvements. The CCT flow distribution box is shown in Figure 5-13.

Figure 5-13. Secondary flow distribution structure (left) and CCT flow distribution box (right)

5.5.9 Filtration

The existing sand gravity filters (Figure 5-14) and pump station were constructed in 1993 to replace the original pressure filtration system. No structural deficiencies are evident and the filters and pump station are generally in good condition. The filters are sized to operate most efficiently at 2 mgd and have a peak capacity of approximately 4 mgd based on the maximum recommended loading rate and existing pumping capacity.

Figure 5-14. Gravity filters with sand media

The primary deficiency associated with the existing sand filters and appurtenances is a lack of capacity to treat flows beyond 4 mgd and decreasing removal efficiencies beyond flow rates of 2 mgd. No improvements are recommended at this time.

5.5.10 Disinfection and Dechlorination Equipment

Performance of the existing disinfection and dechlorination system is generally good, but the entire system should be evaluated during the predesign period and updated as required. Figures 5-15 and 5-16 show components of the dechlorination and disinfection systems, respectively.

Figure 5-15. Sodium hypochlorite feed equipment (left) and storage tank in containment area (right)

Figure 5-16. Sulfur dioxide feeding and analytical equipment

The use of peracetic acid (PAA) for disinfection should be considered and pilot tested during prede-sign. The use of PAA has several possible advantages over the use of sodium hypochlorite and would eliminate the need for sulfur dioxide feeding.

5.5.11 Chlorine Contact Tank

CCT deficiencies are related primarily to a hydraulic bottleneck at the flow distribution structure and less than optimal detention volume associated with conveyance of higher flows. The CCT is shown in Figure 5-18.

Figure 5-17. Two views of the CCT

Improvements to the CCT would consist of modifications to eliminate hydraulic restrictions and evaluation of the existing chemical mixing provisions.

5.5.12 Solids Holding Tank

Solids holding tank deficiencies include inadequate mixing and aeration capacity. Odors are generated in the tank because of pockets of material under anaerobic conditions. Insufficient mixing results in the accumulation of grit and heavy solids on the bottom of the tank, which reduces storage capacity and damages the diffuser grid over time. Poor mixing also reduces dewatering performance due to excessive variability in the concentration of the dewatering feed solids, making polymer addition during dewatering imprecise. Rags accumulating in the tank have caused extensive damage to the coarse-bubble diffuser grid as recently as 2014. Periodic cleaning of the tank or repair of the diffuser grid requires a temporary means to dispose of WAS.

Figure 5-18 shows the exterior of the solids holding tank and blowers.

Figure 5-18. Solids holding tank exterior (left) and blowers (right)

Improvements for the solids holding tank would include a new coarse-bubble diffuser grid, new blowers, and air piping.

5.5.13 Dewatering Facilities

The existing BFP has a capacity of approximately 35 gpm and dewaters the feed material to about 13 percent concentration on average. The BFP is more than 20 years old and nearing the end of its expected life; the manufacturer is no longer in business. Appurtenances include an in-line grinder, air-actuated diaphragm feed pump, polymer conditioning and feed system, and dewatered cake conveyor.

Figure 5-19 shows the existing dewatering equipment.

Figure 5-19. Solids feed pump, BFP, and conveyor

A new BFP or screw press is recommended with additional capacity and better dewatering performance. Dewatering appurtenances should be evaluated and upgraded if appropriate at the time the dewatering device is replaced. The interior of the building should be updated as part of this project.

5.5.14 Solids Stabilization

Solids stabilization is not currently practiced at the WWTP. It is assumed that solids will continue to be disposed of in a manner not requiring stabilization. There are no recommendations for new solids stabilization facilities.

Section 6

Alternatives Considered

This section evaluates possible alternatives to address WWTP deficiencies identified in Section 5. Alternatives for liquid stream alternatives will be presented first. Alternatives for treatment of solids will be presented later in this section.

6.1 Liquid Stream Alternatives

The existing WWTP can effectively convey and treat up to 7 mgd but requires a peak hydraulic capacity of approximately 13 mgd, which corresponds to the projected 5-year recurrence peak flow event. Addressing regulatory limits will require a means to convey and manage peak flows, screening of influent, additional secondary treatment capacity, and general improvements to upgrade or improve facilities. Three general alternatives were considered for addressing the deficiencies:

- *Alternative 1: further reduce I/I in the collection system.* Establish additional I/I reduction program to reduce peak flows to a level that can be conveyed and treated by the existing WWTP.
- *Alternative 2: flow equalization/storage.* A storage facility would be sized to equalize peak wet weather flows (PWWF) to a level that can be conveyed and treated by the existing WWTP.
- *Alternative 3: WWTP upgrade.* Upgrade the existing treatment facility to increase both hydraulic and treatment capacity.

The listed alternatives are not necessarily mutually exclusive. Alternative 1 or 2 also require improvements to the treatment facilities at the WWTP. Conversely, Alternative 3 requires an expansion of both hydraulic and treatment capacity to treat the full range of existing and projected flows.

Planning-level costs presented in this section are estimates of the cost to construct or modify each of the affected processes and are only for comparison. These costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Section 8 presents total project costs (including the aforementioned items) for the selected alternative. Table 6-1 summarizes the comparative costs for the alternatives. WWTP costs are separated into costs common to all alternatives to treat 7 mgd and incremental costs to treat up to 13 mgd of peak flows for Alternative 3. Alternative 3C is the lowest cost by a wide margin.

Table 6-1. Comparative Construction Cost Summary for Liquid Stream Alternatives[a]

Alternative	Description	Collection System Costs, ($)	WWTP Costs to Treat 7 mgd, ($)	WWTP Added Costs to Treat Peak Flows up to 13 mgd, ($)	Total
1	Further reduce I/I	28M	10M	0	38M
2	Flow equalization/storage	28M	10M	0	38M
3a	Parallel secondary process	0	N/A	N/A	N/A
3b	MBR with HRC	0	20.1M	6.7M	27M
3c	Upgrade existing and add HRC	0	12.2M	6.7M	19M

a. *Costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Biosolids improvements are also not included here. See Section 8 for total project costs.*

Cost information for the selected alternative, and the basis of the estimate for common and specific WWTP upgrade elements are presented in Appendix F. Each cost was developed using standard cost estimating procedures using layouts, equipment quotations, and unit costs based on a November 2015 *Engineering News-Record* (ENR) index.

Each alternative, and its costs, are discussed in greater detail below.

6.1.1 Alternative 1: Further Reduce I/I in the Collection System

Based on peak flow projections, approximately 6.3 mgd of additional flow is required to be removed in order to meet the WWTP's existing capacity of 7 mgd.

Successful implementation of this alternative assumes that I/I within the collection system could be cost-effectively reduced beyond what has already been achieved by the four-phase collection system rehabilitation project described in Section 4. Reducing peak flows to this level would eliminate the need for a parallel conveyance and treatment facility, but would not eliminate the need for liquid stream improvements required to increase secondary treatment capacity and reliability of the existing facilities.

Table 6-2 lists the estimated rehabilitation costs associated with additional I/I removal projects and the expected reduction in peak RDII associated with each project.

Table 6-2. Future R&R Work Cost Effectiveness

Sanitary basin(s)[a]	Type of R&R	Cost of remaining R&R work ($)	Peak RDII removed[b] (mgd)	Cost-effectiveness, ($/gallon RDII removed)	Rank
1	Full rehabilitation, complete uppers	1,620,000	0.18	9.0	12
2, 19	Complete uppers	310,000	0.17	1.8	1
3	R&R work complete	0	0	0	N/A
4	Complete uppers	820,000	0.14	5.7	7
5, 6, 21	Complete uppers	970,000	0.39	2.5	2
7, 13, 14, 17	Full rehabilitation	7,350,000	1.55	4.7	6
8	Full rehabilitation, complete uppers	2,720,000	0.28	9.9	13
9	Full rehabilitation, complete uppers	910,000	0.29	3.1	4
10	Full rehabilitation, complete uppers	2,990,000	0.42	7.1	11
11, 12	Full rehabilitation	3,770,000	0.53	7.1	10
15	Full rehabilitation	2,130,000	0.31	6.8	8
16	Full rehabilitation	2,520,000	0.58	4.4	5
18	Full rehabilitation	1,130,000	0.37	3.1	3
20	Complete uppers	630,000	0.09	7.0	9
Total		**$27,900,000**	**5.30**	**5.3**	

a. *Basins grouped together because of flow monitoring locations and model calibration methodology.*

b. *Assumes 65% reduction in RDII for full rehabilitation, 30% reduction for completing uppers.*

This I/I effort is costly and falls 1 mgd short of the 6.3 mgd reduction needed to treat wet weather flows expected at the WWTP. WWTP costs to more reliably treat 7 mgd (common to all alternatives) are also required. Costs to treat 1 mgd of wet weather flows are ignored as they are not significant for this analysis.

Table 6-3 summarizes the total cost of this alternative and includes; RDII costs, WWTP upgrade costs for 7 mgd, and WWTP costs for wet weather improvements.

Table 6-3. Summary of Construction Costs for Alternative 1

Component	Cost ($)
Peak RDII removal	
Remaining R&R work	28M
WWTP Upgrades for 7 mgd	
Influent pumping	3.3M
Mechanical Bar Screen (1 screen)	0.5M
Aeration improvements for existing basins	0.6M
Secondary clarifier improvements	0.5M
Tertiary filtration	3.3M
Existing CCT and disinfection improvements	0.1M
Outfall improvements	0.4M
Civil site work	0.5M
Miscellaneous improvements	0.5M
Standby generator	0.3M
WWTP Upgrades for 13 mgd	N/A
Total	**$38M**

6.1.2 Alternative 2: Flow Equalization Storage

A storage basin could be constructed for detention of flows exceeding the hydraulic capacity of the existing treatment system. Once high flows subside, the stored volume would be pumped or otherwise conveyed to the WWTP for treatment, thereby providing storage for the next storm event. A storage facility would require a separate pumping system, either for filling an above ground tank or for emptying a below-ground structure. Similar to Alternative 1, flow equalization can be reasonably assumed to eliminate the need for a parallel conveyance and treatment facility for flows exceeding 7 mgd, but would not eliminate the need for improvements to existing WWTP facilities required to more reliably treat 7 mgd.

To evaluate this alternative, results from the collection system predictive model described in Section 4 were used to quantify multiple-day storms in the historical record. The storm event occurring in late December 2005 was identified as one of the larger events occurring within the record, having flows in excess of 7 mgd over consecutive days. Flows in excess of 7 mgd resulting from this storm were used as the basis for evaluating the storage volume requirement. The analysis calculated the volume required for storage of this single storm event by assuming an empty storage basin at the beginning of the analysis. This analysis showed this event to have a 5-year recurrence storage volume requirement. An analysis of the entire rainfall record might determine that additional volume is required resulting from consecutive storms. The resulting hydrograph, existing WWTP hydraulic capacity, and resulting storage volume requirement are shown in Figure 6-1.

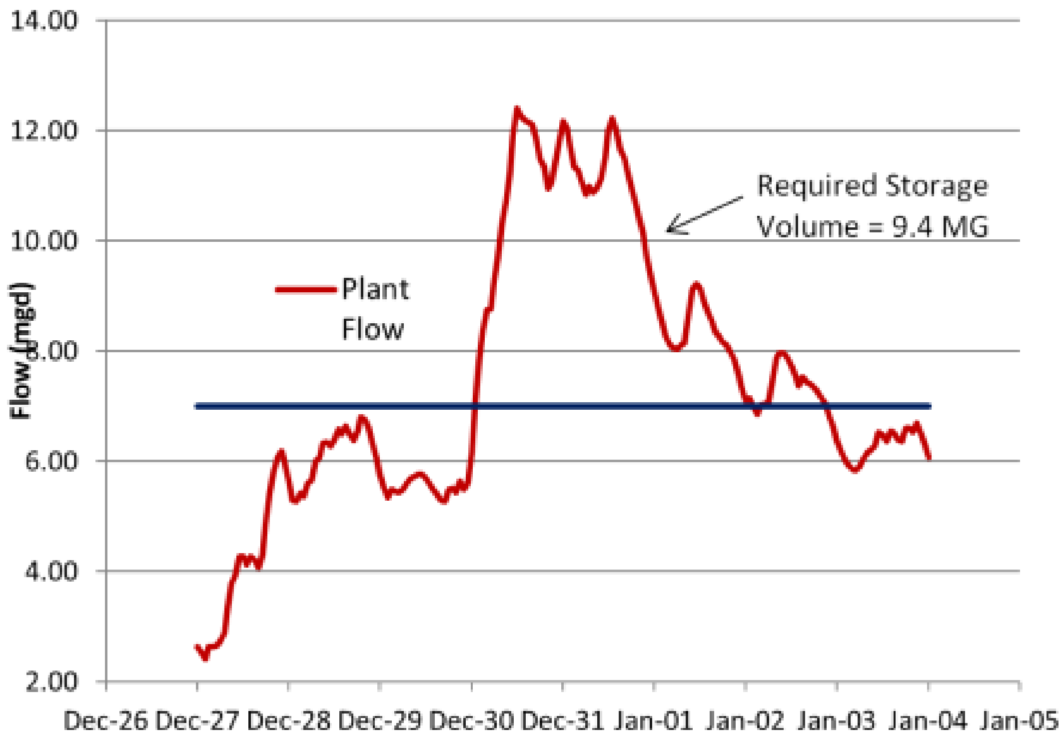


Figure 6-1. Storage requirement during wet weather event

Based on this evaluation, the storage basin volume would need to be approximately 9.4 million gallons (MG). Storage of this volume equates to a single below ground basin that is 14 feet deep and covers an area of approximately 2 acres or an above ground tank that is 50 feet tall and 180 feet in diameter. Storage tank construction costs generally range from $2 per gallon for above ground tanks to $6 or more per gallon for buried structures. Siting such a structure would also require land acquisition and permitting costs. Pumping, piping, valves, and controls would further add to the cost. Applying a conservatively low unit cost of $3 per gallon for such a flow equalization facility yields a capital expenditure estimate of approximately $28M.

It is expected that this alternative would not require wet-weather technology upgrades at the plant as the storage volume would be metered back to the plant during low spots in the diurnal cycle such that plant flows are limited to 7 mgd or below.

Table 6-4 below itemizes these costs for Alternative 2 and, as for Alternative 1, includes the costs for liquid stream improvements required to increase secondary treatment capacity and reliability of the existing facilities to handle 7 mgd.

Table 6-4. Summary of Construction Costs for Alternative 2

Component	Cost ($)
Flow Equalization Facilities	
Storage Tank	25M
Land acquisition	0.5M
Pumping	2M
Flow control structure/gates	0.5M
WWTP Upgrades for 7 mgd	
Influent pumping	3.3M
Mechanical bar screen (1 screen)	0.5M
Aeration improvements for existing basins	0.6M
Secondary clarifier improvements	0.5M
Tertiary filtration	3.3M
Existing CCT and disinfection improvements	0.1M
Outfall improvements	0.4M
Civil site work	0.5M
Miscellaneous improvements	0.5M
Standby generator	0.3M
WWTP Upgrades for 13 mgd	N/A
Total	**$38M**

6.1.3 Alternative 3: WWTP Upgrade

Alternative 3 incorporates three possible sub-alternatives for upgrading secondary treatment in addition to expansion or construction of new facilities for influent pumping, flow conveyance, disinfection, and solids handling. The three sub-alternatives for providing secondary treatment are discussed first. Additional elements common to all of the alternatives are discussed later in this section.

6.1.3.1 Secondary Treatment Improvements

Three options for conveying and treating the full range of projected flows and loads are described below.

6.1.3.1.1 Secondary Treatment Alternative 3A

Expand the existing aeration, sedimentation, and conveyance elements of the secondary treatment system to convey and treat the full range of flows.

At Sweet Home, the vast majority of flows are less than 3 mgd while the projected peaks range up to 13 mgd. Flows greater than 5 mgd are infrequent and dilute. The existing secondary treatment system is not capable of effectively conveying flows in excess of 7 mgd because of the size of hydraulic structures and interconnecting pipes. Increasing hydraulic throughput beyond this level will flood structures and promote clarifier failure, resulting in potential permit limit violations. Conveyance and treatment of up to 13 mgd would essentially require a parallel secondary treatment system that would be used infrequently.

Influent loadings are currently insufficient to sustain the biological process within the expanded volume year-round, and bringing the standby capacity online for peak flow events would take days to weeks to establish sufficient MLSS to enable effective treatment.

For these reasons, providing conventional secondary treatment for the full range of flows is not a viable alternative. As such no flow schematic or cost has been generated.

6.1.3.1.2 Secondary Treatment Alternative 3B

Construct a new secondary treatment system using membrane bioreactors (MBRs) and a parallel wet weather treatment system.

This alternative involves abandoning the existing aeration basins, secondary clarifiers, and sand filters in favor of a new MBR process for flows up to 7 mgd, and construction of a parallel wet weather treatment system for conveyance and treatment of flows exceeding 7 mgd. Other improvements required for this alternative include a new IPS, new headworks, parallel wet weather disinfection tank, and new outfall pipe and diffuser. The MBR process is discussed below. Other improvements associated with this alternative are described later in this section.

The MBR process uses a suspended-growth biological reactor and membrane filtration in lieu of secondary clarification and tertiary filtration. The process provides for high removal efficiencies of nitrogen, phosphorus, bacteria, BOD, and TSS. The high-quality effluent produced by MBRs makes them particularly applicable to reuse applications and for surface water discharge applications requiring nitrogen and phosphorus removal. The use of membranes for removal of solids eliminates the possibility of clarifier failure when treating high flows. Membrane filtration allows for a higher biomass concentration in the mixed liquor and use of smaller bioreactors when compared to conventional treatment.

MBR technology was evaluated for Sweet Home because it would replace the existing aeration basins and secondary clarifiers, both of which require upgrades and are shallow by modern standards. MBR would also remove the need to upgrade or replace the sand filters as MBR effluent quality is superior to even filtered effluent. Effluent produced by MBRs should meet future regulatory limits for TSS, BOD, and ammonia, and should be ideal for reuse of effluent for irrigation. Low concentrations of solids and bacteria in the MBR effluent also enable reliable disinfection and low chemical dosing rates.

The primary disadvantages of MBR systems are high capital and O&M costs when compared to conventional systems of similar capacity. Maintenance costs associated with MBR include routine membrane cleaning for fouling control and eventual membrane replacement. Energy costs are higher because the membranes require continuous air scouring and permeate must be continuously pumped or drawn through the membranes. In addition, the waste sludge generated by an MBR process may exhibit poor dewatering performance, resulting in the need for additional polymer for thickening and dewatering processes.

MBR tankage and support equipment would fit within the area adjacent to the existing WWTP in the area currently occupied by the City's maintenance facility. The plant would use a new IPS and headworks. Effluent would be disinfected in the existing CCT and discharged through an improved outfall.

For this alternative, a parallel wet weather treatment system using high-rate clarification (HRC) would be required to convey and treat flows greater than 7 mgd. Dilute influent conveyed through the HRC would be treated and disinfected prior to discharge. The HRC process is described in detail later in this section. A flow schematic showing the basic components of an MBR-based process for Sweet Home is shown on Figure 6-2 below. A layout showing the existing plant with the proposed improvements and associated yard piping is provided in Figure 6-3 below.

Table 6-5 lists the component construction costs for this alternative.

Table 6-5. Summary of Construction Costs for Alternative 3B	
Component	Cost ($)
WWTP Upgrades for 7 mgd	
Influent pumping	3.3M
Mechanical Bar Screen (1 screen)	0.5M
MBR facility	15.0M
Existing disinfection improvements and new WW CCT	0.1M
Outfall upgrade	0.4M
Civil work	0.5M
Standby generator	0.3M
WWTP Upgrades for 13 mgd	
Influent pumping capacity expansion	0.4M
Mechanical bar screen (2nd screen)	0.3M
Flow diversion pipe and structure	0.1M
Grit removal	2.0M
Wet weather treatment (HRC)	3.5M
Wet weather disinfection facility	0.4M
Total	**$27M**

6.1.3.2 Secondary Treatment Alternative 3C

Upgrade the existing secondary treatment system and construct a parallel wet weather treatment system.

This alternative makes use of the existing infrastructure to the highest degree possible and provides opportunities to phase some proposed improvements and defer capital expenditures. The primary elements of this alternative include a new IPS, new headworks, improvements to the existing secondary treatment process, third aeration basin, parallel wet weather treatment process and disinfection tank, and new outfall pipe and diffuser. The third aeration basin is discussed in detail below. Other improvements associated with this alternative are described later in this section.

Aeration Basin Improvements. The initially proposed upgrades include conversion of the existing surface aerators to a fine-bubble aeration system, and modification of the effluent weirs controlling the flow split between the secondary clarifiers. Peak hydraulic capacity of the secondary treatment system would remain at approximately 7 mgd, but plant performance would be extended by providing additional process air and by balancing the flow split between the three clarifiers. Replacement of the existing surface aerators with a fine-bubble aeration system would also reduce energy requirements for aeration.

When required based on flows and loads, a third aeration basin would be constructed and operated in series with the two existing aeration basins for additional treatment of BOD and ammonia loads, enhance settleability of MLSS, and provide for more effective management of biological solids during peak flow events. The third basin would be deeper than the existing basins to increase oxygen transfer efficiency. The new basin would be smaller in footprint and incorporate an anoxic selector zone to enhance sludge settleability, reduce aeration requirements, and recover alkalinity. Multiple feed points (i.e., step feed) for influent and RAS would be incorporated to enable a flow configuration capable of concentrating and conserving MLSS during peak flow events. The additional volume provided by the third basin would facilitate extending the MLSS concentration and sludge age as required for effective treatment of ammonia. Timing of the third basin would be determined by the performance of the two existing basins with aeration and hydraulic improvements and future load increases.

Implementation of these upgrades would help with treatment efficiency and solids inventory during wet weather events.

Secondary Clarifier Mechanism Replacement. This upgrade includes installation of new clarifier mechanisms in the two original secondary clarifiers with energy-dissipating baffles, deeper flocculating skirts, and improved effluent baffles. This upgrade is recommended to improve performance, enhance clarifier reliability, and reduce ongoing maintenance.

RAS and WAS Pumping Improvements. This upgrade would increase the RAS recycle capacity by approximately 100 percent to enhance treatment reliability during wet weather events. It would also appropriately size the WAS pumps and enhance their controls to optimize solids management. Currently, when high flow conditions exist for extended periods, the sludge blanket can accumulate to a point where it can overflow the weirs of the two original clarifiers. Increasing the RAS recycle capacity by approximately 100 percent is recommended to address this deficiency and enhance treatment reliability. Establishing a lower MLSS inventory by increasing WAS pumping during high flow events is currently practiced to reduce the rate of blanket accumulation and subsequent solids washout. Active management of the solids inventory would continue to be required until a third aeration basin can be constructed. The existing WAS pumps are oversized and therefore operated for only a few minutes every day. New WAS pumps, appropriately sized for the optimal wasting rate and fitted with VFDs, would enhance operations but could be deferred. Specific requirements for RAS and WAS pumping would be evaluated during predesign to arrive at the most cost-effective solutions for optimizing secondary solids management.

HRC. A wet weather treatment system using HRC would be required to convey and treat flows greater than 7 mgd. Dilute influent conveyed through the HRC would be treated and disinfected prior to discharge. The HRC facility is described in detail later in this section.

Table 6-6 below lists the component costs for this alternative. A flow schematic showing the basic components of an Alternative 3C for Sweet Home is shown on Figure 6-2. A layout showing the existing plant with the proposed improvements and associated yard piping are shown on Figure 6-5.

Alternative 3B

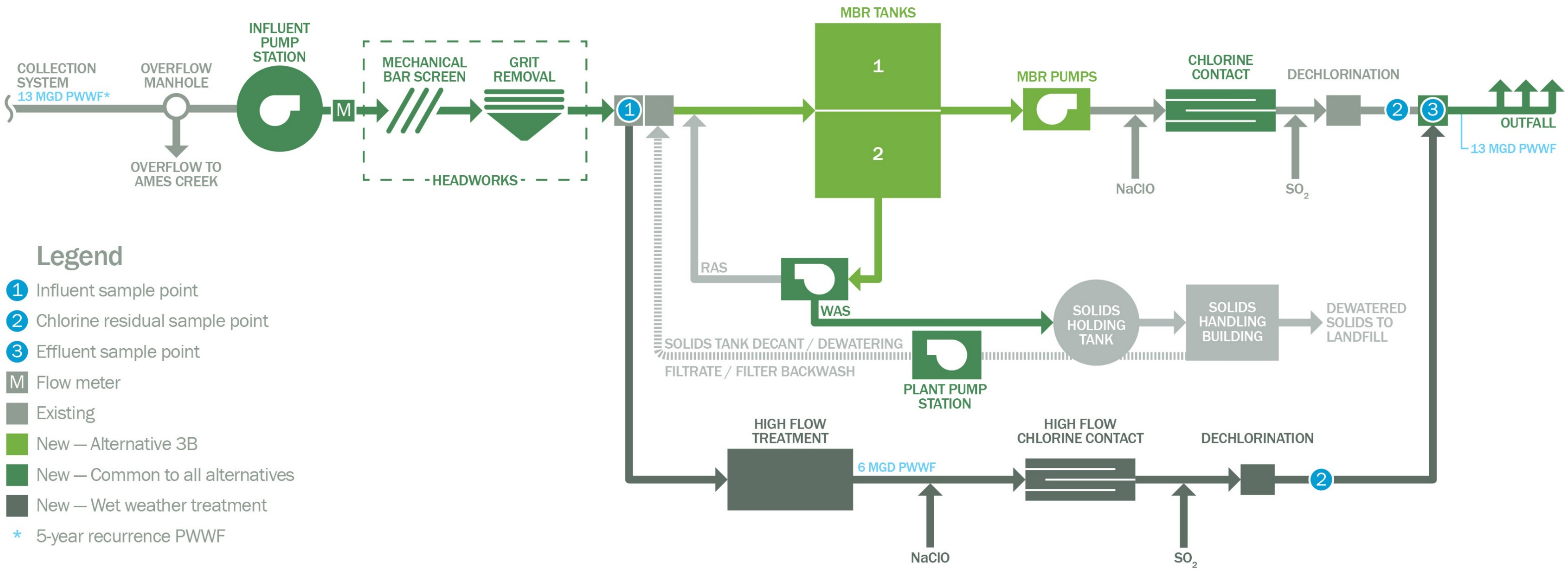


Figure 6-2. Alternative 3B MBR and HRC process schematic

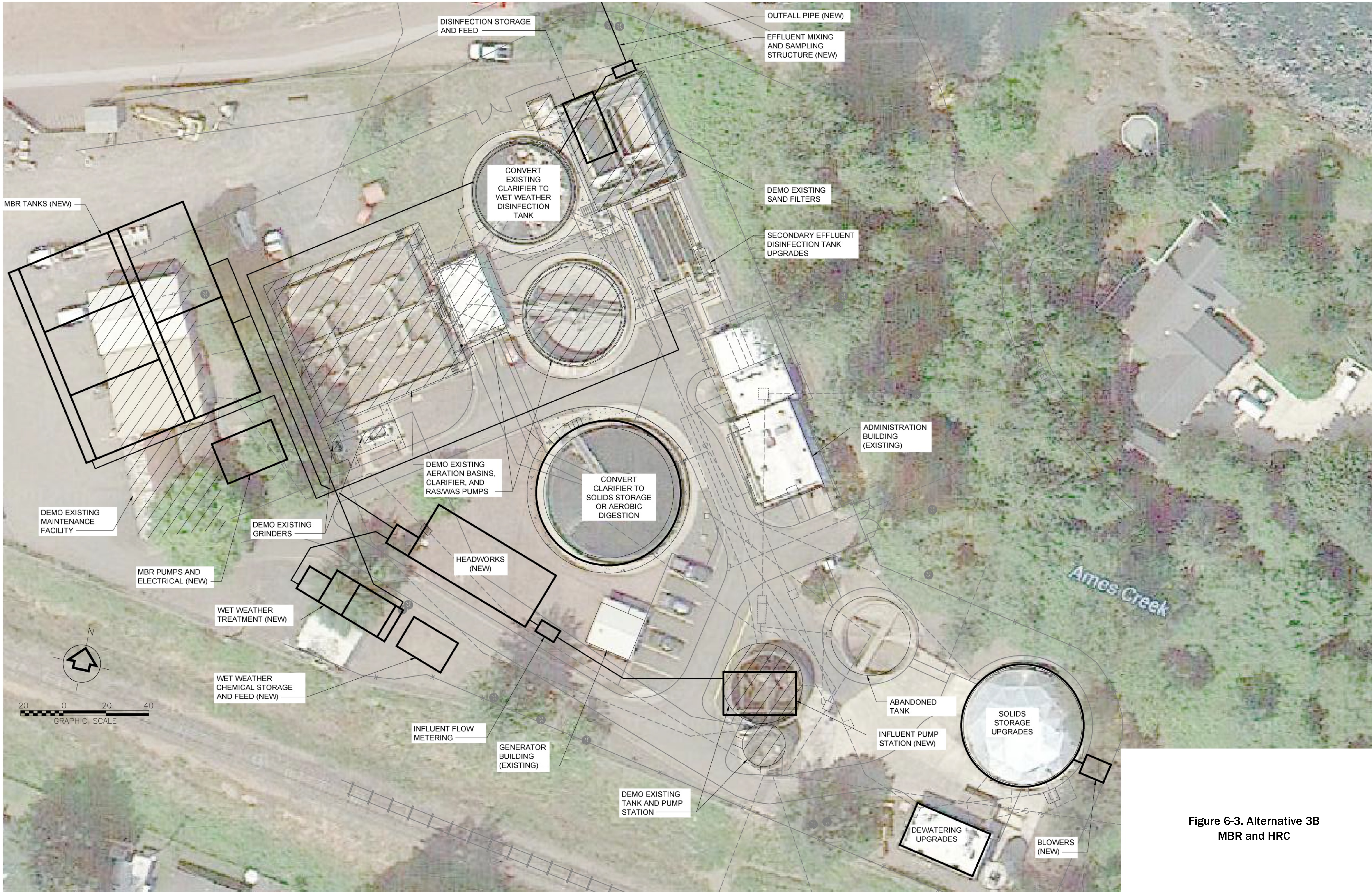


Figure 6-3. Alternative 3B MBR and HRC

Alternative 3C

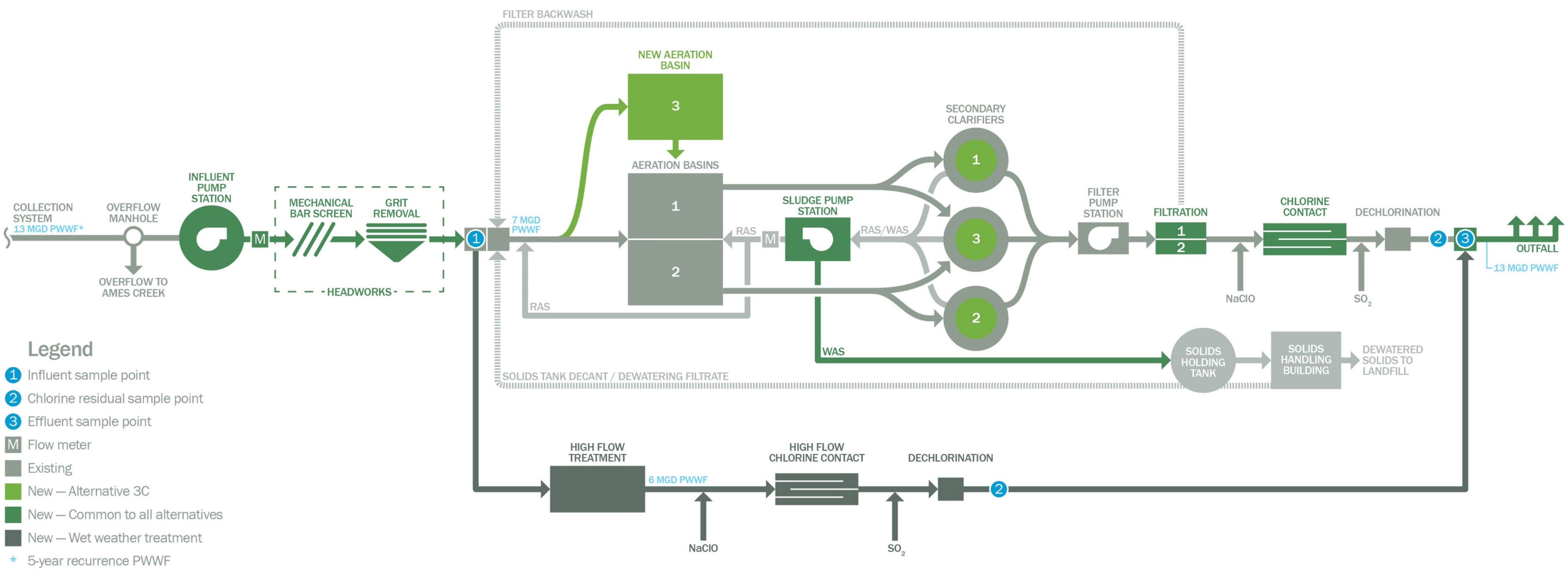


Figure 6-4. Alternative 3C Upgrade existing and add HRC process schematic

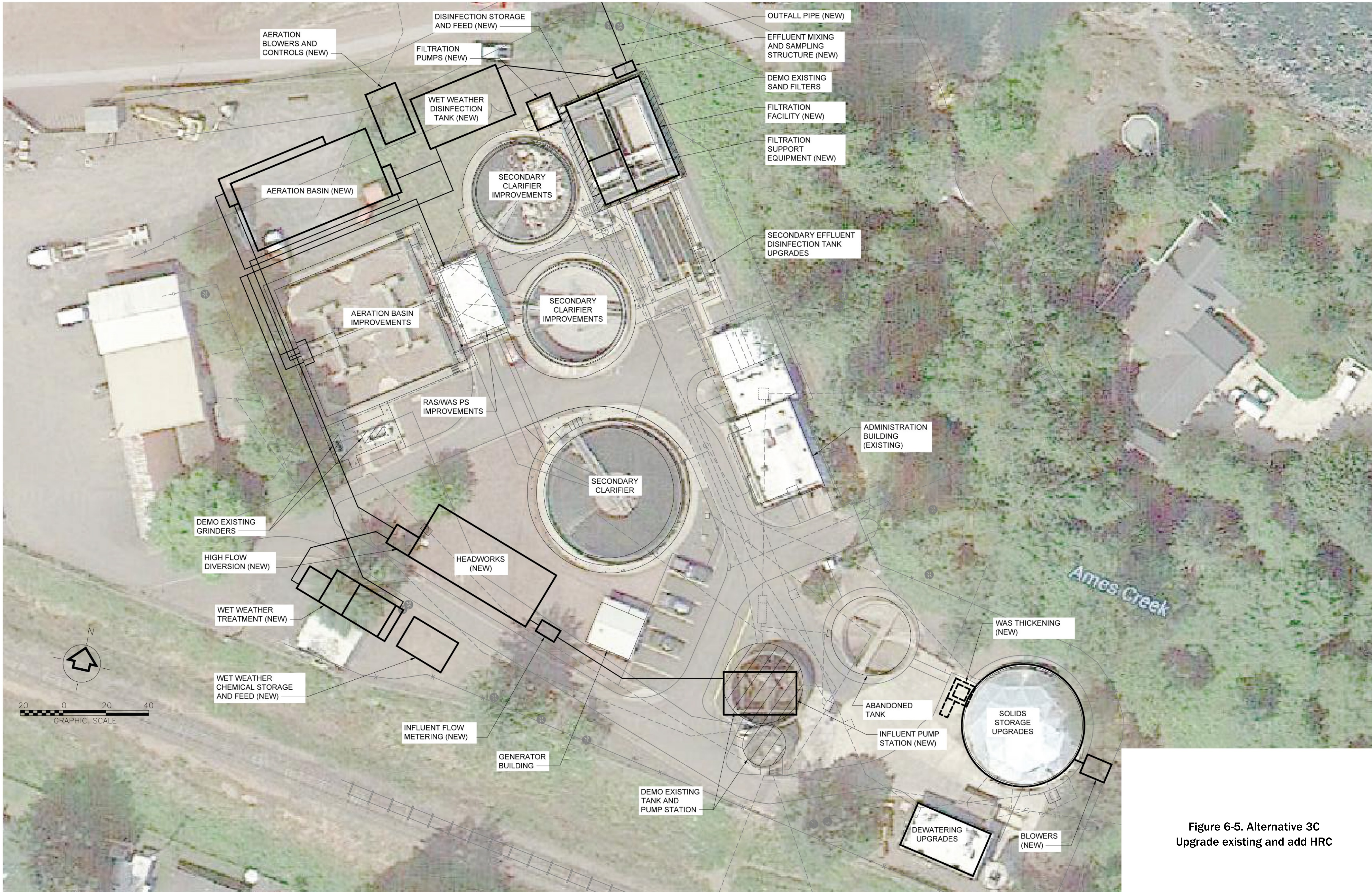


Figure 6-5. Alternative 3C
Upgrade existing and add HRC

Table 6-6. Summary of Construction Costs for Alternative 3C

Component	Cost ($)
WWTP Upgrades for Design Flows	
Influent pumping	3.3M
Mechanical bar screen (1 screens)	0.5M
Aeration improvements for existing basins	0.6M
Existing CCT and disinfection improvements	0.1M
New aeration basin	2.2M
Outfall improvements	0.4M
Secondary clarifier improvements	0.5M
Tertiary filtration	3.3M
Standby generator	0.3M
Miscellaneous improvements	0.5M
Civil site work	0.5M
WWTP Upgrades for Peak Flows	
Influent pumping capacity expansion	0.4M
Mechanical bar screen (2nd screen)	0.3M
Flow diversion pipe and structure	0.1M
Grit removal	2.0M
Wet weather treatment (HRC)	3.5M
Wet weather disinfection facility	0.4M
Total	**$19M**

6.2 Liquid Stream Alternative Common Elements

The following section discusses elements common to one or more of the listed alternatives.

6.2.1 Influent Pump Station

The existing IPS has a firm capacity of approximately 6 mgd, is showing signs of advanced aging, and suffers from excessive grit accumulation due to poor inlet hydraulics. The physical size of the pump station and other limitations described in Sections 4 and 5 make it impractical to sufficiently increase the peak capacity.

For adoption of Alternatives 2 and 3, additional pump station capacity is required to convey the projected peak flow through the WWTP and avoid overflows to Ames Creek. A new pump station, sized to provide a firm capacity of at least 13.5 mgd, is required for Alternative 3.

For cost estimating and layout purposes, a self-cleaning, trench-style pump station design was assumed using multiple submersible pumps. The most effective approach will usually incorporate one smaller pump providing service for low to average flow conditions and three larger pumps for providing the firm peak design capacity plus redundancy. The pump station would be constructed below grade with associated electrical gear located in either a new or existing building. All pumps would be equipped with VFDs.

The construction of a new IPS addresses the conveyance of planning-level flows and would be subject to a design sufficient to meet the reliability requirements set forth by the City and State.

6.2.2 Headworks

Removal of screenings would protect and extend the life of downstream equipment and reduce plastics in the solids stream, which is consistent with beneficial reuse options. Grit removal, when implemented, would reduce maintenance requirements associated with periodic cleaning of the aeration basins and solids holding tank and prevent grit-related damage to equipment.

New headworks consisting of both screening and grit removal facilities are planned. The initial phase of the headworks facility construction would consist of dual-channel headworks incorporating a mechanical bar screen and a screenings washer/compactor. For the purpose of sizing the headworks footprint and development of the cost estimate, a dual-channel facility incorporating one mechanical bar screen and a stacked-tray grit removal system was assumed. A stacked-tray grit removal system minimizes the facility footprint and optimizes grit removal performance. To defer construction of the grit removal element, the screening and grit removal structures will need to be designed and constructed as separate but adjacent structures. Specific details for the new headworks will be developed during predesign.

The addition of screenings and grit removal at the facility addresses many operational concerns that the plant currently faces. In addition, these facilities are required for the implementation of recommended wet weather treatment technologies.

6.2.3 Filter Upgrades

Tertiary treatment is implemented year-round at the WWTP to ensure that effluent meets the more stringent permit limits under dry weather conditions and to improve wet weather performance when effluent quality can be degraded by high hydraulic loading on the secondary clarifiers.

The existing sand filters and filter pump station are in good working order but have limited capacity. The filters are designed to operate most effectively at a loading rate of 2 gpm/ft^2 of surface area, which corresponds to a plant flow of approximately 2 mgd. The filters have a peak loading capacity of 4 gpm/ft^2, providing a filtering capacity of approximately 4 mgd. As expected, filter performance will be better at the lower end of the range. The filters can reduce effluent TSS to well below the dry weather limit of 10 mg/L when plant flows are low but do not have capacity to treat flows beyond 4 mgd. The filters can help reduce BOD that is connected to suspended solids, but does not biologically treat BOD. If wet weather TSS and BOD/CBOD discharge limits were maintained or tightened for wet weather flows, additional filtration capacity would likely be required.

Given the large footprint needed for gravity sand filters and the limited space available, the best option for providing additional filter capacity would be to install modular, high-rate filters in the area currently occupied by the sand filter units. High-rate modular filters having hydraulic loading rates in the range of 30 to 40 gpm/ft^2, or about 10 times the capacity per square foot as a sand media filter use synthetic media in a variety of configurations, from a variety of manufacturers. Modular units, each with capacity to treat up to 2.5 mgd, could be installed to provide filter capacity adequate to treat the entire range of flows. A modular unit manufactured by Schreiber is shown in Figure 6-6. The filter unit shown in the figure consists of four modular filters with a combined capacity of 10 mgd. The existing footprint of the filter facility would allow for installation of up to six modular units. The cost estimate developed for new filtration facilities is based on five modular units with firm capacity of 10 mgd and total capacity of 12.5 mgd.

Keeping the existing filters addresses mass loading permit requirements for select flow ranges at the WWTP. Installing a completely new filtration system capable of treating the entire planning-level flow range would come at a significant cost.

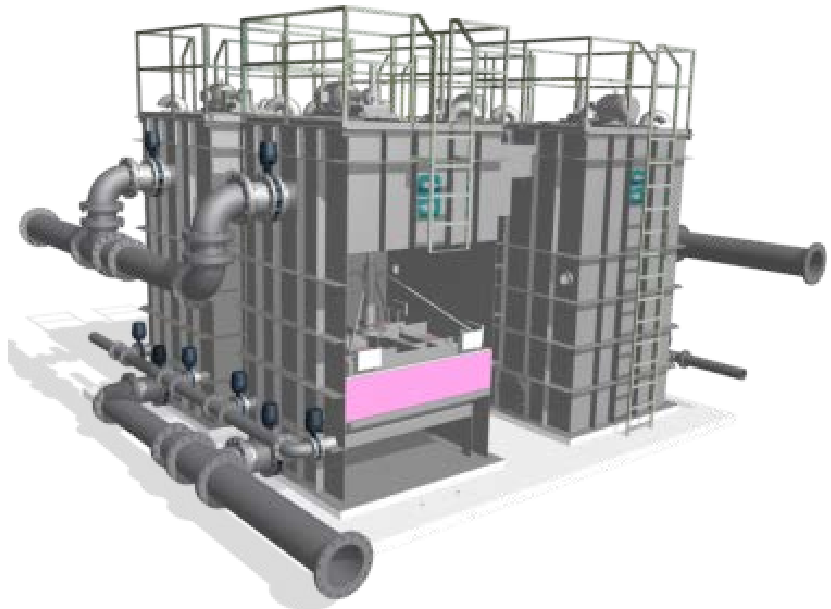


Figure 6-6. 10 mgd modular high-rate filter unit

Source: Schreiber

6.2.4 Disinfection Improvements

The existing CCT is not adequate for conveying or disinfecting flows beyond 7 mgd; therefore, additional disinfection capacity is required if Alternative 3 is implemented. Three options for increasing disinfection capacity were identified:

- *Disinfection Alternative A*: Construct a new wet weather CCT and use the existing tank in parallel with the new tank for chemical disinfection. This alternative would include hydraulic improvements to the existing CCT.
- *Disinfection Alternative B*: Construct a new disinfection facility using ultraviolet (UV) disinfection for the entire range of flows. This alternative would require a new tank and UV equipment with capacity for the full range of flows. Increasing the hydraulic capacity of the existing CCT to facilitate this alternative would not be practical given the extent of modifications that would be required.
- *Disinfection Alternative C*: Install UV disinfection equipment in the existing CCT for secondary effluent up to 7 mgd, and use chemical disinfection in a parallel tank for flows above 7 mgd. This alternative would reduce the equipment costs associated with providing UV equipment for the entire range of flows, but would add an additional disinfection system to operate and maintain.

UV disinfection has the advantages of eliminating potentially harmful disinfection by-products formed by combining chlorine with partially nitrified ammonia and eliminates the need to deliver, store, and manage large quantities of chemicals. UV disinfection systems will also reduce tank volume requirements when compared to chemical disinfection. UV disinfection is disadvantaged because it requires a large capital investment in equipment, ongoing maintenance and replacement of UV lamps, and a significant input of electric power for operation.

The estimated comparison costs for Disinfection Alternatives A, B, and C are $0.86M, $2.6M, and $1.66M, respectively. It is recommended that the City continue to implement chemical disinfection based on the additional capital and O&M costs required for UV disinfection.

Addressing the deficiencies at the CCT will allow for better disinfection of effluent and peak hydraulic conveyance.

6.2.5 Outfall and Effluent Mixing Structure

The existing 14-inch-diameter outfall is undersized for flows above 7 mgd and not optimized for mixing of effluent with receiving waters. If liquid stream Alternative C is implemented, a new parallel outfall sized to convey the full range of flows is required. The new outfall would be equipped with multiple discharge ports to enhance mixing and dilution of the effluent and receiving waters; a mixing zone study would be required to optimize the design parameters. A new parallel outfall could be used for flows up to 7 mgd, and the existing outfall could be used for flow exceeding 7 mgd. Another possibility would be to construct a larger-diameter outfall with sufficient capacity to convey the full range of flows. Predesign activities should include a condition assessment of the existing outfall, constructability assessment within the existing alignment, and mixing analysis to develop the best design from a regulatory and process perspective.

A mixing structure, constructed at the upstream end of the outfall pipe, would provide a convenient location for recombining parallel flows and for sampling final effluent. It could also serve as a hydraulic control point if dual outfalls are used. The structure should incorporate features necessary for convenient withdrawals of effluent for reuse purposes, should effluent reuse be implemented at a future date to mitigate thermal loads.

6.2.6 Electrical and SCADA

The existing SCADA system requires updating and integration with proposed improvements. SCADA and electrical system updates will be identified as part of predesign activities.

6.2.7 Standby Power

The City will need a new standby generator to provide power sufficient to provide full treatment during a power outage. The generator ideally would be sized and the electrical system would be designed such that the entire plant including the operations building is energized when outside electrical service is interrupted. At a minimum, the generator will be sized so that all processes required to meet effluent limitations are powered.

A preliminary capacity calculation recommends a 500 kW generator for the purposes of developing planning costs. The assumptions for cost estimating assume that the new generator is pad-mounted in a self-contained, sound-attenuated enclosure, and equipped with a hospital-grade muffler. The capacity of the standby generator is based on a preliminary estimate of pump and equipment horsepower requirements. A more detailed evaluation of the generator sizing should be conducted during predesign.

Including this equipment in the upgrade project would bring the WWTP into compliance with having reliable and selective redundant systems online during power outages.

6.2.8 Wet Weather Parallel Treatment

The proposed wet weather parallel treatment process is described below. Parallel wet weather treatment would be required for all variations of Alternative 3 to address flow and treatment requirements at the WWTP.

6.2.8.1 Wet Weather Peak Flow Conveyance and Treatment

The regulatory peak flow that must be conveyed and treated is approximately 13 mgd, while the capacity of the existing WWTP is approximately 7 mgd. Conveyance and treatment of flows exceeding

7 mgd would use a parallel treatment system incorporating chemically enhanced and ballasted HRC. HRC requires the use of a coagulant, polymer, and fine ballast material to facilitate flocculation and rapid settlement of suspended solids. pH adjustment may be required depending on the alkalinity of the treated water. Rapid settling of ballasted particles allows for use of a small tank footprint when compared to traditional primary clarification, which relies on long detention times (and thus much larger tanks).

6.2.8.2 Process Description

Two HRC processes were considered, Veolia's Actiflo process and Evoqua's CoMag process. Both processes use a chemical coagulant, polymer, and ballast material to promote flocculation and rapid settling of particles. Additionally, both processes return settled solids to the settling tank to further enhance settling. Actiflo introduces fine-grained sand to ballast the floc, whereas CoMag uses magnetite, an iron-based ballasting agent with high specific gravity and magnetic properties. Solids captured by the process would be returned to the influent stream and ultimately be removed in the secondary clarifiers.

Typically, the HRC unit consists of three process zones: one for chemical introduction, one for floc maturation, and one for settling. The main advantage of these systems is a small tank footprint and reduced cost when compared to conventional clarifiers treating the same flow rate. The process can also be brought online quickly in response to high flow conditions. Actiflo uses lamella tube settlers to enhance settling. CoMag uses a conventional clarifier design relying on flocculation and dense ballast material for rapid settling. Actiflo uses a cyclone for recovery of ballast sand while CoMag uses a magnetic drum to extract the magnetic ballast material. Both processes require periodic addition of ballast material to account for material that escapes the recovery process. Other manufacturers offer systems very similar to the Actiflo process, whereas the CoMag process is patent-protected and unique in some respects.

A conceptual layout of the Actiflo process is shown in Figure 6-7. The layout shows the chemical injection tank on the left side of the figure, maturation tank, and settling tank with lamellar settling plates. The underflow pumps removing the captured solids and ballast pump the slurry to the cyclones pictured above the maturation tank. The cyclones separate the ballast and return it to the maturation tank; the solids component of the slurry would be conveyed to the influent stream and ultimately removed in secondary treatment.

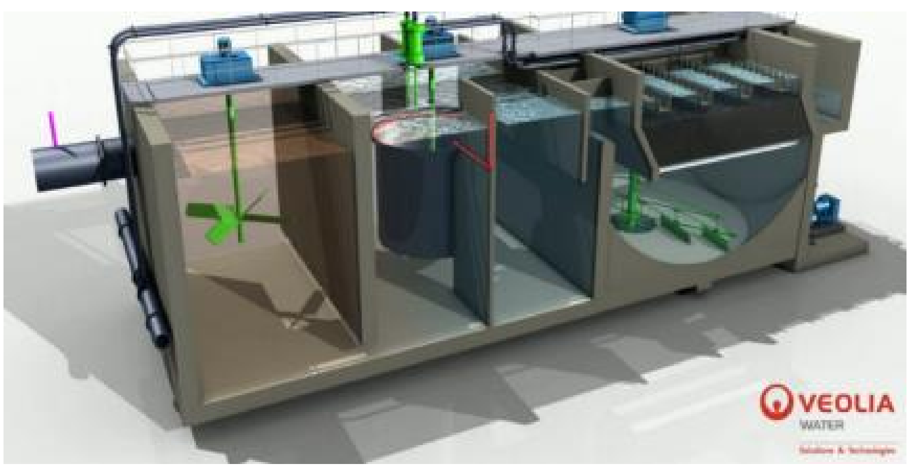


Figure 6-7. Actiflo HRC

Source: Veolia Water

The CoMag HRC is illustrated in Figure 6-8. The primary differentiators including use of magnetic drum for recovery of ballast materials and gravity clarifier design are illustrated in the figure. A support building would be required for ballast recovery, chemical feed equipment, and chemical storage.

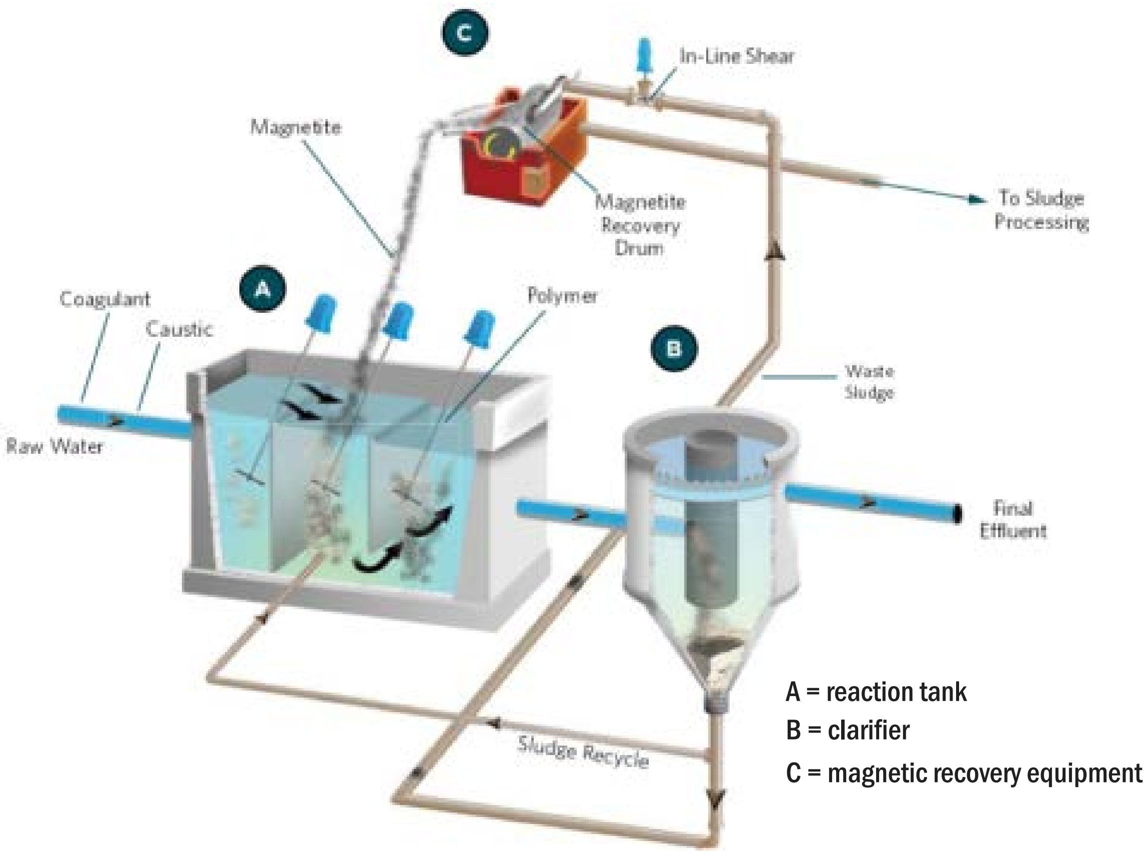


Figure 6-8. CoMag HRC

Source: Evoqua

Both HRCs have similar footprints and both have the potential to be used for tertiary polishing in lieu of filters as described in a subsequent section. Onsite pilot testing is required to confirm the performance for both wet weather treatment and tertiary polishing.

Both HRCs could use a small biological contact tank (CT) receiving raw wastewater and a RAS stream to provide a means to reduce soluble BOD in the influent. The soluble BOD would be incorporated into the waste solids that get returned to the WWTP for treatment. Provisions could be made for adding a CT later should biological treatment become a mandated condition.

6.3 Solids Handling Alternatives

Since 2009, the WWTP has consistently been hauling about 203 DT of solids per year or about 3.8 DT per week on average to the landfill according to operating records. Solids are dewatered to 13.5 percent concentration on average, meaning the City is producing approximately 28 tons of wet solids on a weekly basis. This section discusses possible options for future solids disposal.

6.3.1 Landfilling Solids at Wasco County Landfill

The City currently stores WAS in the solids holding tank until it can be dewatered and hauled to disposal at the Wasco County Landfill, located near the city of The Dalles, OR. The dewatering equipment is typically operated 5 days per week during hours that the plant is staffed. Dewatered solids are temporarily stored in a 20 yd^3 container located outside of the solids handling building. The City contracts with a local hauler to supply and transport the containers and pays a tipping fee at the landfill.

The City pays approximately $100k annually to haul and dispose of solids at the Wasco County Landfill. The economics of the existing program are listed in Table 6-7.

Table 6-7. Current Solids Disposal Program Costs	
Program element	**Annual cost ($)**
Hauling of solids from WWTP to Wasco County @ $203/trip	38,150
Tipping fee at Wasco County @ $40.50/wet ton	60,900
Total annual cost for hauling and tipping fee	**$99,050**

Table 6-7 is based on an annual disposal of 203 DT of solids at 13.5 percent concentration and trucking costs of $203 per trip and a landfill tipping fee of $40.50 per wet ton hauled.

If the City were to upgrade its existing dewatering equipment and increase solids concentration from 13.5 percent to 18 percent, the volume of wet solids hauled annually could be reduced by approximately 25 percent. The City recently completed pilot testing of a dewatering screw press and the results indicate that the City's WAS can be dewatered in the range of 18 to 23 percent depending on the degree of polymer addition. By upgrading the dewatering equipment and assuming a conservative thickened solids content of 18 percent on average, the City could reduce the annual cost of hauling and landfilling its solids to $75k. This estimate assumes other associated costs were held constant.

6.3.2 Solids Stabilization and Class B Land Application Program

Residual solids can be treated to EPA Class B quality standards through stabilization by the following methods:

- air drying for a prescribed time and temperature,
- composting to specific requirements,
- anaerobic digestion,
- aerobic digestion, or
- lime addition.

Air drying and composting are not viable alternatives given that the WWTP is located within the city limits. Anaerobic digestion is not possible given the lack of primary sedimentation facilities at the WWTP.

Lime stabilization and aerobic digestion are possible alternatives. Both were previously practiced by the City but lime stabilization is the lower capital and energy cost alternative of the two, thus it will be used as the basis for evaluating Class B land application of biosolids at the City.

The City has previously administered a Class B land application program using lime stabilization. The lime silo is still installed but the conveyance and mixing augers have been removed. Challenges associated with the land application program included the lack of preliminary treatment to remove plastics from the solids stream, increased labor requirements, health and safety concerns associated with handling lime, odor generation during stabilization, and the lack of a dewatered solids storage facility needed when wet weather conditions make land application impractical.

Considerable capital expenditure would be required to re-establish a Class B land application program. For estimating purposes, it was assumed that a modern lime feeding and mixing system would be used to effectively implement lime stabilization and that an enclosed building with odor control provisions would be required for seasonal storage of dewatered solids. A truck, specifically equipped for hauling and spreading dewatered solids, was also included in the cost estimate.

A modern lime stabilization system would consist of a new lime feeder, dewatered cake conveyor between the dewatering device and the lime/solids mixer, mixing tank to effectively mix lime with the dewatered solids, conveyor between the mixer and the transport trailer, and programmable controls for automation of the system. Environment controls would be incorporated with the lime stabilization system including provisions for dust control and an acid wash system for control of scale formed by the reaction between lime, water, and air. The planning-level cost associated with a new lime stabilization system is $0.38M.

A 5,300 ft^2 enclosed building on-slab with foundation and drainage provisions for managing dewatered solids was assumed for seasonal storage of solids. These costs would be in addition to the solids storage tank improvements and dewatering facility improvements required as essential upgrades to the solids handling program in general. These additional costs for re-establishment of a Class B land application program are summarized in Table 6-8.

Table 6-8. Solids Stabilization and Land Application Option

Item	Estimated cost ($)
Lime stabilization equipment	380,000
Land application truck with spreader	180,000
Wet weather solids storage building	530,000
Front end loader	100,000
Cost of additional dewatering equipment and building	1,200,000
Annuitized cost of equipment and storage building [a]	**74,000**
Annual operating cost assumption [b]	100,000
Total annual cost	**$174,000**

a. Calculation assumes an annual interest rate of 2% and a 20-year loan.

b. Annual operating cost assumption includes one full-time equivalent (FTE) and fuel for equipment.

Non-cost factors associated with this option would include increased potential for odor generation at the WWTP, health and safety concerns associated with lime stabilization, and additional recordkeeping requirements.

6.3.3 Haul Liquid Solids to the Willow Lake Facility

Another possible option for solids disposal would be liquid hauling of thickened solids to the City of Salem's Willow Lake Water Pollution Control Facility (WPCF). This option offers the advantages of eliminating the need to stabilize or dewater the solids but would require thickening of WAS to 5 percent concentration.

Willow Lake has excess digester capacity and could negotiate a long-term agreement for treating the City's solids, according to Salem WPCF staff. This option would require installation of a thickener in lieu of new dewatering equipment and a new load-out station to transfer thickened solids from the solids holding tank to a dedicated tank trailer.

A small capacity, platform-mounted thickener could be installed to discharge thickened WAS directly to the solids holding tank. The sludge would be thickened to the degree that it could be effectively mixed and aerated within the holding tank and eventually transferred to the truck. For estimating purposes, it was assumed that solids could be thickened to 5 percent concentration and still be pumped and transported in liquid form.

Hauling of liquid solids would remove the requirement to dewater solids, thus removing the need to upgrade dewatering equipment and eliminating O&M costs associated with dewatering operations. Odors associated with the dewatering operations could be avoided. Mass loads from the dewatering equipment centrate return could be avoided. Odors associated with thickening of solids could be managed by totally enclosing the thickener. Table 6-9 provides information related to this option.

Table 6-9. Willow Lake Facility Hauling Option

Item	Estimated annual cost ($)
Hauling of solids to Salem [a, c]	48,000
Tipping fee @ $0.05/gallon [b, c]	50,000
Total annual cost for hauling and tipping [c]	**$98,000**

a. *Cost to haul based on adjusted cost currently paid to haul dewatering solids to Wasco County Landfill.*

b. *Tipping fee from City of Salem (2015).*

c. *Calculations based on hauling thickened solids at 5% solids and an annual production of 1 MG of thickened solids, resulting in 200 trips per year.*

Section 7

Selection of Treatment Alternatives

This section evaluates the alternatives presented in the previous section, and provides the basis for the recommended alternative presented in Section 8. Selection of alternatives for liquid stream treatment is presented first. Solids handling alternatives are discussed later in the section.

7.1 Liquid Stream Alternatives

Three general alternatives were considered for addressing the deficiencies identified in Section 6:

- *Alternative 1: further reduce I/I in the collection system.* Establish additional I/I reduction program to reduce peak flows to a level that can be conveyed and treated by the existing WWTP.
- *Alternative 2: flow equalization storage.* A storage facility and appurtenances would be sized to equalize peak wet weather flows to a level that can be conveyed and treated by the existing WWTP.
- *Alternative 3: WWTP upgrade.* Upgrade the existing treatment facility to increase both hydraulic and treatment capacities.

The listed alternatives are not necessarily mutually exclusive. Adoption of Alternative 1 or 2 would also require improvements to the treatment facilities, as described in Section 6, to increase secondary treatment capacity and reliability of the existing facilities to confidently handle 7 mgd for the planning horizon. Conversely, Alternative 3 would incorporate an expansion of both hydraulic capacity and treatment capacity sufficient to treat the full range of existing and projected flows.

As in Section 6, planning-level costs presented in this section are estimates of the cost to construct or modify each of the affected processes and are only for comparison. These costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Total project costs for the recommended alternative are presented in Section 8.

7.1.1 Alternative 1: Further Reduce I/I in the Collection System

This alternative assumes additional removal of collection system I/I to achieve reductions of existing peak flows at the WWTP by approximately 6 mgd. This reduction has an immediate benefit to the WWTP and collection system by mitigating peak flows and the potential for sewer overflows. However, the option is costly and does not increase treatment reliability or capacity at the plant.

Between 2002 and 2012, the City invested more than $15M on planning, design, and construction of four collection system R&R projects within the service area. The construction costs for each of the already completed phases are listed in Table 7-1. The areas incorporated into the four projects were carefully chosen to maximize the cost-effectiveness of improvements (i.e., cost per gallon I/I removed).

Table 7-1. Summary of R&R Costs by Phase

Construction phase	Construction cost ($)
Phase 1	1.3M
Phase 2	1.7M
Phase 3	3.1M
Phase 4	6.0M

The projects listed above account for approximately 35 percent of the main line sewers and 30 percent of laterals with an estimated removal of 50 percent of the peak hour I/I in the service area. The reduction in I/I has reduced peak flow to the WWTP from approximately 22 mgd to 11.5 mgd. This equates to approximately $1.10 spent for every 1 gallon of I/I removed.

The analysis described in Section 6 assessed the continuation of the reduction projects and if they would be a cost-worthy investment. The remaining basins were ranked and the highest-priority basins were identified. The results show that the most cost-effective basins have been largely addressed and there is a diminishing rate of return on further investment in collection system rehabilitation.

Rehabilitation of the remaining basins would be required at an estimated cost of $28M, or $5.30 per gallon of I/I removed, and would still fall 1 mgd short of reaching the goal of limiting peak hour flows to the WWTP to 7 mgd. In addition $10M would be required to make improvements to the WWTP to more reliably treat 7 mgd.

7.1.2 Alternative 2: Flow Equalization Storage

Analysis completed in Section 6, based on results from the collection system model described in Section 3, indicate that a treatment volume of approximately 9.4 MG is required to mitigate flows above 7 mgd. The storage of peak flows requires additional tankage, land acquisition, and pumping equipment and is expected to cost at least $28M. Implementation of this alternative also requires an additional $10M to make WWTP improvements required to more reliably treat 7 mgd.

7.1.3 Alternative 3: Secondary Treatment Improvements

Alternative 3 includes three possible sub-alternatives for upgrading secondary treatment in addition to expansion or construction of new facilities for influent pumping, conveyance, disinfection, and solids treatment. The three sub-alternatives for providing secondary treatment and the associated costs are discussed below.

7.1.3.1 Secondary Treatment Alternative 3A

Expand the existing aeration, sedimentation, and conveyance elements of the secondary treatment system to convey and treat the full range of flows.

Flows at the WWTP greater than 5 mgd are infrequent and dilute. The vast majority of flows are less than 3 mgd, while the projected peaks range upward of 13 mgd. Conveyance and treatment of flows between 3 and 13 mgd would require a parallel secondary treatment system that would be used infrequently. Influent loadings would be insufficient to sustain the biological process within the expanded volume year-round, and bringing the standby capacity online for peak flow events would take several days to weeks to establish sufficient MLSS to enable effective treatment. For these reasons, providing traditional secondary treatment for the full range of flows is impractical and not considered a viable alternative.

7.1.3.2 Secondary Treatment Alternative 3B

Construct a new secondary treatment system using MBR and a parallel wet weather treatment system.

This alternative involves abandoning the existing aeration basins and secondary clarifiers in favor of a new MBR process for flows up to 7 mgd and a parallel wet weather treatment system for conveyance and treatment of flows exceeding 7 mgd. For this alternative, a parallel wet weather treatment system using HRC would be required to convey and treat flows greater than 7 mgd. Dilute influent conveyed through the HRC would be disinfected prior to discharge.

7.1.3.3 Secondary Treatment Alternative 3C

Upgrade the existing secondary treatment system and construct a parallel wet weather treatment system.

This alternative involves the following:

- **Aeration basin improvements:** Improve oxygen transfer at the existing aeration basins with the installation of fine-bubble diffusion. Add a third, deeper aeration basin to increase treatment capacity. Improve flow splitting to the clarifiers.
- **HRC:** Addition of a parallel wet weather treatment system using HRC would be required to convey and treat flows greater than 7 mgd. Dilute influent conveyed through the HRC would be disinfected prior to discharge.

  An added benefit to the installation of HRC at the WWTP is that it may be used as a potential tertiary treatment system as a polishing filter during dry weather. This concept, which has been implemented at other facilities, has the benefit of avoiding the capital costs associated with new filters. The existing filter pump station could be upgraded and used for conveyance of secondary effluent to the HRC for chemically enhanced sedimentation prior to disinfection. Pilot testing of the concept is recommended to determine the effluent quality possible with HRC and the level of chemical addition required. This option also offers the potential benefit of providing for chemical removal of phosphorus should nutrient reduction become a regulatory requirement.
- **Secondary clarifier mechanism replacement:** Install new clarifier mechanisms in the two original secondary clarifiers with energy-dissipating baffles, deeper flocculating skirts, and improved effluent baffles.
- **RAS and WAS pumping improvements:** Increase the RAS recycle capacity by approximately 100 percent to enhance treatment reliability during wet weather events. Appropriately size the WAS pumps and enhance their controls to optimize solids management.
- **Filtration:** The capacity range of the existing traveling-bridge sand filters is limited to approximately 2 to 4 mgd. The filter tankage and equipment are generally in good condition. Treatment beyond 4 mgd requires the installation of new filters. Options exist for new media-type filtration or use of the HRC, as described above.

7.2 Discussion of Liquid Stream Alternatives

By eliminating the expansion of the existing treatment facility, Alternative 3A, four alternatives are presented for further evaluation. This section includes a discussion of general issues as well as costs, phasing, O&M, and seismic considerations for each alternative. The four alternatives are:

- Additional reduction of I/I in the collection system to limit wet weather flows
- Temporary storage of flows exceeding existing treatment capacity
- New MBR-based secondary treatment processes and new wet weather treatment process
- Improvements to existing secondary treatment facilities and a new wet weather treatment process

7.2.1 General

Based on the summary presented above, the first two alternatives mitigate peak flows being conveyed to the WWTP during high flow events, but neither alternative on its own improves existing treatment infrastructure or provides additional treatment capacity required for existing and future conditions. Additionally, because these two options are not mutually exclusive, some level of repair and/or upgrades at the WWTP is required.

A new MBR-based secondary treatment process for flows up to 7 mgd in conjunction with a wet weather treatment process does offer advantages in terms of reducing the need for downstream filtration and achieves comparable effluent quality. However, the MBR-based alternative does not make the best use of the existing infrastructure, thus significantly increasing the overall cost of the project.

Improvements to the existing secondary treatment process coupled with a wet weather treatment installation does not provide as much new infrastructure, However, it makes the best use of the existing infrastructure. The use of existing infrastructure has the significant added benefit of providing phasing opportunities to increase affordability.

7.2.2 Costs

Planning-level costs presented in Table 7-2 are an estimate of the cost to construct or modify each of the affected processes. The costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Each construction cost estimate was developed using standard cost estimating procedures including layouts, equipment quotations, and unit costs based on a November 2015 ENR index. These construction costs are intended to provide a reference point for comparison for the possible alternatives. Total capital costs are presented in Section 8 for the recommended alternative. The basis of the cost estimates and further cost information are presented in Appendix F.

The lowest-cost alternative is 3C, which upgrades the existing facility with the addition of HRC.

Table 7-2. Comparative Construction Cost Summary for Liquid Stream Alternatives[a]

Alternative	Description	Collection system costs, ($)	WWTP costs to treat 7 mgd, ($)	WWTP added costs to treat peak flows up to 13 mgd, ($)	Total
1	Further reduce I/I	28M	10M	0	38M
2	Flow equalization/storage	28M	10M	0	38M
3a	Parallel secondary process	0	N/A	N/A	N/A
3b	MBR with HRC	0	20.1M	6.7M	27M
3c	Upgrade existing and add HRC	0	12.2M	6.7M	19M

a. *Costs do not include engineering, construction management, administration, or escalation to the midpoint of construction. Biosolids improvements are also not included here. See Section 8 for total project costs.*

7.2.3 Phasing

The presented alternatives were evaluated on their ability to incorporate phasing into their construction sequencing. The costs presented above would have significant impacts on the City's revenue needs if they were incorporated as one project in the near term. Shown in bullets below is a phasing discussion for each alternative:

- I/I reduction can be phased based on sewer sheds, as identified in the analysis presented in Section 6. The phasing plan would likely take many years to complete
- Temporary storage cannot be phased.
- The MBR-based alternative has limited phasing opportunity; the first phase would be a comprehensive WWTP upgrade with large capital costs.
- Upgrading the existing system allows for multiple phasing options.

7.2.4 Operations and Maintenance

The O&M costs for the proposed alternatives are generally based on the existing O&M costs (baseline), plus or minus for the new or excluded components:

- I/I reduction and storage, Alternatives 1 and 2, generally do not appreciably increase the baseline O&M costs. Additional oversight and pumping energy costs are assumed for the infrequent storage of peak flows.
- O&M costs for the MBR-based Alternative 3B were not developed in detail, as there would need to be a large reduction in O&M costs to demonstrate long-term financial benefits of this option. On the contrary, MBR systems are energy-intensive and would certainly add costs to the baseline.
- The upgrade-based Alternative 3C would be very similar to baseline O&M costs with the addition of infrequently used energy and chemical costs for the new HRC. It is not expected that the addition of HRC will require additional plant staff certifications – as such, existing staff will be capable of operating the new equipment. Usage of the HRC system is assumed to be 5 days at 4 mgd, which far exceeds Sweet Home's actual experience with frequency of flows beyond the WWTP's 7 mgd capacity. See Table 7-3 for an estimate of O&M costs for an HRC system.

Table 7-3. Summary of Annual HRC O&M Costs [a, b]

Item	Costs ($)
Additional Labor	0
Coagulant	3,800
Polymer	400
Sand	100
Maintenance	1,200
Electrical	500
Total	**6,000**

a. *Unit prices for chemicals taken from Lake Oswego Tigard Water Treatment Plant 2016. Chemical dosages and sand consumption taken from King County Water Reuse Technology Demonstration Project Pilot Study Ballasted Flocculation 2002.*

b. *Assumed annual flow treated is 5 days at 4 mgd or 20 MG total.*

7.2.5 Seismic Considerations

Based on the age of the existing WWTP structures, and past history of similar structures of this type, the WWTP may suffer varying levels of failure during a future significant seismic event. Based on recent, much-publicized research, the risk of a major earthquake triggered by the Cascadia Subduction Zone is higher than previously understood. Seismic retrofit of existing facilities is not likely feasible and no costs are included for any such efforts. However, new facilities would be designed to the latest seismic standards.

Seismic resilience of each alternative is briefly assessed below:

- I/I reduction and storage alternatives do not improve the seismic reliability of the WWTP at any phase of their implementation.
- The MBR-based alternative allows for a comprehensive upgrade of the plant, thus allowing the new elements to be designed to the new seismic reliability standards.

- The upgrade-based alternative provides new seismic rated IPS, HRC, and disinfection upgrades which will provide significant treatment while repairs/replacement of existing damaged structures are completed.

During predesign, a seismic assessment of the existing WWTP structures should be conducted to identify weaknesses and how they can be addressed and to plan for future, incremental upgrades to provide enhanced resilience.

7.3 Selected Liquid Stream Alternative

The recommended alternative is 3C, upgrade the existing secondary treatment system and construct a parallel wet weather treatment system. It is the lowest-cost, most affordable option for the City because of its use of existing WWTP facilities and opportunities for phasing. O&M cost increases beyond present costs are minimal, and the HRC system will provide the most effective treatment for flows beyond 7 mgd. This alternative also allows phased-in seismic enhancements as new facilities can be designed to current standards.

7.4 Selected Solids Handling Alternative

It is recommended that thickening of solids and liquid hauling to Willow Lake be further considered in predesign, as it provides benefit to operations at the plant and was identified as the most cost-effective solids handling solution. Because a hauling agreement with Willow Lake was not in effect at the time of this report, a newly designed on-site dewatering biosolids system has been included in the cost estimate presented in Section 8. The on-site dewatering of solids should also be further evaluated in predesign.

Re-establishment of the City's biosolids land application program is not recommended based on the high capital costs, ongoing O&M costs, and potential for odors.

Section 8

Recommended Alternative

This section presents a comprehensive plan for improvements drawing from previous sections and provides design data and a description of the recommended alternative. A schematic layout and hydraulic profile are provided. Detailed capital costs associated with the recommended improvements are presented along with a schedule for phasing.

8.1 Description of Recommended Alternative

The recommended alternative is upgrade of the existing secondary treatment system and construction of a parallel wet weather treatment system. The associated improvements make the most cost-effective use of the existing infrastructure, control O&M costs, meet existing and anticipated regulatory requirements, and provide seismic resilience for new facilities.

Fundamental improvements associated with the recommended alternative are a new IPS and preliminary treatment improvements, secondary treatment improvements, a parallel system for conveying and treating flows greater than 7 mgd, disinfection improvements, hydraulic improvements including a new outfall pipe and diffuser, and improvements to solids handling facilities. The improvements composing the recommended alternative and their functions are listed in Table 8-1 and described in greater detail below.

Table 8-1. Recommended Alternative Elements

Improvement	Function
New IPS	Increase pumping capacity to convey peak flows
Mechanical bar screening facilities	Remove rags and debris and prolong equipment life
Grit removal facilities	Reduce maintenance and prolong equipment life
Wet weather treatment (HRC)	Provide effective treatment for peak flows
Third aeration basin and improvements for existing basins	Increase secondary treatment capacity
Clarifier upgrades	Improve secondary effluent quality
New tertiary filters	Improve final effluent quality
Disinfection improvements	Treat higher flows
Outfall upgrades	Increase peak conveyance capacity
Biosolids storage improvements	Reduce odors, improve performance, and reduce maintenance
Biosolids dewatering improvements	Reduce hauling and solids disposal costs
Miscellaneous improvements	Increase reliability of existing systems by modifying or improving existing equipment

8.1.1 New Influent Pump Station

A new IPS, sized to provide a firm capacity of at least 13 mgd, is required to convey the 5-year, peak-hour flow. For cost estimating and layout purposes, a self-cleaning, trench-style pump station design is assumed using multiple submersible pumps. The most effective approach would incorporate one or two smaller pumps providing service for average flow conditions and two or three larger pumps for providing the firm peak design capacity plus redundancy. For cost estimating purposes, the firm capacity of the IPS would be 13 mgd and all pumps would be equipped with VFDs. The IPS would be constructed below grade with the associated electrical gear located in either a new or existing building.

8.1.2 Plant Pump Station

A small, submersible pump station with capacity sufficient to pump the process return streams to a location downstream of the IPS should be constructed when the new IPS is constructed. A plant pump station would recycle the process streams to a location downstream of the influent sampling location and eliminate double counting of BOD and TSS loads. The size and arrangement of the plant pump station will be evaluated during predesign. It is expected that a duplex system will be provided with redundant pumps.

8.1.3 Mechanical Bar Screening Facilities (Headworks)

New headworks, consisting of screening and grit removal facilities, would be ideal from an O&M perspective but the grit removal components can be postponed to defer considerable capital expenditures. The initial phase of the headworks facility construction would consist of a dual-channel headworks incorporating a mechanical bar screen, screenings washer/compactor, and flow diversion provisions. Removal of screenings would improve secondary clarifier performance, protect and extend the life of downstream equipment, and reduce plastics in the solids stream, which increases options for solids disposal. Screening is also required for the HRC process.

8.1.4 Grit Removal (Headworks)

Grit removal, when implemented, would reduce maintenance requirements associated with periodic cleaning of the aeration basins and solids holding tank, and prevent grit-related damage to equipment. The City and their contract operator will determine when to construct this facility depending on when funds are available and whether applicable life-cycle costs show it to be a sound investment. For the purpose of sizing the grit removal facility and developing the cost estimate, a stacked-tray grit removal system was assumed. A stacked-tray grit removal system minimizes the facility footprint, optimizes grit removal performance, and reduces energy requirements when compared to other grit removal technologies. To defer construction of the grit removal element, the screening and grit removal structures would need to be designed and constructed as separate but adjacent structures. Specific details for the new headworks would be developed during predesign.

8.1.5 Third Aeration Basin and Improvements for Existing Basins

A third aeration basin is ultimately recommended for treatment of future loads. Initially, this element of the recommended alternative would incorporate improvements to the existing aeration basins. When required, a third aeration basin would enable treatment of additional BOD and ammonia loads, enhance settleability of MLSS, and provide for more effective management of biological solids during peak flow events.

Interim improvements to the existing aeration basins would incorporate conversion from the existing surface aerators to a fine-bubble aeration system and modification of the effluent weirs controlling the flow split between the clarifiers. Peak hydraulic capacity of the secondary treatment system would still be limited to approximately 7 mgd, but plant performance would be extended by providing additional process air and by balancing the flow split to each clarifier. Replacement of the existing surface aerators with a fine-bubble aeration system would also reduce energy requirements for aeration.

When required, a third aeration basin would be constructed and during normal conditions would be operated in series with the two existing aeration basins to avoid potential issues with splitting flow and uneven treatment between two different sized/configured tanks. Provisions for parallel operation would be considered in predesign for low flow and/or maintenance activities and to provide operational flexibility for the future.

The third basin would be deeper than the existing basins to increase oxygen transfer efficiency, but would have similar footprint dimensions. The new basin would incorporate baffle walls to provide an anoxic selector for enhanced sludge settleability, reduce aeration requirements, and improve recovery of alkalinity. Multiple feed points for influent and RAS would be incorporated to enable flow configurations capable of concentrating and conserving MLSS during peak flow events. The additional volume provided by the third basin would also enable extending the sludge age as required for effective treatment of ammonia. Timing for the third basin would be determined by the performance of the two existing basins with aeration and hydraulic improvements and future load increases.

8.1.6 Secondary Clarifier Improvements

This element would replace the clarifier mechanisms in the two existing clarifiers with new, modern mechanisms to improve performance and reduce maintenance. The existing mechanisms were installed in 1974. New mechanisms would incorporate energy-dissipating inlets and rapid-rate sludge removal arms. Repair of eroded concrete surfaces would be incorporated into the improvements.

8.1.7 High-Rate Clarification

The regulatory peak flow that must be conveyed and treated is approximately 13 mgd, while the capacity of the existing WWTP is approximately 7 mgd. Conveyance and treatment of flows exceeding 7 mgd would use a parallel treatment system incorporating chemically enhanced and ballasted HRC. HRC requires the use of a coagulant, polymer, and fine ballast material to facilitate flocculation and rapid settlement of suspended solids. Rapid settling of ballasted particles allows for use of a small tank footprint when compared to traditional primary clarification, which relies on long detention times and thus much larger tanks.

8.1.8 Filter Upgrades

If wet weather permit limits were tightened for TSS, additional filtration would be required in the absence of other significant improvements to secondary treatment. Because of the low surface loading rate inherent to gravity sand filters, it would not be practical to expand the existing filters. Timing of new filters would depend on performance of the WWTP with interim improvements, timing of load increases, and future permit limits associated with wet weather flows.

8.1.9 Disinfection Improvements

The following section describes the plant’s existing disinfection system along with alternatives for future improvements.

Existing Disinfection. The cost estimates developed for all alternatives incorporating UV disinfection show that this technology is not competitive from either a capital cost or O&M perspective when compared to continued use of chemical disinfection. No regulatory drivers would currently favor discontinuation of chemical disinfection. For these reasons, it is assumed that Disinfection Alternative A will be implemented, and chemical disinfection will be continued at the WWTP.

The existing disinfection storage, feed, and instrumentation should be evaluated during predesign and upgraded as required. The chemical mixing equipment and instrumentation associated with disinfection, and dechlorination should be evaluated and updated as required.

PAA for Disinfection. Conversion to PAA for disinfection of flows below 7 mgd should be pilot tested and considered during predesign activities. Price reductions for PAA make this oxidant a viable alternative to chlorine disinfection that is gaining acceptance in the United States. The use of PAA allows for reduced retention time and contact volume requirement following chemical addition, and eliminates the need for a quenching agent for residual removal. PAA is effective at a lower dosage, which can reduce storage capacity requirements and is more stable than sodium hypochlorite over time, eliminating issues associated with off-gassing that can block pipes, cause leaks, and airlock feed pumps. PAA will not form undesirable by-products possible with chlorine disinfection, and is generally thought to have low potential for environmental harm when dosed at appropriate levels.

PAA will still require operator attention during handling/processing. The chemical is a mild to strong oxidant that may cause skin irritation or spontaneous combustion if mixed with non-inert materials such as wood products. Additionally, bulk quantity storage is unlikely due to long term stability concerns which could affect disinfection efficiency. Testing prior to use would be needed if PAA is stored for long periods.

Peak Flow Disinfection. A new CCT will be required for treatment of flows above 7 mgd. The new CCT should be equipped with flash mixing provisions and reliable instrumentation to optimize chemical dosing. The CCT will be sized to meet regulatory requirements.

Predesign activities should also include an evaluation of PAA for peak flow disinfection for the same reasons described above for disinfection of flows below 7 mgd.

Outfall Upgrades and Effluent Mixing Structure. A new, parallel outfall pipe and diffuser sized to convey either the full range of flows or some portion of it is required. The new outfall should be equipped with multiple discharge ports designed to enhance mixing of the effluent and receiving waters. A new parallel outfall could be used for flows up to 7 mgd, and the existing outfall could be used for flows exceeding 7 mgd. Predesign activities should include a condition assessment of the existing outfall, a constructability assessment within the existing alignment, and a mixing analysis to develop the best alternative from a regulatory and process perspective. A mixing structure, constructed at the upstream end of the outfall pipe, would provide a convenient location for recombining parallel flows and for sampling final effluent. It would also serve as a hydraulic control point if dual outfalls are used. The structure should incorporate features necessary for convenient withdrawals of effluent for reuse purposes should effluent reuse be implemented at a future date to mitigate thermal loads.

8.1.10 Solids Storage Tank Improvements

Recommended improvements for the existing storage tank include additional blower capacity and a robust diffuser system adequate to thoroughly mix the contents of the tank and keep the stored solids in an aerobic state. Maintenance of aerobic conditions would greatly reduce generation of foul odors, improve dewatering performance, and reduce tank maintenance requirements.

8.1.11 Solids Dewatering

A new dewatering device with additional hydraulic capacity and better dewatering performance would reduce solids storage requirements by increasing the volume of material processed per day, and reducing the volume of material hauled, thus reducing hauling and disposal costs.

The existing solids feed pump, in-line grinder, and polymer feed equipment should be evaluated and upgraded based on their condition and ability to meet future process requirements when the dewatering equipment is upgraded.

8.1.12 Miscellaneous Improvements

Peak RAS capacity should be increased by adding more RAS pumps or replacing existing with larger variable speed units. This will allow more effective management of the sludge blanket depth in the secondary clarifiers and avoid washout of solids under high flow and load conditions.

The WAS pumps should be replaced with smaller, variable speed units to better maintain a consistent MLSS concentration. There are no provisions for WAS metering or automatic control of wasting through SCADA. A check valve associated with the piston-type WAS pump tends to stick open, which allows mixed liquor to siphon from the clarifiers to the solids holding tank if the line is inadvertently not isolated between pumping cycles.

8.1.13 Design Data Summary

Design data for the proposed plant upgrade and for future plant upgrades are shown in Table 8-2.

Table 8-2. Existing and Proposed Design Data Summary

System	Existing	Proposed (by 2025)	Future additions (beyond 2025)
	Data/type	Data/type	Data/type
IPS			
Pump 1 (gpm/TDH/hp)	3,500 @ 50 ft TDH	**4,861/100 ft/200**	
Pump 2 (gpm/TDH/hp)	700 @ 40 ft TDH	**2,569/60 ft/55**	
Pump 3 (gpm/TDH/hp)	3,500 @ 50 ft TDH	**4,861/100 ft/200**	
Pump 4 (gpm/TDH/hp)	-	**-**	**4,861/100 ft/215**
Peak pump capacity (mgd)	6	**7**	**13**
Preliminary			
Sewage grinders (decommissioned)			
Type	In-line	-	-
Number	2	-	-
Width (in.)	12/18	-	-
Capacity/unit (mgd)	1.70/5.76	-	-
Total capacity (mgd)	7.4	-	-
Bypass channel (mgd)	8.5	-	-
Horsepower	5	-	-
Bar screen			
Type	Manual	**Mechanical**	**Mechanical**
Number	2	**1**	**add 1**
Opening size (in.)	2	**1/4**	**1/4**
Horsepower	N/A	**2**	**2**
Grit removal			
Type	-	-	**Stacked-tray**
Pump type	-	-	**Recessed-impeller**
Capacity (gpm)	-	-	**450**
Horsepower	-	-	**7.5**

Table 8-2. Existing and Proposed Design Data Summary			
System	Existing	Proposed (by 2025)	Future additions (beyond 2025)
	Data/type	Data/type	Data/type
Secondary treatment			
Aeration basin			
Number	2	2	**3**
Length, width, depth (ft)	64/30/12	64/30/12	**Add new 64/30/19**
Volume/basin (gal)	172,000	172,000	**273,000**
Aeration			
Type	Surface	**Fine bubble**	
Multi-speed	Yes, 2-speed	**N/A**	
Number (total)	2	**720 diffusers**	
Horsepower/mixer	8.6/15	N/A	
Aeration blowers			
Type	-	**PD rotary lobe**	
Number	-	**2 duty and 1 standby**	**Add 1 duty**
Capacity (scfm)	-	**1,400**	
Pressure (psi)	-	**10.1**	
Horsepower (each)	-	**50**	**50/60**
Wet weather treatment			
Type	-	**HRC**	
Capacity (mgd)	-	**8**	
Horsepower (total)	-	**35**	
Secondary clarifiers			
Type	Circular	Circular	
Number	3	3	
Diameter (ft)	45/45/60	45/45/60	
Avg. depth (ft)	12/12/15	12/12/15	
Collection mechanism type	Scraper	**Rapid sludge removal**	
Misc.	None	**Energy dissipating inlet**	
Overflow rate			
ADWF (gal/ft^2/day)	250	250	
PWWF (gal/ft^2/day)	4,160	4,160	
AWWF (gal/ft^2/day)	732	732	
Sludge pumps			
WAS pumps			
Type	Self-priming and piston	**TBD**	
Capacity (gpm)	3 @ 200	**3 @ 200**	
RAS pumps			
Type	Vertical centrifugal	**Vertical centrifugal**	
Capacity (gpm)	3 @ 685 2 @ 650	**3 @ 1,100** **2 @800**	

Table 8-2. Existing and Proposed Design Data Summary			
System	Existing	Proposed (by 2025)	Future additions (beyond 2025)
	Data/type	Data/type	Data/type
Biosolids holding tank			
Capacity (gal)	231,000	231,000	
Aeration type	-	**PD rotary lobe**	
Number	-	**2 duty and 1 standby**	
Capacity (scfm)	-	**1,400**	
Pressure (psi)	-	**7.2**	
Horsepower (each)	-	**50**	
Solids processing			
Type	BFP	**TBD**	
Number	1	**1**	
Size, meter	0.7	**TBD**	
Capacity (gpm)	35	**35**	
Anticipated cake (%)	18	**18–22**	
Lime auger	1		
Lime storage silo (tons)	20		
Polymer type		**Liquid emulsion**	
Tertiary treatment			
Type	Traveling-bridge sand filter		-
Number	2		-
Bed depth (in.)	11		-
MWWM flow (mgd)	4.0 (total)		-
MDWM flow (mgd)	2.2		-
Peak loading rate (gpm/ft^2)	4		-
Average loading rate (gpm/ ft^2)	2		-
Type	-	-	**Package filter plant**
Number of units	-	-	**6**
Capacity (mgd)	-	-	**15**
Blowers	-	-	**1 duty and 1 standby**
Horsepower (each)	-	-	**100**
Disinfection			
Chlorine gas, online (lb)	1 @ 2,000	**TBD**	
Chlorine gas, storage (lb)	2 @ 2,000	-	
Contact tank (gal)	50,000	**TBD**	
L:W ratio	17.3:1	-	
Detention		-	
ADWF (minutes)	38	-	
PWWF (minutes)	2.9	-	

Table 8-2. Existing and Proposed Design Data Summary			
System	Existing	Proposed (by 2025)	Future additions (beyond 2025)
	Data/type	Data/type	Data/type
Chlorination capacity (ppd)	200	-	
Mixer		-	
Horsepower	2	-	
Velocity gradient, s^{-1}	500	-	
Dechlorination: sulfonator		**TBD**	
Tank volume (gal)	1,184	-	
Sulfur dioxide cylinders		-	
Online (lb)	2 @ 150	-	
Storage (lb)	2 @ 150	-	
Sulfonator capacity (ppd)	100	-	
Mixer		-	
Horsepower	2	-	
Velocity gradient, s^{-1}	573	-	
Standby generator			
Rated capacity (kW)	150	**500**	

Notes: *Data adapted from WWTP Expansion drawings by KCM, Inc., 1992, and from information from plant operations staff.*
A " - " indicates equipment/component not provided for time period identified.
Blank spaces/cells indicate equipment/component continues its duty into the next planning period.

8.2 Layout of Recommended Improvements

A process schematic showing the existing plant facilities with the recommended improvements is shown on Figure 8-1. An aerial view showing the preliminary layout of the recommended improvements integrated with existing facilities is shown on Figure 8-3. The hydraulic profile associated with the recommended improvements is shown on Figure 8-4.

Alternative 3C

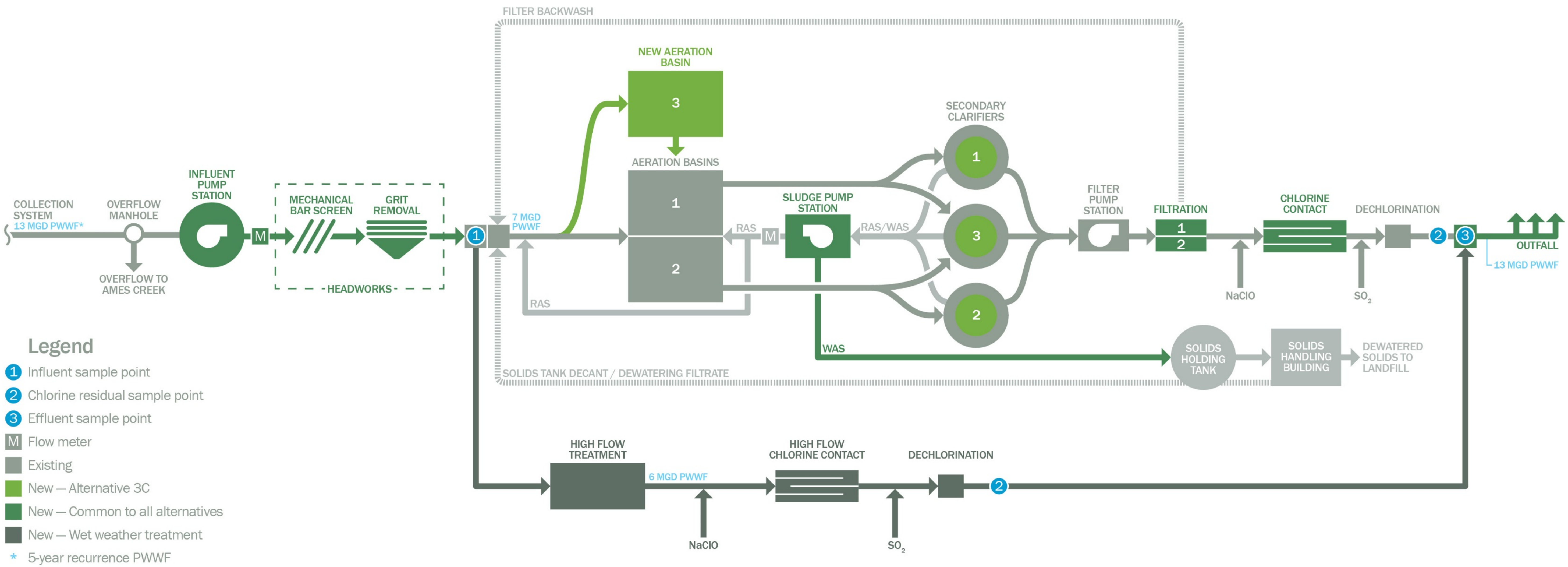


Figure 8-1. Alternative 3C Upgrade Existing and Wet Weather Treatment with HRC process schematic

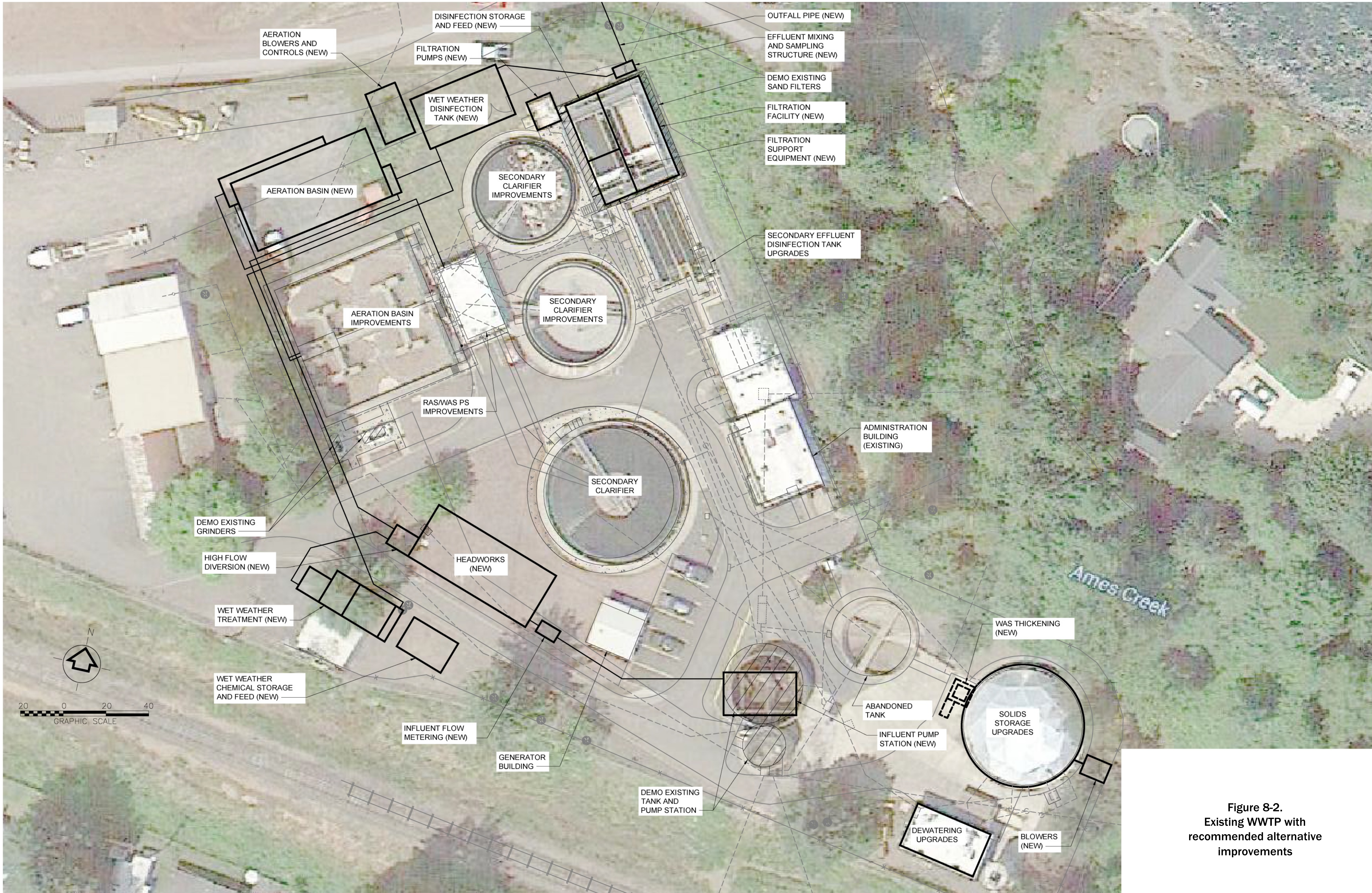
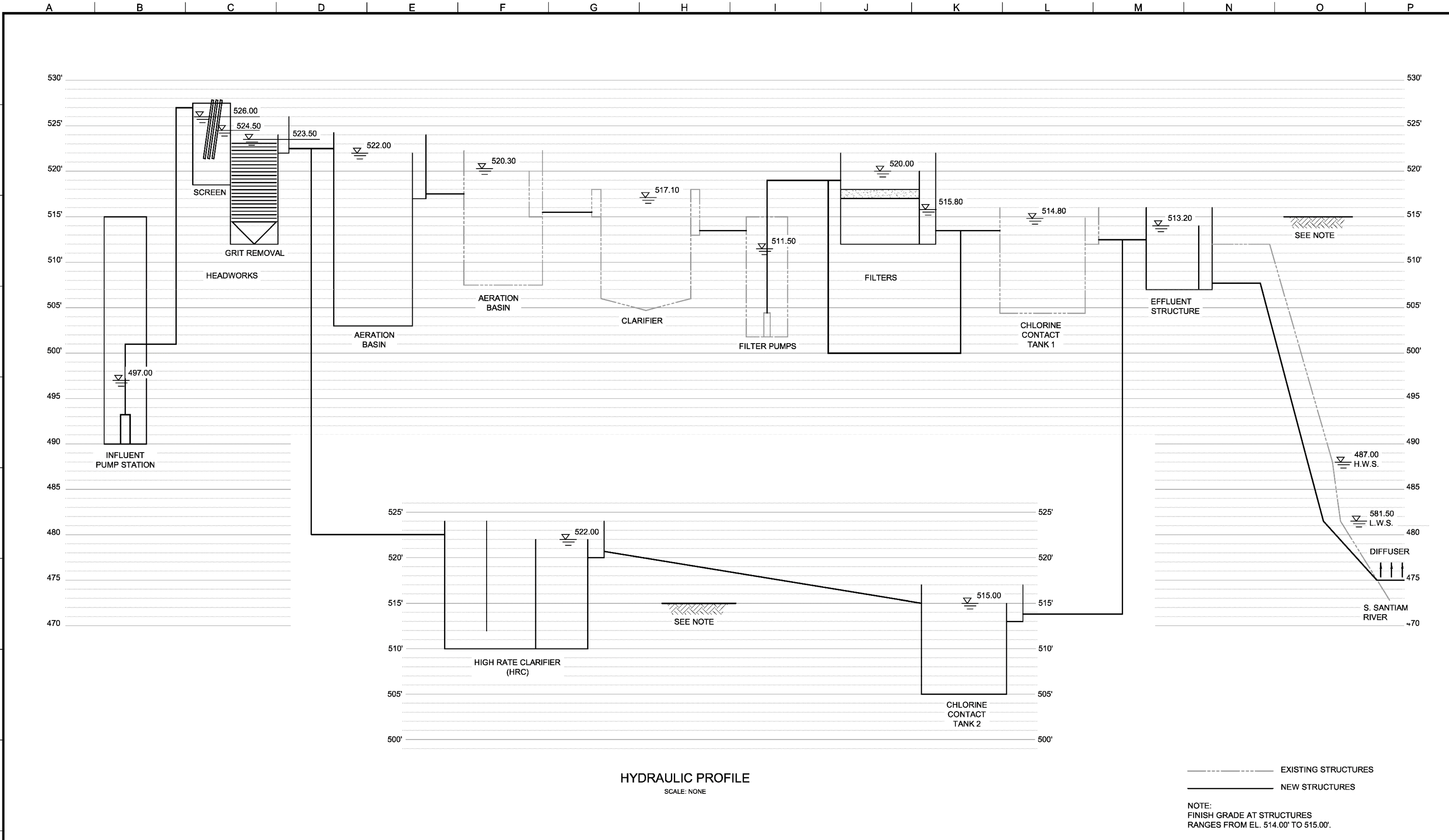


Figure 8-2. Existing WWTP with recommended alternative improvements



Brown AND Caldwell

PORTLAND, OREGON

SUBMITTED: ______ PROJECT MANAGER DATE: ______

APPROVED: ______ BROWN AND CALDWELL DATE: ______

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2" - SCALE ACCORDINGLY)

DESIGNED: DSM
DRAWN: NSW
CHECKED: DWL
CHECKED: RMG
APPROVED: DWL

EXTERNAL REFERENCE FILES

REVISIONS			
ZONE	REV.	DESCRIPTION	DATE

Figure 8-3. Recommended alternative hydraulic profile

FILENAME 144602-G-001.DWG
BC PROJECT NUMBER 144581
SCALE NONE
DRAWING NUMBER G-001
SHEET NUMBER OF

Path: P:\144602 Sweet Home WWFP\CAD_CAD\2-Sheets\G-General
Plot Date/Time: 9/15/2015 1:40 PM CAD: persson, doug

Table 8-3 lists recommended improvements by WWTP process area and construction costs.

Table 8-3. Recommended WWTP Alternative Elements

Item	Process area	Estimated construction cost ($) [a, b]	Notes
Influent pumping			
IP-1	New submersible pump station (initially with firm capacity = 7 mgd, upgraded by IP-2 to a firm capacity of 13.5 mgd)	3,294,000	Conceptual layout based on 4-pump submersible pump station with 3 pumps installed initially. One small pump provided for average day flows and two large pumps for peak flow conveyance; firm capacity = 7 mgd initially. Future pump installed as IP-2 project would be a third large pump to meet peak flow requirements with one pump in reserve (i.e., firm capacity = 13.5 mgd).
Influent pumping capacity expansion			
IP-2	Expand new pump station capacity to 14 mgd	430,000	Adds third large pump, controls, and electrical improvements to increase firm pump station capacity to 13.5 mgd.
Mechanical bar screen facility (one screen)			
PT-1	New mechanical bar screen in dual channel structure	492,000	New dual-channel 15 mgd mechanical bar screen facility with one mechanical screen; no grit removal.
Additional mechanical bar screen			
PT-2	Second mechanical bar screen in existing structure	285,000	Second mechanical bar screen in structure described under PT-2.
Flow diversion pipe and structure			
PT-3	Pipes plus diversion structures	113,000	Diversion structure allows the new IPS and mechanical bar screen to be used primarily for secondary treatment and for HRC when treating high flows.
Grit removal			
PT-4	15 mgd grit removal facility	1,994,000	Grit removal facility consisting of single stacked-tray grit removal unit, tank, and appurtenances.
Aeration improvements for existing basins			
ST-1	New fine-bubble diffusers, blowers, and controls for existing aeration basins	626,000	Replacement of the existing surface aerators in the aeration basins with fine-bubble diffusers, new blowers, and controls.
Secondary clarifier improvements			
ST-2	New mechanisms for existing secondary clarifiers	447,000	Includes replacement of original secondary clarifier mechanisms in two existing clarifiers.
New aeration basin			
ST-3	New aeration basin and appurtenances	2,229,000	New aeration basin with diffused aeration, internal recycle, selector zone, and provisions for operating in sludge re-aeration mode.
Tertiary filtration			
TT-1	New 12.5 mgd rapid-rate filters and appurtenances	3,333,000	Based on use of high-rate synthetic media filter units and appurtenances in outdoor covered structure.
Wet weather treatment (HRC)			
WWT-1	Ballasted floc HRC	3,520,000	Facility includes process equipment, chemical feed equipment, building, process tankage, and electrical. The facility would serve to convey and treat peak wet weather flows in parallel with secondary treatment. Based on use of Evoqua CoMag or Kruger Actiflo HRCs.

Table 8-3. Recommended WWTP Alternative Elements			
Item	Process area	Estimated construction cost ($) [a, b]	Notes
Existing CCT and disinfection improvements			
D-1	Upgrades to existing CCT and disinfection feed equipment	99,000	Includes new chemical feed pumps with appurtenances and upgrades to existing CCT including instrumentation, mixing, and appurtenances as required to optimize performance.
Wet weather disinfection facility			
D-2	New wet weather disinfection CT plus appurtenances and improvements to the existing disinfection CT	408,000	New wet weather tank sized to provide 15 minutes detention at 7.5 mgd in accordance with DEQ regulation plus disinfection equipment and appurtenances.
Outfall improvements			
OI-1	New outfall, diffuser, and upstream mixing/sampling structure	362,000	A new, larger-diameter outfall pipe replacing existing plus construction of new mixing/sampling structure for recombining and sampling of combined HRC + secondary effluents.
Biosolids handling			
BS-1	Solids holding tank and dewatering equipment improvements	1,187,000	Dewatering improvements (new screw press or BFP) and appurtenances for dewatering plus improvements to the solids holding tank including new blowers and diffusers for mixing and aeration of stored solids.
Civil site work			
CS-1	Site civil work including plumbing, grading, drainage, paving, landscaping, and restoration	532,000	Outside process piping improvements included with alternatives listed above.
Miscellaneous improvements			
M-1	RAS/WAS pumping improvements, hydraulic improvements to existing aeration basins, demolition of in-line grinders, concrete repair, etc.	479,000	RAS pumping improvements for increased capacity, replacement of miscellaneous valves in the RAS/WAS pump room, and hydraulic improvements to the CCT to alleviate the hydraulic bottleneck.
Standby generator			
SG-1	New standby generator set, automatic transfer switch (ATS), and controls	260,000	500 kW outdoor generator set in sound-attenuated enclosure, foundation, ATS switch, and controls, slab on grade.

a. *Estimate for planning purposes; AACEI Class 4 estimate ranges from -30% to +50%.*

b. *Construction costs incorporate markup on labor, materials, equipment, subcontractors, and a 35% contingency. Allowances for engineering, administration, construction management, and escalation are NOT included.*

8.3 Plant Classification

The City's WWTP is listed as a Level III Treatment System under its 2005 NPDES permit. The collection system is listed as Level II and Level III, with conditional statements identifying compliance with Schedule C of the permit. It is expected that the System Level for both the plant and collection system will remain Level III following planned upgrades. The City and/or contract operators shall provide staff to operate the plant with at least one member trained and rated as a Level III operator for both systems. No other certifications were identified as being needed beyond those identified herein.

8.4 Project Implementation Plan

This section describes the phasing of the recommended improvements.

8.4.1 Prioritizing Improvements

The evaluation of relative priority for each of the project elements is based on the following criteria:

- **Environmental:** This criterion considers whether a project element offers greater environmental protection via enhanced plant reliability, performance, or compliance.
- **Condition:** This item captures whether a project element is needed to address a portion of the plant that has deteriorated and is in need of upgrade or replacement.
- **Peak flow treatment:** This criterion applies to those elements that will increase the plant's ability to treat wet weather flows beyond the current 7 mgd capacity.
- **O&M:** This factor addresses whether a project element improves functionality, flexibility, or safety or reduces equipment wear or maintenance.

Table 8-4 shows an initial ranking in four priority levels (1 is highest priority, 4 is lowest) for each project element as a result of applying these criteria.

Table 8-4. Recommended Alternative Elements and Implementation Triggers

Improvement	Environmental	Condition	Peak flows	O&M	Priority
New IPS	✓	✓	✓		2
New IPS capacity expansion	✓		✓		3
Mechanical bar screening facilities (one screen)			✓	✓	3
Mechanical bar screening facilities (additional screen)				✓	4
Grit removal facilities				✓	4
Improvements for existing aeration basins	✓	✓	✓	✓	1
Third aeration basin	✓			✓	3
Clarifier upgrades	✓	✓	✓	✓	1
Parallel wet weather treatment facilities	✓		✓		3
Filtration upgrade	✓				4
Disinfection improvements	✓	✓	✓		2
Outfall upgrades	✓	✓	✓		2
Solids storage improvements		✓		✓	3
Solids dewatering improvements		✓		✓	3
Miscellaneous improvements	✓	✓	✓	✓	4
Larger standby generator	✓				2

In general, the project elements with the highest priority should be implemented sooner and those with the lowest priority implemented last. Exceptions to this rule are those middle-ranking project elements that are essential to the normal operation of the WWTP and that require replacement because of their condition. Replacement of existing dewatering equipment and improvements for the solids holding tank are two examples of this. Other exceptions are those project elements that rank highly but are not affordable until existing debt levels are reduced and revenue increases accumulate, for example the wet weather treatment facility.

8.4.2 Project Phasing and Cost Summary

Project phasing recommendations are based on the above discussion. Three phasing options were considered:

- Option A is a baseline condition that assumes all elements of the recommended alternative are constructed as a single project with midpoint of construction in 2020.
- Option B is a three-phase approach with construction midpoints ranging from 2020 to 2030.
- Option C is a four-phase approach that makes initial improvements immediately (construction midpoint in 2018); completes peak flow treatment upgrades next (midpoint 2025); and finishes with two phases of improvements (2035 and 2045) that are dependent on O&M needs, growth, and future permit limits.

The three options for phasing and their associated costs are shown below. The subtotal and total costs shown represent the fully weighted project costs developed by adding allowances for engineering, administration, construction management, and escalation to the construction costs provided in Sections 6 and 7.

8.4.3 Phasing Option A

Phasing Option A is shown in Table 8-5. Option A assumes that the entire project is completed in a single project in the near term. This requires a large capital investment. The impact this option has on customers is a revenue increase of approximately 185 percent over a 10-year period, which is unaffordable for city residents. This option is therefore not considered further.

Table 8-5. Estimated Capital Cost and Timing for Option A

Item	Description [a]	Cost ($)
Phase 1: midpoint of construction, 2020		
IP-1	Influent pumping	3,294,000
IP-2	Influent pumping capacity expansion	430,000
PT 1 + PT2	Mechanical bar screen (2 screens)	777,000
PT-3	Flow diversion pipe and structure	113,000
PT-4	Grit removal	1,994,000
WWT-1	Wet weather treatment (HRC)	3,520,000
ST-1	Aeration improvements for existing basins	626,000
D-1	Existing CCT and disinfection improvements	99,000
ST-3	New aeration basin	2,229,000
D-2	Wet weather disinfection facility	408,000
OI-1	Outfall improvements	362,000
BS-1	Biosolids handling	1,187,000
ST-2	Secondary clarifier improvements	447,000
TT-1	Tertiary filtration	3,333,000
SG-1	Standby generator	260,000
M-1	Miscellaneous improvements	479,000
CS-1	Civil site work	532,000
Total project cost for Option A including escalation [b]		**$29,113,000**

a. Individual line item costs are presented as the construction costs previously shown in Sections 6 and 7.

b. The total project cost includes 15% for engineering and administration, 10% for construction management, and 3% annual escalation to the midpoint of construction.

8.4.4 Phasing Option B

Phasing Option B is shown in Table 8-6. For this option, the recommended project would be completed in three phases. Selection of the project elements associated with the initial phase is based primarily on the above priority ranking. Some project elements cannot be separated without impacting other elements; for instance, a new IPS is required with the wet weather treatment facility to convey the peak flow through the WWTP. As a result, the first phase of Option B assumes a large project initiated early in the planning period requiring a large upfront capital investment. The impact of Option B is a revenue increase of approximately 163 percent over 10 years. This level of revenue adjustment is also not recommended because it is unaffordable for city residents. Therefore, this option was eliminated from further consideration.

Table 8-6. Estimated Capital Cost and Timing for Option B

Item	Description [a]	Cost ($)
Phase 1: midpoint of construction, 2020		
IP-1	Influent pumping	3,294,000
PT-1	Mechanical bar screen facility (one screen)	492,000
PT-3	Flow diversion pipe and structure	113,000
WWT-1	Wet weather treatment (HRC)	3,520,000
ST-1	Aeration improvements for existing basins	626,000
D-1	Existing CCT and disinfection improvements	99,000
D-2	Wet weather disinfection facility	408,000
OI-1	Outfall improvements	362,000
BS-1	Biosolids handling	1,187,000
ST-2	Secondary clarifier improvements	447,000
SG-1	Standby generator	260,000
M-1	Miscellaneous improvements	479,000
CS-1	Civil site work	370,000
	Subtotal project cost including escalation [b]	**$16,900,000**
Phase 2: midpoint of construction, 2025		
IP-2	Influent pumping capacity expansion	430,000
ST-3	New aeration basin	2,229,000
PT-2	Additional mechanical bar screen	285,000
CS-1	Civil site work	100,000
	Subtotal project cost including escalation [b]	**$5,100,000**
Phase 3: midpoint of construction, 2030		
TT-1	Tertiary filtration	3,333,000
PT-4	Grit removal	1,994,000
CS-1	Civil site work	62,000
	Subtotal project cost including escalation [b]	**$10,500,000**
Total project cost for Option B		**$32,500,000**

a. *Individual line item costs are presented as the construction costs previously shown in Sections 6 and 7.*

b. *The subtotal and total costs include 15% for engineering and administration, 10% for construction management, and 3% annual escalation to the midpoint of construction.*

8.4.5 Phasing Option C

Phasing Option C is shown in Table 8-7. For this option, the recommended project is phased in four separate projects. Like Option B, selection of the project elements associated with each of the phases is based primarily on the above-described priority evaluation, in some cases, on the impact to revenue requirements. Project elements with lower priority are deferred later in time. Related project elements are grouped as part of the same project, for instance, the new IPS is grouped with the wet weather treatment facilities because both are required for treatment of peak flows. Option C allows for the most economical breakdown of the project elements when considering the impact to revenue requirements. Based on affordability, Option C is the recommended phasing option. The impact of Option C is a revenue increase of approximately 33 percent over 10 years. Option C is the basis for the financial analysis provided in Section 9.

Table 8-7. Estimated Capital Cost and Timing for Option C

Item	Description [a]	Cost ($)
	Phase 1: midpoint of construction, July 2018	
ST-1	Aeration improvements for existing basins	626,000
OI-1	Outfall improvements	362,000
BS-1	Biosolids handling	1,187,000
ST-2	Secondary clarifier improvements	447,000
M-1	Miscellaneous improvements	479,000
	Subtotal project cost including escalation [b]	**$4,200,000**
	Phase 2: midpoint of construction, July 2025	
IP-1	Influent pumping	3,294,000
PT-1	Mechanical bar screen facility (one screen)	492,000
PT-3	Flow diversion pipe and structure	113,000
WWT-1	Wet weather treatment (HRC)	3,520,000
D-1	Existing CCT and disinfection improvements	99,000
D-2	Wet weather disinfection facility	408,000
SG-1	Standby generator	260,000
CS-1	Civil site work	265,000
	Subtotal project cost including escalation [b]	**$14,200,000**
	Phase 3: midpoint of construction, July 2035	
IP-2	Influent pumping capacity expansion	430,000
ST-3	New aeration basin	2,229,000
PT-2	Additional mechanical bar screen	285,000
CS-1	Civil site work	95,000
	Subtotal project cost including escalation [b]	**$6,900,000**
	Phase 4: midpoint of construction, July 2045	
TT-1	Tertiary filtration	3,333,000
PT-4	Grit removal	1,994,000
CS-1	Civil site work	172,000
	Subtotal project cost including escalation [b]	**16,700,000**
Total project cost for Option C		**$42,000,000**

a. *Individual line item costs are presented as the construction costs previously shown in Sections 6 and 7.*

b. *The sub total and total costs include 15% for engineering and administration, 10% for construction management, and 3% annual escalation to the midpoint of construction.*

8.4.6 Phasing Schedule

The proposed timeline for Option C is shown in Table 8-8.

Table 8-8. Timeline for Improvements, Midpoint of Construction	
Phase 1	July 2018
Phase 2	July 2025
Phase 3	July 2035
Phase 4	July 2045

Section 9
Financial Analysis

This section summarizes the characteristics of the City's user base, provides information on the existing wastewater rate structure, summarizes revenues and costs, and provides information on long-term debt and available reserves.

9.1 User Profile

User classifications include single-family residential, multifamily residential, and commercial/industrial. The number of accounts for each category of users is listed in Table 9-1.

Table 9-1. Summary of 2014 User Accounts

User	Number of accounts
Single-family residential	3,289
Multifamily residential	85
Commercial/industrial	256
Total	**3,630**

9.2 Rate Structure

Wastewater services for the City are funded entirely from service charges that are levied on all system users. Ongoing O&M costs and debt service are paid with the revenue from these fees. The City also charges system development charges (SDCs) for all new connections to the system. Revenue from these charges is restricted to capital expenditures.

9.2.1 User Fees

User fees are generally assessed based on both a flat fee and a commodity charge based on the user's water use during the non-irrigation season from November through April. Table 9-2 summarizes the existing rates from Resolution 19 for 2014. Commodity charges shown in Table 9-2 are based on the cubic feet (ft^3) of water used by customers.

Table 9-2. Existing Wastewater User Fees

Description	Monthly charge
Flat rate	\$36.70
Commodity charge, residential	\$6.45/100 ft^3 over 400 ft^3
Commodity charge, commercial: low strength	\$5.59/100 ft^3
Commodity charge, commercial: medium strength	\$6.77/100 ft^3
Commodity charge, commercial: high strength	\$8.77/100 ft^3
Unmetered residential	\$49.60

The overall average charge for a single-family residence is $44 per month. An example of a residential user that consumes an average of 5,000 gallons per month is as follows:

> 5,000 gallons is equivalent to 668 ft^3. Subtracting the base volume of 400 ft^3 and dividing by 100 ft^3 yields a multiplier of 2.68. Apply this rate to the standard residential commodity charge (second row in the table) and add the flat rate. This user pays a monthly user fee of approximately $17.30 + $36.70 = $54.00.

9.2.2 System Development Charges

New connections to the wastewater system are charged a SDC based on the size of their water service. A typical residential user with a 3/4-inch meter will pay $624. In addition, any user that is connected to the federally-funded sanitary sewer line is required to pay $900 per connection. Charges are based on Resolution 3 for 2005.

9.2.3 Resources

Resources for the wastewater program are derived primarily from user fees. Table 9-3 shows the actual resources for fiscal years (FY) 2012–13 and 2013–14, and the budgeted resources for 2014–15 and 2015–16.

Table 9-3. Wastewater Fund Resources

Description	Actual ($)		Budgeted ($)	
	FY 2012–13	FY 2013–14	FY 2014–15	FY 2015–16
Available cash on hand	621,715	360,514	19,613	119,466
Interest	2,890	0	1,501	1,005
User fee revenue	1,921,093	1,944,077	2,301,469	2,300,00
Miscellaneous	6,181	8,194	5,000	5,000
Total resources	**2,551,879**	**2,312,785**	**2,327,583**	**2,425,471**

The budgeted revenue from user fees for FY 2015–16 listed in Table 9-3 is essentially the same as that for the previous fiscal year.

9.2.4 Costs

Expenditures for wastewater services include personnel, materials and services, capital outlays, debt service, and administrative costs. The City contracts for the O&M of the WWTP and the contract cost for these services is included in materials and services. Administrative costs are paid by transfers to the general fund. Historical and budgeted costs are summarized in Table 9-4.

Table 9-4. Wastewater Fund Expenditures				
Description	Actual ($)		Budgeted ($)	
	2012–13	2013–14	2014–15	2015–16
Personnel services	584,918	577,421	583,103	601,283
Materials and services	546,640	587,547	633,410	647,601
Capital outlays	10,889	12,293	16,530	7,200
Transferred to other funds				
General fund administrative charges	111,300	0	111,034	114,366
Equipment reserve	0	3,500	3,500	11,210
Capital construction fund	0	0	0	0
Depreciation reserve	0	0	0	0
Operating contingency	0	0	40,427	41,374
Debt service	937,618	926,978	880,693	887,802
Total expenditures	**2,191,365**	**2,107,739**	**2,268,697**	**2,310,836**

As shown in Table 9-3, the total system resources are about $2.4M for FY 2015–16. This compares to a budgeted system cost of $2.3M, which provides limited operating reserves.

9.2.5 Debt Service

Loans taken by the City, summarized in Table 9-5, consist of indebtedness incurred to construct collection system improvements as mandated by DEQ. In December 2013, the City retired the debt for the most recent WWTP upgrade.

Table 9-5. Existing Loan Summary		
Description	Principal amount ($)	Final payment date
SRF Loan R89750	4,000,000	June 30, 2025
SRF Loan R89751	6,000,000	June 30, 2031
SRF Loan R89752	5,000,000	June 30, 2032

The total annual debt service for each bond issue is shown in Table 9-6. For the next 10 years, the annual debt service cost for existing debt is about $880k, which represents nearly 40 percent of the City's annual wastewater fund expenditures.

Table 9-6. Debt Service Costs				
FY	Loan number			Total debt service ($)
	R89750	R89751	R89752	
2015–16	280,561	336,616	270,625	887,802
2016–17	279,551	335,594	269,375	884,520
2017–18	278,509	334,543	268,125	881,177
2018–19	277,433	333,461	266,875	877,769
2019–20	276,324	332,347	265,625	874,296
2020–21	275,179	331,201	264,375	870,755
2021–22	273,999	330,021	263,125	867,145
2022–23	272,781	328,807	261,875	863,463
2023–24	271,524	327,557	260,625	859,706
2024–25	270,247	326,271	256,375	852,893
2025–26		324,947	258,125	583,072
2026–27		323,585	256,875	580,460
2027–28		322,182	255,625	577,807
2028–29		320,703	254,375	575,078
2029–30		319,254	253,125	572,379
2030–31		159,269	251,875	411,144
2031–32			250,625	250,625

9.2.6 Reserves

The City maintains a wastewater depreciation fund that is funded from rates. Funds from this reserve are designated for upgrades and maintenance of the wastewater collection system. As of July 1, 2015, the City had $1M available in this reserve. Most of this reserve has been budgeted for I/I correction, repairing sewer laterals, and professional services related to planning for MAO compliance.

Revenue from SDCs is deposited in the wastewater system development fund. These funds are restricted to projects designed to increase the capacity of the system or for system expansion. As of July 1, 2015, this fund had a balance of $378k, of which $215k has been budgeted for MAO compliance/system expansion.

9.3 Projected Resources and Expenditures

Growth in Sweet Home is relatively modest. Because the projections are long-term, an annual growth rate of 0.5 percent has been incorporated beginning in FY 2018–19. Currently, the rate of inflation is relatively modest but some level of increase should be anticipated. In Oregon, the cost of personnel services has increased at a higher rate than general inflation to help fund the public employee retirement system reserve shortfall. The following annual inflation rates are anticipated for operation of the wastewater fund:

- Labor 3.0 percent
- Materials and services 2.5 percent
- General fund administrative charge 3.0 percent
- Capital cost 1.0 percent

Based on this level of inflation, Table 9-7 shows projected wastewater fund balances.

Table 9-7. Wastewater Fund Projection			
Description	Projected ($)		
	2015–16	2016–17	2017–18
Resources			
Available cash on hand	119,466	159,554	165,694
Interest	1,500	1,500	1,500
User fees	2,301,000	2,301,000	2,301,000
Miscellaneous	5,000	5,000	5,000
Total resources	**2,426,966**	**2,467,054**	**2,473,194**
Expenditures			
Personnel services	601,000	619,000	638,000
Materials and services	649,000	665,000	682,000
Capital outlays	12,000	12,120	12,240
Transferred to other funds			
General fund administrative charges	114,000	117,000	121,000
Equipment reserve	3,610	3,720	3,830
Debt service	887,802	884,520	881,177
Total expenditures	**2,267,412**	**2,301,360**	**2,338,247**
Fund balance	**159,554**	**165,694**	**134,947**

Revenue from user fees shown in Table 9-7 does not include a sewer service fee increase through FY 2017–18. By 2016–17 the fund balance begins to decrease because expenditures are projected to be greater than revenue. Because capital will be needed by that time to fund WWTP improvements, an annual rate increase of 3 percent should be implemented beginning in FY 2016–17.

9.4 Capital Improvement Plan

Based on the plan identified in Section 8, the recommended phasing plan, shown again in Table 9-8, has an impact to the City's capital expenditure and revenue forecasts. Figure 9-1 shows the monthly revenue required from an average single-family residence for a period of 30 years for the selected phasing option.

The subtotal and total costs presented in Table 9-8, include engineering, contingencies, administration, and construction management. In addition, costs are escalated to the midpoint of construction at a rate of 3 percent per year.

Table 9-8. Estimated Capital Cost and Timing for Option C		
Item	Description [a]	Cost ($)
Phase 1: midpoint of construction, July 2018		
ST-1	Aeration improvements for existing basins	626,000
OI-1	Outfall improvements	362,000
BS-1	Biosolids treatment	1,187,000
ST-2	Secondary clarifier improvements	447,000
M-1	Miscellaneous improvements	479,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**$4,200,000**
Phase 2: midpoint of construction, July 2025		
IP-1	Influent pumping	3,294,000
PT-1	Mechanical bar screen facility (one screen)	492,000
PT-3	Flow diversion pipe and structure	113,000
WWT-1	Wet weather treatment (HRC)	3,520,000
D-1	Existing CCT and disinfection improvements	99,000
D-2	Wet weather disinfection facility	408,000
SG-1	Standby generator	260,000
CS-1	Civil site work	265,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**$14,200,000**
Phase 3: midpoint of construction, July 2035		
IP-2	Influent pumping capacity expansion	430,000
ST-3	New aeration basin	2,229,000
PT-2	Additional mechanical bar screen	285,000
CS-1	Civil site work	95,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**$6,900,000**
Phase 4: midpoint of construction, July 2045		
TT-1	Tertiary filtration	3,333,000
PT-4	Grit removal	1,994,000
CS-1	Civil site work	172,000
	Subtotal project cost including engineering, administration, construction management, and escalation [b]	**16,700,000**
	Total project cost for Option C	**$42,000,000**

a. Individual line item costs are presented as the construction costs previously shown in Sections 6 and 7.

b. The subtotal and total costs include 15% for engineering and administration, 10% for construction management, and 3% annual escalation to the midpoint of construction.

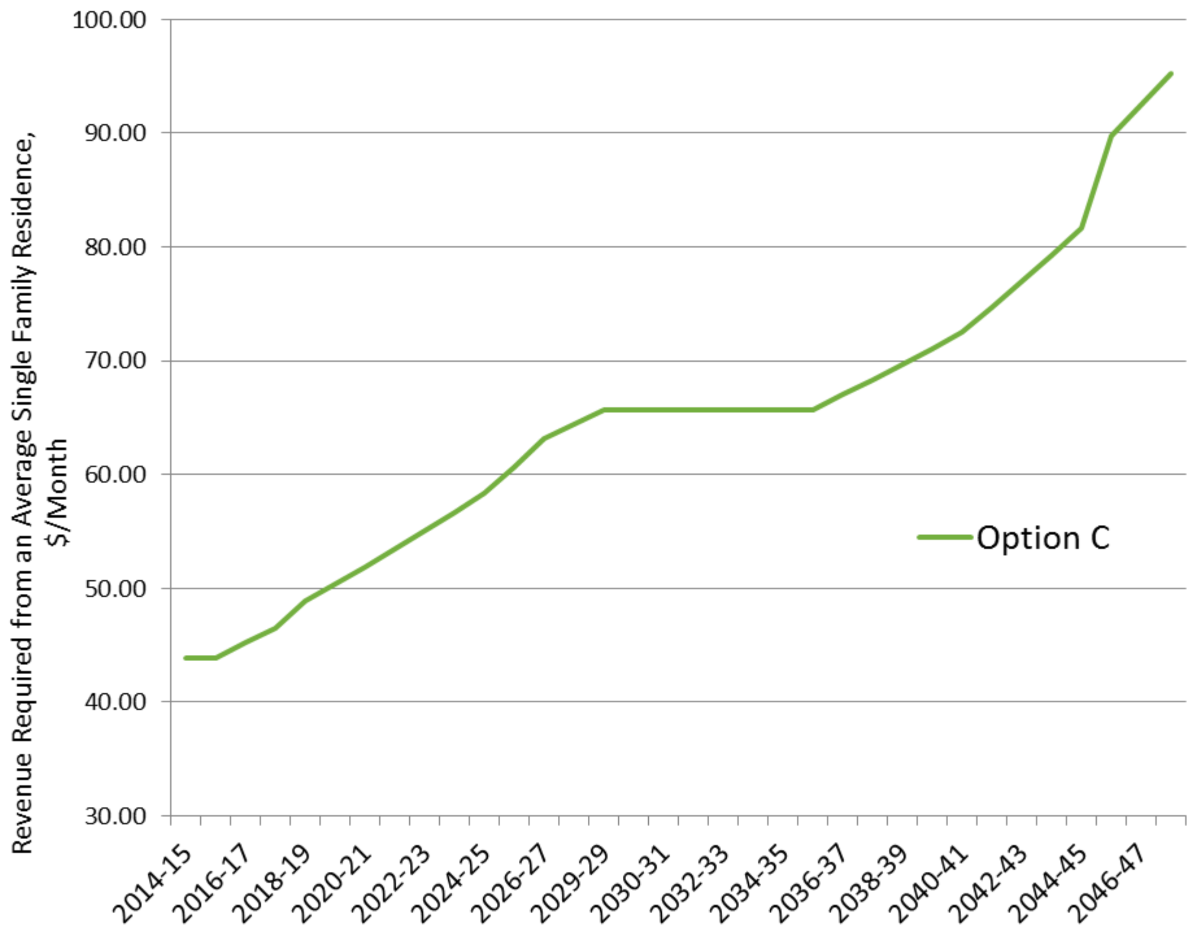


Figure 9-1. Monthly revenue required from an average single-family dwelling

The revenue estimates start at the 2015 estimated average monthly user fee of approximately $44 per month. Revenue requirements increase each year based on the escalation of planned costs plus applicable inflation rates. Steep increases shown in Figure 9-1 are related to the debt service required to support borrowing for capital improvements.

Table 9-9 presents a summary of City expenditures, revenues, and project expenses for the planning period. The table and associated Figure 9-1 show information beyond the planning period to demonstrate the revenue impacts of Phase 4.

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Table 9-9. Projected Revenue Requirements for Option C

Item	2015–16	2016–17	2017–18	2018–19	2019–20	2020–21	2021–22	2022–23	2023–24	2024–25	2025–26	2026–27	2027–28	2028–29	2029–30	2030–31	2031–32	2032–33	2033–34	2034–35	2035–36	2036–37	2037–38	2038–39	2039–40	2040–41	2041–42	2042–43	2043–44	2044–45	2045–46	2046–47	2047–48
Revenue																																	
Available cash on hand	119,466	159,554	208,694	175,947	112,378	101,042	144,007	243,322	400,059	618,293	815,070	579,398	444,058	346,091	285,563	191,434	211,240	353,255	705,585	1,014,595	1,238,285	1,035,645	860,655	714,315	595,615	505,555	445,125	457,315	545,115	710,515	856,505	516,075	303,225
Interest	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500
User fees	2,301,000	2,370,000	2,441,000	2,563,000	2,653,000	2,746,000	2,842,000	2,941,000	3,044,000	3,151,000	3,293,000	3,441,000	3,527,000	3,615,000	3,633,000	3,651,000	3,669,000	3,687,000	3,705,000	3,724,000	3,743,000	3,837,000	3,933,000	4,031,000	4,132,000	4,235,000	4,383,000	4,536,000	4,695,000	4,859,000	5,369,000	5,557,000	5,751,000
Miscellaneous	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000
Total resources	2,426,966	2,536,054	2,656,194	2,745,447	2,771,878	2,853,542	2,992,507	3,190,822	3,450,559	3,775,793	4,114,570	4,026,898	3,977,558	3,967,591	3,925,063	3,848,934	3,886,740	4,046,755	4,417,085	4,745,095	4,987,785	4,879,145	4,800,155	4,751,815	4,734,115	4,747,055	4,834,625	4,999,815	5,246,615	5,576,015	6,232,005	6,079,575	6,060,725
Expenditures																																	
Personnel services	601,000	619,000	638,000	657,000	677,000	697,000	718,000	740,000	762,000	785,000	809,000	833,000	858,000	884,000	911,000	948,000	976,000	1,005,000	1,035,000	1,066,000	1,098,000	1,131,000	1,165,000	1,200,000	1,236,000	1,273,000	1,311,000	1,350,000	1,391,000	1,433,000	1,476,000	1,520,000	1,566,000
Materials and services	649,000	665,000	682,000	699,000	716,000	734,000	752,000	771,000	790,000	810,000	830,000	851,000	872,000	894,000	916,000	939,000	962,000	986,000	1,011,000	1,036,000	1,062,000	1,089,000	1,116,000	1,144,000	1,173,000	1,202,000	1,232,000	1,263,000	1,295,000	1,327,000	1,360,000	1,394,000	1,429,000
Capital outlays	12,000	12,120	12,240	12,360	12,480	12,600	12,730	12,860	12,990	13,120	13,250	13,380	13,510	13,650	13,790	13,930	14,070	14,210	14,350	14,490	14,630	14,780	14,930	15,080	15,230	15,380	15,530	15,690	15,850	16,010	16,170	16,330	16,490
Transferred to other funds																																	
Gen. fund admin. charges	114,000	117,000	121,000	125,000	129,000	133,000	137,000	141,000	145,000	149,000	153,000	158,000	163,000	168,000	173,000	178,000	183,000	188,000	194,000	200,000	206,000	212,000	218,000	225,000	232,000	239,000	246,000	253,000	261,000	269,000	277,000	285,000	294,000
Equipment reserve	3,610	3,720	3,830	3,940	4,060	4,180	4,310	4,440	4,570	4,710	4,850	5,000	5,150	5,300	5,460	5,620	5,790	5,960	6,140	6,320	6,510	6,710	6,910	7,120	7,330	7,550	7,780	8,010	8,250	8,500	8,760	9,020	9,290
Existing debt service	887,802	884,520	881,177	877,769	874,296	870,755	867,145	863,463	859,706	852,893	583,072	580,460	577,807	575,078	572,379	411,144	250,625	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
New debt service		26,000	142,000	258,000	258,000	258,000	258,000	258,000	258,000	346,000	1,142,000	1,142,000	1,142,000	1,142,000	1,142,000	1,142,000	1,142,000	1,142,000	1,142,000	1,184,000	1,565,000	1,565,000	1,565,000	1,565,000	1,565,000	1,565,000	1,565,000	1,565,000	1,565,000	1,666,000	2,578,000	2,552,000	2,436,000
Total expenditures	2,267,412	2,327,360	2,480,247	2,633,069	2,670,836	2,709,535	2,749,185	2,790,763	2,832,266	2,960,723	3,535,172	3,582,840	3,631,467	3,682,028	3,733,629	3,637,694	3,533,485	3,341,170	3,402,490	3,506,810	3,952,140	4,018,490	4,085,840	4,156,200	4,228,560	4,301,930	4,377,310	4,454,700	4,536,100	4,719,510	5,715,930	5,776,350	5,750,780
Ending fund balance	**159,554**	**208,694**	**175,947**	**112,378**	**101,042**	**144,007**	**243,322**	**400,059**	**618,293**	**815,070**	**579,398**	**444,058**	**346,091**	**285,563**	**191,434**	**211,240**	**353,255**	**705,585**	**1,014,595**	**1,238,285**	**1,035,645**	**860,655**	**714,315**	**595,615**	**505,555**	**445,125**	**457,315**	**545,115**	**710,515**	**856,505**	**516,075**	**303,225**	**309,945**
Revenue increase, %	0.00	3.00	3.00	5.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	2.00	2.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.00	2.00	2.00	2.00	2.00	3.00	3.00	3.00	3.00	10.00	3.00	3.00
Approx. revenue required, $	43.89	45.21	46.56	48.89	50.36	51.87	53.42	55.03	56.68	58.38	60.71	63.14	64.40	65.69	65.69	65.69	65.69	65.69	65.69	65.69	65.69	67.01	68.35	69.71	71.11	72.53	74.71	76.95	79.26	81.63	89.80	92.49	95.27
Capital expenditures																																	
Engineering		420,000								1,440,000										690,000										1,650,000			
Construction			1,890,000	1,890,000							12,960,000										6,210,000										14,850,000		
Phase 1																																	
Total capital cost																																	
Debt service: engineering		26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	26,000	0	0
Debt service: construction			116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	0
Debt Service: construction				116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000	116,000
Total debt service for Phase 1		26,000	142,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	258,000	232,000	116,000
Phase 2																																	
Debt service: engineering										88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000	88,000
Debt service: construction											796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000	796,000
Total debt service for Phase 2										88,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000	884,000
Phase 3																																	
Debt service: engineering																				42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000	42,000
Debt service: construction																					381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000	381,000
Total debt service for Phase 3																				42,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000	423,000
Phase 4																																	
Debt service: engineering																														101,000	101,000	101,000	101,000
Debt service: construction																															912,000	912,000	912,000
Total debt service for Phase 4																														101,000	1,013,000	1,013,000	1,013,000

Section 10

Limitations

This document was prepared solely for the City of Sweet Home, Oregon (City) in accordance with professional standards at the time the services were performed and in accordance with contract amendment No. 10 between City and Brown and Caldwell (BC) for the contract dated August 27, 2001. This document is governed by the specific scope of work authorized by the City; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on information or instructions provided by the City and other parties and, unless otherwise expressly indicated, have made no independent investigation as to the validity, completeness, or accuracy of such information.

Appendix A: NPDES Permit

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 Sweet Home
1140 Twelfth Avenue
Sweet Home, OR 97386

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 31.5
Emergency Overflow		
Pump Station Overflow	002	Ames Creek R.M. 0.1

FACILITY TYPE AND LOCATION:

Activated Sludge
Sweet Home STP
1357 Pleasant Valley Road
Sweet Home
Treatment System Class: Level III
Collection System Class: Level II (prior to compliance with Schedule C, Condition 6)
Collection System Class: Level III (after compliance with Schedule C, Condition 6)

RECEIVING STREAM INFORMATION:

Basin: Willamette
Sub-Basin: South Santiam
Receiving Stream: South Santiam
Hydro Code: 1230064446867 31.5 D
County: Linn

EPA REFERENCE NO: OR002034-6

Issued in response to Application No. 983953 received November 10, 2003. This permit is issued based on the land use findings in the permit record.

for Mark E Hamlin
Michael H. Kortenhof, Western Region Water Quality Manager

April 22, 2005
Date

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 wastewater 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

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
$CBOD_5$ (See Note 1)	10 mg/L	15 mg/L	120	180	240
TSS	10 mg/L	15 mg/L	120	180	240

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
$CBOD_5$ (See Note 1)	15 mg/L	23 mg/L	290	460	630
TSS	20 mg/L	30 mg/L	350	520	690

* Average dry weather design flow to the facility equals 1.38 MGD. Mass load limits have been individually assigned and are based upon prior permit.

(3) Other parameters (year-round except as noted) (see Note 2)

Parameter	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 3)
pH	Shall be within the range of 6.3 - 9.0
$CBOD_5$ and TSS Removal Efficiency (May through October)	Shall not be less than 85% monthly average for $CBOD_5$ and TSS.
$CBOD_5$ and TSS Removal Efficiency (November through April)	Shall not be less than 70% monthly average for $CBOD_5$ and TSS.
Total Residual Chlorine	Shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum concentration of 0.05 mg/L (see Note 4)
Ammonia-N (May through October)	Shall not exceed a monthly average concentration of 5.1 mg/L and a daily maximum concentration of 11 mg/L (see Note 5)

(4) 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 South Santiam River extending from a point ten (10) feet upstream of the outfall to a point one hundred (100) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within ten (10) feet of the point of discharge.

b. Emergency Overflow Outfall 002

(1) No wastes shall be discharged from these outfalls, unless the cause of the discharge is due to storm events as allowed under OAR 340-041-0009 (6) or (7) as follows:

(2) Raw sewage discharges are prohibited to waters of the State 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.

c. 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).

d. Septage shall not be accepted at this facility for treatment or processing without written approval from the Department.

NOTES:

1. The $CBOD_5$ concentration limits are considered equivalent to the minimum design criteria for BOD_5 specified in Oregon Administrative Rules (OAR) 340-041. These limits and $CBOD_5$ mass limits may be adjusted (up or down) by permit action if more accurate information regarding $CBOD_5/BOD_5$ becomes available.

2. No thermal load limits were proposed in this permit. This permit may be re-opened, and new temperature and/or thermal load limits assigned upon approval of a Total Maximum Daily Load for temperature for this sub-basin, or when more accurate effluent temperature data becomes available.

3. 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.

4. When the total residual chlorine limitation is lower than 0.10 mg/L, the Department will use 0.10 mg/L as the compliance evaluation level (i.e. daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitation).

5. The ammonia limits in Schedule A, Condition 1.a (3) shall become effective upon completion of the compliance schedule contained in Schedule C, Condition 5. The ammonia limits are based on the estimated dilution in the mixing zone and the 1986 EPA Gold Book Criteria. The limits are considered interim. The permittee may request that this permit be re-opened, and the limits modified or eliminated upon completion of the mixing zone dilution study required by Schedule C, Condition 4. In addition, 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:

Parameter	Limitations
Ammonia-N	No limit

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 samples and measurements are taken at the headworks on the aeration basins. Composite samples are taken from the pump station wet well. The composite sampler is located in the control building.

Item or Parameter	Minimum Frequency	Type of Sample
$CBOD_5$	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent grab samples and measurements are taken after the final weir. Composite samples are taken just before the final weir. The composite sampler is located in the control building.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
$CBOD_5$	2/Week	Composite
Ammonia (NH3-N)	2/Week	Composite
TSS	2/Week	Composite
Hardness (mg/L $CaCO_3$)	See Notes 1 and 2	Grab
pH	3/Week	Grab
E. coli	2/Week	Grab (See Note 3)
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
Pounds Discharged ($CBOD_5$ and TSS)	2/Week	Calculation
Average Percent Removed ($CBOD_5$ and TSS)	Monthly	Calculation
Effluent Temperature, Daily Max	Daily	Continuous (see Note 4)
Toxics:		
Cadmium, copper, lead, mercury, silver and zinc (measured as total in mg/L) and cyanide	Semi-annually (See Note 1)	24-hour Composite (See Note 5)
Whole Effluent Toxicity	Annually (See Note 6)	Acute & chronic
Priority Pollutants	(See Note 2)	24-hour Composite

b. Treated Effluent Outfall 001 (Continued)

Item or Parameter	Minimum Frequency	Type of Sample
Nutrients (see Note 7):		
TKN, NO2+NO3-N, Total Phosphorus	1/Week (May-Oct)	24-hour Composite

c. Biosolids Management

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 (See Note 8)
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.
Quantity and type of alkaline product used to stabilize biosolids (when required to meet federal pathogen and vector attraction reduction requirements in 40 CFR 503.32(b)(3) and 40 CFR 503.33(b)(6))	Each occurrence	Measurement
Initial time when solids that received alkaline agent ascended to pH >= 12	Each batch	Date, time, and actual pH measurement (corrected to standard at 25°C)
2 hours after initial alkaline addition and sustained at pH >= 12	Each batch	Date, time, and actual pH measurement (corrected to standard at 25°C)
24 hours after initial alkaline addition and pH >= 11.5 was sustained	Each batch	Date, time, and actual pH measurement (corrected to standard at 25°C)

d. Emergency Overflow Outfall 002

Item or Parameter	Minimum Frequency	Type of Sample
Flow	Daily (during each occurrence)	Duration and volume

e. South Santiam River

Item or Parameter	Minimum Frequency	Type of Sample
Cadmium, copper, lead, mercury, silver and zinc (measured as total in mg/L) and cyanide	Semi-annually (See Note 9)	Grab
TSS	See Note 9	Grab
Hardness (mg/L $CaCO_3$)	See Note 9	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 appropriate Department 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. 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. During the first two years after permit issuance, special monitoring for cadmium, copper, lead, mercury, silver, zinc and cyanide shall be conducted on the effluent at the specified frequency. TSS and hardness shall be monitored simultaneously. The special monitoring for cadmium, copper, lead, 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.

2. 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. 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.

3. *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.		

4. When continuous monitors are used, indicate the time interval between temperature readings, and results are to be tabulated and submitted in an annual report. Continuous temperature monitors must be audited in June and December, following procedures described in DEQ Procedural Guidance for Water Temperature Monitoring. Continuous temperature monitors are to be checked visually monthly to insure that the devices are still in place and submerged.

5. 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.

6. 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.

7. Starting in 2006, the permittee shall monitor nutrients at the specified frequency and season for two years. After two years, nutrient monitoring of the effluent may be eliminated unless otherwise notified in writing by the Department.

8. Composite samples from the Dewatered biosolids shall be taken from reference areas in the Dewatered biosolids pursuant to <u>Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.</u>

   Inorganic pollutant monitoring must be conducted according to <u>Test Methods for Evaluating Solid Waste, Physical/Chemical Methods</u>, Second Edition (1982) with Updates I and II and third Edition (1986) with Revision I.

9. During the first year after permit issuance, the South Santiam River shall be monitored for cadmium, copper, lead, mercury, silver, zinc and cyanide shall be conducted on the effluent at the specified frequency. TSS and hardness shall be monitored simultaneously. The special monitoring for cadmium, copper, lead, 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 year, South Santiam River monitoring 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. Within 180 days of permit issuance, the permittee shall submit to the Department for review and approval an updated proposed program and time schedule for identifying and reducing inflow. Within 60 days of receiving written Department comments, the permittee shall submit a final approvable program and time schedule. The program shall consist of the following:

   a. Identification of all overflow points and determination that sewer system overflows are or are not occurring up to a 24-hour, 5-year storm event or equivalent;

   b. Monitoring of all pump station overflow points;

   c. A program for identifying and removing all inflow sources into the permittee's sewer system over which the permittee has legal control; and

   d. If the permittee does not have the necessary legal authority for all portions of the sewer system or treatment facility, a program and schedule for gaining legal authority to require inflow reduction and a program and schedule for removing inflow sources.

2. The permittee shall annually appropriate and expend a minimum of $50,000 exclusively for the purpose of identifying and reducing inflow and infiltration into the sewage collection system. Qualified expenditures shall not include routine maintenance, repairs, cleaning or unplugging activities. An annual report shall be submitted to the Department by March 1 each year which details the following items:

   a. A summary of sewer collection maintenance activities and associated expenditures that have been done in the previous year.

   b. An analysis of sewer system flow data that evaluates the effectiveness of the City's efforts to control and reduce inflow and infiltration.

   c. A summary and associated budget of maintenance activities scheduled for the upcoming year for identifying and reducing inflow and infiltration.

   d. Documentation as necessary to verify that at a minimum of $50,000 have been expended for the purpose of reducing inflow and infiltration into the sewage collection system.

3. Industrial Waste Survey Update/Pretreatment Program

   a. As soon as practicable, but by no later than six (6) months from permit issuance date, the permittee shall submit to the Department an update to the industrial waste survey. The update should be completed as described in 40 CFR 403.8(f)(2)(i-iii) and 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.

4. By no later than December 31, 2005, the permittee must submit for Department approval a plan and schedule for conducting a mixing zone dilution study using a dye study or other Department approved method.. By no

later than one year after Department approval, the permittee must submit the results of the study to the Department. If the dilution achieved is significantly different than the computer model prediction, the permittee may request a permit modification to adjust the total residual chlorine limit and/or the ammonia limit, and/or other limits, as appropriate

5. By no later than June 30, 2007, the permittee shall submit an evaluation of whether or not the discharge has the potential to violate the ammonia limits. If the evaluation indicates the permittee has a reasonable potential to violate the ammonia limits, the permittee shall complete the following schedule:

   a. By no later than December 31, 2007, the permittee shall submit to the Department an evaluation of alternatives for corrective action that will result in compliance with the ammonia limits.

   b. By no later than December 31, 2008, the permittee shall submit to the Department for approval final engineering plans and specifications for the corrective actions necessary to comply with the ammonia limits.

   c. By no later than December 31, 2009, the permittee shall complete construction of all necessary improvements and comply with the ammonia limits.

6. By no later than December 31, 2006, the permittee shall provide one or more collection system operators who hold valid certification at Level III or above.

7. By no later than December 31, 2006, the permittee shall submit an evaluation of whether or not the biosolids processing can consistently comply with the vector attraction and pathogen reduction requirements in 40 CFR Part 503. If the biosolids processing cannot consistently comply, the submittal must include proposed plan and schedule for coming into compliance. Upon Department approval of the plan and schedule, the permittee shall implement the plan.

8. 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. Unless otherwise approved in writing by the Department, all inflow sources are to be permanently disconnected from the sanitary sewer system.

2. 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 Linn County. All currently approved sites are located in Linn 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.

3. 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.

4. **Whole Effluent Toxicity Testing**

   a. The permittee shall conduct Whole Effluent Toxicity (WET) tests as specified in Schedule B of this permit.

   b. 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). Chronic tests shall be run using the following dilution series: 12.5%, 25%, 50%, 75%, and 100%

   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 test 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 test 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.

5. 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 5.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 5.b. above.

6. 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.

7. 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.

SCHEDULE F
NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

SECTION A. STANDARD CONDITIONS

1. Duty to Comply with Permit
The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of Oregon Revised Statutes (ORS) 468B.025, and 40 Code of Federal Regulations (CFR) Section 122.41(a), and grounds for an enforcement action. Failure to comply is also grounds for the Department to modify, revoke, or deny renewal of a permit.

2. Penalties for Water Pollution and Permit Condition Violations
ORS 468.140 allows the Department to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit. Additionally 40 CFR 122.41 (A) provides that any person who violates any permit condition, term, or requirement may be subject to a federal civil penalty not to exceed $25,000 per day for each violation.

Under ORS 468.943 and 40 CFR 122.41(a), unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000 imprisonment for not more than one year, or both. Each day on which a violation occurs or continues is a separately punishable offense.

Under ORS 468.946, a person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a Class B felony punishable by a fine not to exceed $200,000 and up to 10 years in prison. Additionally, under 40 CFR 122.41(a) any person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a federal civil penalty not to exceed $100,000, and up to 6 years in prison.

3. Duty to Mitigate
The permittee must take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit that has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee must correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application must be submitted at least 180 days before the expiration date of this permit.

The Department may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions
This permit may be modified, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge
d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a Total Maximum Daily Load (TMDL)
e. New information or regulations
f. Modification of compliance schedules
g. Requirements of permit reopener conditions
h. Correction of technical mistakes made in determining permit conditions
i. Determination that the permitted activity endangers human health or the environment

j. Other causes as specified in 40 CFR 122.62, 122.64, and 124.5

The filing of a request by the permittee for a permit modification, revocation or reissuance, termination, or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants
The permittee must comply with any applicable effluent standards or prohibitions established under Oregon Administrative Rules (OAR) 340-041-0033 for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights and Other Legal Requirements
The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege, or authorize any injury to persons or property or invasion of any other private rights, or any infringement of federal, tribal, state, or local laws or regulations.

8. Permit References
Except for effluent standards or prohibitions established under OAR 340-041-0033 for toxic pollutants and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

9. Permit Fees
The permittee must pay the fees required by Oregon Administrative Rules.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance
The permittee must at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) that are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems that are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Duty to Halt or Reduce Activity
For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee must, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It is not a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions
(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation or the diversion is due to nonuse of nonessential treatment units or processes at the treatment facility.
(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes that causes them to become inoperable, or substantial and permanent loss of natural resources that can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.
(1) Bypass is prohibited unless:
(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;
(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of

equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass that occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Department may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Department determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, a written notice must be submitted to the Department at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee must submit notice of an unanticipated bypass as required in General Condition D.5.

4. Upset

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology-based permit effluent limitations if the requirements of General Condition B.4.c are met. A determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance is not final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the causes(s) of the upset;

(2) The permitted facility was at the time being properly operated;

(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and

(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. Treatment of Single Operational Upset

For purposes of this permit, A Single Operational Upset that leads to simultaneous violations of more than one pollutant parameter will be treated as a single violation. A single operational upset is an exceptional incident that causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational upset does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational upset is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.

(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of storm related overflows. Storm related overflows of raw sewage are prohibited to waters of the State. However, the Environmental Quality Commission (EQC) recognizes that it is impossible to design and construct a conveyance system that will prevent overflows under all storm conditions. The State of Oregon has determined that all wastewater conveyance systems should be designed to transport storm events up to a specific size to the treatment facility. Therefore, such storm related overflows will not be considered a violation of this permit if:

(1) The permittee has conveyance and treatment facilities adequate to prevent overflows except during a storm event greater than the one-in-five-year, 24-hour duration storm from November 1 through May 21 and except during a storm event greater than the one-in-ten-year, 24-hour duration storm from May 22 through October 31;

(2) The permittee has provided the highest and best practicable treatment and/or control of wastes, activities, and flows and has properly operated the conveyance and treatment facilities in compliance with General Condition B.1.;

(3) The permittee has properly implemented a Department approved Overflow Response Plan; and

(4) The permittee has implemented a program to evaluate and maintain the capacity of the conveyance system

c. Prohibition of other overflows. All overflows other than stormwater-related overflows (discussed in Schedule F, Section B, Condition 6.b.) are prohibited unless:

(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and

(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

d. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

e. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5. Reports concerning storm related overflows must include information about the amount and intensity of the rainfall event causing the overflow.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee must take such steps as are necessary to alert the public about the extent and nature of the discharge. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must be disposed of in such a manner as to prevent any pollutant from such materials from entering waters of the state, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein must be representative of the volume and nature of the monitored discharge. All samples must be taken at the monitoring points specified in this permit, and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points may not be changed without notification to and the approval of the Department.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices must be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges.

The devices must be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected must be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures
Monitoring must be conducted according to test procedures approved under 40 CFR part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering
The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit may, upon conviction, be punished by a fine of not more than $10,000 per violation, imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

5. Reporting of Monitoring Results
Monitoring results must be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports must be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee
If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR part 136 or as specified in this permit, the results of this monitoring must be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency must also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value must be recorded unless otherwise specified in this permit.

7. Averaging of Measurements
Calculations for all limitations that require averaging of measurements must utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records
Except for records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR part 503). The permittee must retain records of all monitoring information, including: all calibration, maintenance records, all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit for a period of at least 3 years from the date of the sample, measurement, report, or application. This period may be extended by request of the Department at any time.

9. Records Contents

Records of monitoring information must include:

a. The date, exact place, time, and methods of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

10. Inspection and Entry

The permittee must allow the Department representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and
d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes
The permittee must comply with OAR chapter 340, division 52, "Review of Plans and Specifications" and 40 CFR Section 122.41(l) (1). Except where exempted under OAR chapter 340, division 52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and specifications are submitted to and approved by the Department. The permittee must give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance
The permittee must give advance notice to the Department of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

3. Transfers
This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit may be transferred to a third party without prior written approval from the Department. The Department may require modification, revocation, and reissuance of the permit to change the name of the permittee and incorporate such other requirements as may be necessary. The permittee must notify the Department when a transfer of property interest takes place.

4. Compliance Schedule
Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit must be submitted no later than 14 days following each schedule date. Any reports of noncompliance must include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting
The permittee must report any noncompliance that may endanger health or the environment. Any information must be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. During normal business hours, the Department's Regional office must be called. Outside of normal business hours, the Department must be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission must also be provided within 5 days of the time the permittee becomes aware of the circumstances. Pursuant to ORS 468.959 (3) (a), if the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days of the time the permittee becomes aware of the circumstances. The written submission must contain:
a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected;
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
e. Public notification steps taken, pursuant to General Condition B.7

The following must be included as information that must be reported within 24 hours under this paragraph:
f. Any unanticipated bypass that exceeds any effluent limitation in this permit;
g. Any upset that exceeds any effluent limitation in this permit;
h. Violation of maximum daily discharge limitation for any of the pollutants listed by the Department in this permit; and
i. Any noncompliance that may endanger human health or the environment.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. Other Noncompliance
The permittee must report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports must contain:
a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information
The permittee must furnish to the Department within a reasonable time any information that the Department may request to determine compliance with this permit. The permittee must also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it has failed to submit any relevant facts or has submitted incorrect information in a permit application or any report to the Department, it must promptly submit such facts or information.

8. Signatory Requirements
All applications, reports or information submitted to the Department must be signed and certified in accordance with 40 CFR Section 122.22.

9. Falsification of Information
Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $100,000 per violation and up to 5 years in prison. Additionally, according to 40 CFR 122.41(k)(2), any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

10. Changes to Indirect Dischargers
The permittee must provide adequate notice to the Department of the following:
a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;
b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.
2. CBOD means five day carbonaceous biochemical oxygen demand
3. TSS means total suspended solids.
4. "Bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
5. FC means fecal coliform bacteria.
6. Total residual chlorine means combined chlorine forms plus free residual chlorine
7. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR Section 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR Chapter 340, Division 41.
8. mg/l means milligrams per liter.
9. kg means kilograms.
10. m^3/d means cubic meters per day.
11. MGD means million gallons per day.

12. 24-hour Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR part 136.
13. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.
14. Quarter means January through March, April through June, July through September, or October through December.
15. Month means calendar month.
16. Week means a calendar week of Sunday through Saturday.
17. POTW means a publicly owned treatment works

Update 2-28-05 AR der
Update 3-16-05 PN 199693 der

Appendix B: Mutual Agreement and Order (MAO)/ MAO Termination Letter

Rec'd 12-5-05

Oregon
Theodore R. Kulongoski, Governor

Department of Environmental Quality
Western Region - Salem Office
750 Front St. NE, Ste. 120
Salem, OR 97301-1039
(503) 378-8240
(503) 378-3684 TTY

December 2, 2005

Mr. Michael J. Adams
Public Works Director
1140 12th Avenue
Sweet Home, OR 97386

RE: **City of Sweet Home**
Addendum No. 3 to
Mutual Agreement and Order (MAO) #WQ/M-WR-98-221
File No. 86840
EPA #OR-002034-6
Linn County
Deadline Modification

Dear Mr. Adams:

The Department of Environmental Quality (Department) has taken into consideration the City of Sweet Home's (City) request to modify the compliance deadline in Paragraph 10.A(4) of Mutual Agreement and Order (MAO) No. WQ/M-WR-98-221. Pursuant to Section 12 of the MAO, the Department has determined that the August 30, 2004 written request includes the description of need for modification.

The Department hereby approves the City of Sweet Home's request. Now, therefore, Paragraph 10.A (4) shall be as follows:

> 10.A (4) By no later than January 1, 2010, the Permittee shall comply with all applicable water quality standards and treat all flows up to the one-in-five-year, 24-hour storm event.

All other Conditions and Schedules contained within the MAO shall remain the same. If you have any questions, please call Raghu Namburi, (503) 378-8240, extension 233, in the Western Region-Salem office.

Sincerely,

Kerri L. Nelson
Western Region Administrator

cc: Enforcement Section, DEQ
Raghu Namburi, DEQ – Salem Office
Dottie Reynolds, DEQ- Salem Office

DEQ/WVR-101 1-03

Oregon
John A. Kitzhaber, M.D., Governor

Department of Environmental Quality
Western Region
1102 Lincoln
Suite 210
Eugene, OR 97401
(541) 686-7838

May 9, 2001

Mr. Michael J. Adams, Director of Public Works
City of Sweet Home
1730 North Ninth Avenue
Sweet Home, OR 97386

Re: City of Sweet Home
Addendum No. 2 to
Mutual Agreement and Order (MAO) WQ/M-WR-98-221
File No. 86840
EPA #OR-002034-6
Deadline Modification

Dear Mr. Adams:

The Department of Environmental Quality (Department) has taken into consideration the City of Sweet Home's (City) request to modify the compliance deadlines in Paragraph 10.A (1), (2) and (4) of Mutual Agreement and Order (MAO) WQ/M-WR-98-221. Pursuant to Section 12 of the MAO, the Department has determined that the March 26, 2001 written request includes the description of need for modification.

The Department hereby approves the City of Sweet Home's request. Now, therefore, Paragraph 10.A (1), (2) and (4) shall be as follows:

> 10.A (1) By no later than May 31, 2002, the Permittee shall submit to the Department a report detailing wastewater collections system flows as determined using the Department approved flow monitoring plan. If the report indicates that the collection system flows continue to exceed the capacity of the wastewater collection and treatment facilities during the one-in-five-year, 24-hour storm event, the Permittee shall comply with the remainder of this schedule.
>
> 10.A (2) By no later than January 31, 2003, the Permittee shall submit a draft facility plan and time schedule that evaluates alternatives for either increasing treatment capacity or reducing raw sewage flows down to the current treatment capacity by not later than October 31, 2005. The facility

DEQ/WR-101 4-98

plan shall describe the collection and treatment facilities necessary to comply with all applicable water quality standards including treatment of all flows up to the one-in-five-year, 24-hour storm event. Within ninety (90) days of receiving written Department comments, the Permittee shall submit a final approvable facilities plan. If the final facilities plan includes treatment facilities that are new, or have increased treatment capacity, the plan shall include an application for a new or modified NPDES permit for the proposed facility.

10.A (4) By no later than October 31, 2007, the Permittee shall comply with all applicable water quality standards and treat all flows up to the one-in-five-year, 24-hour storm event. Moved to Jan 1, 2010

If you have any questions regarding this matter, please call Mark Hamlin, at (503) 378-8240, extension 239, in the Western Region-Salem office.

Sincerely,

Kerri L. Nelson, Western Region Administrator

e:\winword\sweethome\mao2.doc
cc: Western Region, Salem Office
Enforcement Section
EPA-OOO
EPA-Region X

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF: CITY OF SWEET HOME, Permittee,	)))))	ADDENDUM 1 TO MUTUAL AGREEMENT AND ORDER No. WQ/M-WR-98-221 LINN COUNTY

WHEREAS:

1. On January 31, 1992, the Department of Environmental Quality (Department or DEQ) issued National Pollutant Discharge Elimination System Permit Number 100856 (Permit) to the City of Sweet Home (Permittee). The Permit authorizes the Permittee to construct, install, modify or operate wastewater collection, treatment, control and disposal facilities in conformance with the requirements, limitations and conditions set forth in the Permit. The Permit expired on December 31, 1996. The Permit is in effect on this date as the Permittee has made a timely application for renewal of the Permit.

2. The Permittee submitted a flow study report to the Department dated February 18, 1997 that estimated the one-in-five-year, 24-hour storm. The wastewater flows associated with that storm are significantly greater than what the present wastewater facilities can transport and treat. Therefore, an expansion of the wastewater treatment works is likely necessary.

3. Condition 1 of Schedule A of the Permit specifies certain effluent discharge limits for the Permittee's wastewater treatment facilities. On occasion, the Permittee has not been able to comply with these limits.

4. General Condition B.6.b of the Permit prohibits overflows of raw sewage from the wastewater collection system. The Permittee has experienced several overflows each year from the wastewater collection system during heavy storm events.

5. During the time period the Permit has been in effect, Permittee has not met the above conditions in violation of Oregon Revised Statutes (ORS) 468B.025(2), Oregon Administrative Rules (OAR) 340-45-015(5)(b), and the Permit. Failure to comply with Permit

requirements caused the Department to issue Notices of Noncompliance (NON) to the Permittee on June 27, 1996 and March 23, 1998.

6. DEQ and the Permittee recognize that until Permittee completes the actions required by this Mutual Agreement and Order (MAO), Permittee will continue to violate the Permit and Oregon law.

7. The Permittee presently is capable of meeting the following interim effluent limitations, measured as specified in the Permit for wastewater flows up to seven (7) million gallons per day (MGD):

A. June 1 - September 30:

Permittee shall comply with summer time and year-round limits in Schedule A.

B. October 1 - May 31:

Permittee shall comply with winter time and year-round limits in Schedule A except as noted below:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
$CBOD_5$	15 mg/l	23 mg/l	630	1300	1800
TSS	20 mg/l	30 mg/l	840	1800	2300

On any day that the daily flow to the treatment facility exceeds 2.76 MGD (twice the average dry weather design flow of 1.38 MGD), the daily mass load limit shall not apply and pH shall be within the range 5.5 to 9.0. Wastewater flows in excess of seven (7) MGD may be overflowed to Ames Creek.

8. The Department and Permittee recognize that the Environmental Quality Commission has the power to impose a civil penalty and to issue an abatement order for violations of conditions of the Permit. Therefore, pursuant to ORS 183.415(5), the Department and Permittee wish to settle those past violations referred to in Paragraph 3, 4 and 5 and to limit and resolve the future violations referred to in Paragraph 6 in advance by this MAO.

9. This MAO is not intended to limit, in any way, the Department's right to proceed against Permittee in any forum for any past or future violations not expressly settled herein.

NOW THEREFORE, it is stipulated and agreed that:

10. The Environmental Quality Commission shall issue a final order:

A. Requiring Permittee to comply with the following schedule:

(1) By no later than May 31, ~~2001~~ 2002, the Permittee shall submit to the Department a report detailing wastewater collections system flows as determined using the Department approved flow monitoring plan. If the report indicates that the collection system flows continue to exceed the capacity of the wastewater collection and treatment facilities during the one-in-five-year, 24-hour storm event, the Permittee shall comply with the remainder of this schedule.

(2) By no later than January 31, ~~2002~~ 2003, the Permittee shall submit a draft facility plan and time schedule that evaluates alternatives for either increasing treatment capacity or reducing raw sewage flows down to the current treatment capacity by not later than October 31, 2005. The facility plan shall describe the collection and treatment facilities necessary to comply with all applicable water quality standards including treatment of all flows up to the one-in-five-year, 24-hour storm event. Within ninety (90) days of receiving written Department comments, the Permittee shall submit a final approvable facilities plan. If the final facilities plan includes treatment facilities that are new, or have increased treatment capacity, the plan shall include an application for a new or modified NPDES permit for the proposed facility.

(3) By no later thanMarch 1, 2002, and each subsequent March 1 while this MAO is in effect, the Permittee shall submit to the Department an annual progress report summarizing the corrective actions completed in the previous year. → NOW JAN 1, 2010

(4) By no later than October 31, ~~2005~~ 2007, the Permittee shall comply with all applicable water quality standards and treat all flows up to the one-in-five-year, 24-hour storm event.

(5) If the Permittee fails to comply with the requirements of Paragraph 10.A(1) through (4) above and does not achieve corrective actions in accordance with the time schedules identified in the approved facilities plan, the Permittee, upon written notification from the Department, shall not allow any new sewer connections to their sanitary sewer system. Each individual connection shall be a separate violation. The connection prohibition shall remain in effect until the Permittee submits a written request and documentation acceptable to the Department that the Permittee has come into compliance with the time schedule.

B. Requiring the Permittee to meet the interim effluent limitations set forth in Paragraph 7 above until completion of necessary corrective actions as required by the schedule in Paragraph 10.A. In addition, Permittee may discharge raw, untreated sewage that is in excess of an instantaneous flow of 7.0 MGD provided:

(1) The sewerage facilities shall be operated as effectively as practicable to minimize the discharges of untreated sewage;

(2) Incoming sewage that is not in excess of an instantaneous flow of 7.0 MGD shall be treated and meet the effluent limitations in Paragraph 7; and,

(3) The Permittee fully implements the approved plan required in Paragraph 10.A.(1).

C. Requiring Permittee, upon receipt of a written notice from the Department for any violations of this MAO, to pay the following civil penalties:

(1) $250 for each day of each violation of the schedule of compliance set forth in Paragraph 10.A.

(2) $500 for each violation of an interim monthly average waste discharge limitation set forth in Paragraph 10.B.

(3) $100 for each violation of an interim weekly average or daily maximum waste discharge limitation set forth in Paragraph 10.B., or for any other condition of this MAO.

(4) $10,000 for each violation of the sanitary sewer system connection prohibition set forth in Paragraph 10.A (5).

11. Execution of the MAO between the Department and the Permittee shall satisfy the requirements for submission by the Permittee of a formal plan and time schedule for achieving permit compliance as described in the NPV #WQMW-WR-95-181.

12. If any event occurs that is beyond Permittee's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this MAO, Permittee shall immediately notify the Department verbally of the cause of delay or deviation and its anticipated duration, the measures that have been or will be taken to prevent or minimize the delay or deviation, and the timetable by which Permittee proposes to carry out such measures. Permittee shall confirm in writing this information within five (5) working days of the onset of the event. It is Permittee's responsibility in the written notification to demonstrate to the Department's satisfaction that the delay or deviation has been or will be caused by circumstances beyond the control and despite due diligence of Permittee. If Permittee so demonstrates, the Department shall extend times of performance of related activities under this MAO as appropriate. Circumstances or events beyond Permittee's control include, but are not limited to acts of nature, unforeseen strikes, work stoppages, fires, explosion, riot, sabotage, or war. Increased cost of performance or consultant's failure to provide timely reports may not be considered circumstances beyond Permittee's control.

13. Regarding the violations set forth in Paragraphs 3, 4 and 5 above, which are expressly settled herein without penalty, Permittee and the Department hereby waive any and all of their rights to any and all notices, hearing, judicial review, and to service of a copy of

the final MAO herein. The Department reserves the right to enforce this MAO through appropriate administrative and judicial proceedings.

14. The terms of this MAO may be amended by the mutual agreement of the Department and Permittee.

15. The Department may amend the compliance schedule and conditions in this MAO upon finding that such modification is necessary because of changed circumstances or to protect public health and the environment. The Department shall provide Permittee a minimum of thirty (30) days written notice prior to issuing an Amended Order modifying any compliance schedules or conditions. If Permittee contests the Amended Order, the applicable procedures for conduct of contested cases in such matters shall apply.

16. This MAO shall be binding on the parties and their respective successors, agents, and assigns. The undersigned representative of each party certifies that he or she is fully authorized to execute and bind such party to this MAO. No change in ownership or corporate or partnership status relating to the facility shall in any way alter Permittee's obligations under this MAO, unless otherwise approved in writing by DEQ.

17. Unless otherwise directed in writing by the Department, all reports, notices and other communications required under or relating to this MAO should be directed to Mark Hamlin, DEQ Salem Regional Office, 750 Front Street NE, Suite 120, Salem, Oregon 97310; phone number (503) 378-8240, extension 239. Unless otherwise directed in writing by the Permittee, the contact person for Permittee shall be Director of Public Works, City of Sweet Home, 1730 North Ninth Avenue, Sweet Home, Oregon 97386.

18. Permittee acknowledges that it has actual notice of the contents and requirements of the MAO and that failure to fulfill any of the requirements hereof would constitute a violation of this MAO and subject Permittee to payment of civil penalties pursuant to Paragraph 10C. above.

19. Any stipulated civil penalty imposed pursuant to Paragraph 10C. shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the "Oregon State Treasurer" and sent to: Business Office, Department of Environmental Quality, 811 S.W. Sixth Avenue, Portland, Oregon 97204. Within 21 days of receipt of a "Demand for Payment of Stipulated Civil Penalty" Notice from the Department, Permittee may request a hearing to contest the Demand Notice. At any such hearing, the issue shall be limited to Permittee's compliance or non-compliance with this MAO. The amount of each stipulated civil penalty for each violation and/or day of violation is established in advance by this MAO and shall not be a contestable issue.

20. Providing Permittee has paid in full all stipulated civil penalties pursuant to Paragraph 19 above, this MAO shall terminate 60 days after Permittee demonstrates full compliance with the requirements of the schedule set forth in Paragraph 10.A. above.

CITY OF SWEET HOME

1/9/01

Date

Craig D. Fentiman

Mayor

DEPARTMENT OF ENVIRONMENTAL QUALITY

1/19/01

Date

Kerri Nelson, Western Region Division Administrator

FINAL ORDER

IT IS SO ORDERED:

ENVIRONMENTAL QUALITY COMMISSION

1/19/01

Date

Kerri Nelson, Western Region Division Administrator
Department of Environmental Quality
Pursuant to OAR 340-11-136(1)

Oregon
Kate Brown, Governor

Department of Environmental Quality
Western Region Salem Office
4026 Fairview Industrial Dr SE
Salem, OR 97302
(503) 378-8240
FAX (503) 373-7944
TTY 711

September 9, 2015

RECEIVED
SEP 11 2015
BY:

Mr. Michael Adams
Public Works Director
City of Sweet Home
1140 12th Avenue
Sweet Home, OR 97386

RE: Compliance with Mutual Agreement and Order No. WQ/M-WR-98-221
WQ - Linn County / File No. 86840

Dear Mr. Adams:

As you are aware, Mark Hamlin with DEQ notified the City in May of this year that Mutual Agreement and Order No. WQ/M-WR-98-221 to reduce sewage overflows was no longer required. In response to the City's concerns expressed during our August 17 meeting that overflows may still occur, DEQ agreed to review the City's record of overflows in greater detail and consult with the DEQ Office of Compliance and Enforcement on specific terms in the MAO.

We have completed our review and internal discussions and determined that the City of Sweet Home's obligations under this MAO have been fulfilled. MAO Addendum No.3, dated December 2, 2005, approved the City's request to extend the final compliance date to January 1, 2010. The final compliance task was to "comply with all applicable water quality standards and treat all flows up to the one-in-five-year, 24-hour storm event". As required by the MAO, the City completed projects to reduce the inflow and infiltration in the sewage collection system. Prior to completing these projects, the City experienced several raw sewage overflows each year. These projects have been a huge success and from 2010 to 2012 there were only four documented overflows; January 2011, two in January 2012, and one in March 2012. The City has not experienced any overflows since March 2012.

DEQ understands that the City is still working to increase the capacity of the treatment facility to fully assure that sewage overflows do not occur in the future. Accordingly, the City would prefer the MAO remain intact until the proposed treatment system improvements are completed. While DEQ agrees with the project to increase treatment capacity, DEQ does not enter into MAOs for possible future violations. Since the City has not experienced any sewage overflows in over three years during which significant storm events have occurred, DEQ cannot determine the likelihood of future overflows in violation of state rules. Should the City experience a raw sewage overflow in the future, DEQ will examine the evidence for the cause of the overflow and may use enforcement discretion if the overflow was beyond the City's reasonable control. Additional known permit violations may be a basis to negotiate a new MAO.

Finally, please note that the interim limits for $CBOD_5$ and TSS specified in the terminated MAO are also no longer applicable. The City must comply with all limits and conditions in the issued NPDES permit No. 101657.

Thank you for your continued efforts in improving our environment. Should you have any questions, please contact me at 503-378-5039.

Sincerely,

Robert Dicksa
Senior Water Quality Specialist
DEQ WR-Salem Office

Cc: Water Quality File-Salem
ecc: Ranei Nomura, WR-Salem, DEQ
Deborah Nesbit, OCE DEQ Headquarters

Appendix C: Inflow and Infiltration Update Report

I/I Update Report

Prepared for
City of Sweet Home, Oregon
September 24, 2013

REGESTERED PROFESSIONAL
ENGINEER
82,099
OREGON
NOV 12, 2008
ROBERT KYUNG WON LEE
EXPIRES: 12/31/2014

FINAL

Brown AND Caldwell
6500 SW Macadam Avenue, Suite 200
Portland, OR 97239

Table of Contents

List of Figures

List of Tables

List of Abbreviations

AV	area-velocity
BC	Brown and Caldwell
CAPE	Capacity Assurance Planning Environment modeling platform
CCTV	closed-circuit television
cfs	cubic feet per second
CIPP	cured-in-place pipe
City	City of Sweet Home
CMOM	Capacity, Management, Operation, and Maintenance
DEQ	Oregon Department of Environmental Quality
DWF	dry weather flow
FOG	fats, oil, and grease
gpad	gallons per acre per day
gpm	gallons per minute
I/I	infiltration/inflow
LPIII	Log Pearson Type III
MAO	Mutual Agreement and Order
mgd	million gallons per day
NASSCO	National Association of Sewer Service Companies
NOAA	National Oceanic and Atmospheric Administration
LF	linear feet
NPDES	National Pollutant Discharge Elimination System
O&M	operations and maintenance
QA/QC	quality assurance/quality control
R&R	rehabilitation and replacement
RDII	rainfall-derived infiltration/inflow
ROW	right-of-way
SFE	SFE Global
SSO	sanitary sewer overflow
SWII	Stanford Watershed Infiltration/Inflow model
USEPA	U.S. Environmental Protection Agency
WWF	wet weather flow
WWTP	wastewater treatment plant

Executive Summary

The City of Sweet Home (City) retained Brown and Caldwell (BC) in 2002 to analyze sewer system infiltration/inflow (I/I) rates, evaluate system capacity deficiencies, make preliminary recommendations on how to comply with the Oregon Department of Environmental Quality (DEQ) requirement to pass without overflow the 5-year, peak-hour flow in the winter and the 10-year, peak-hour flow in the summer. Through the course of this work with the City, four rehabilitation and replacement (R&R) projects have been completed on portions of the collection system in an effort to reduce I/I. From recent collection system flow monitoring and modeling studies and a hydraulic capacity evaluation, BC concludes the R&R projects have been effective in reducing peak I/I flow rates and the City is moving toward compliance with DEQ regulations.

This report presents the results of the modeling efforts and the demonstrated effectiveness of the four R&R projects.

Findings

The City has invested over $15 million in planning and construction of the first four phases of R&R work in the collection system. The construction costs for each phase are listed in Table ES-1.

Table ES-1. Summary of R&R Costs by Phase

Construction phase	Capital cost, millions of dollars
Phase 1	1.3
Phase 2	1.7
Phase 3	3.1
Phase 4	6.0

Approximately 35 percent of the main line sewers and 30 percent of the laterals in Sweet Home have been rehabilitated using a variety of techniques. Service laterals have been rehabilitated to varying degrees. Due to access constraints, funding requirements, and budget limitations, not all service laterals have been fully rehabilitated all the way to the building. This variable level of rehabilitation should be considered when evaluating the I/I reduction effectiveness results and when planning future R&R work within the City's collection system.

Figure ES-1 shows the extent of rehabilitation for the first four phases of R&R work.

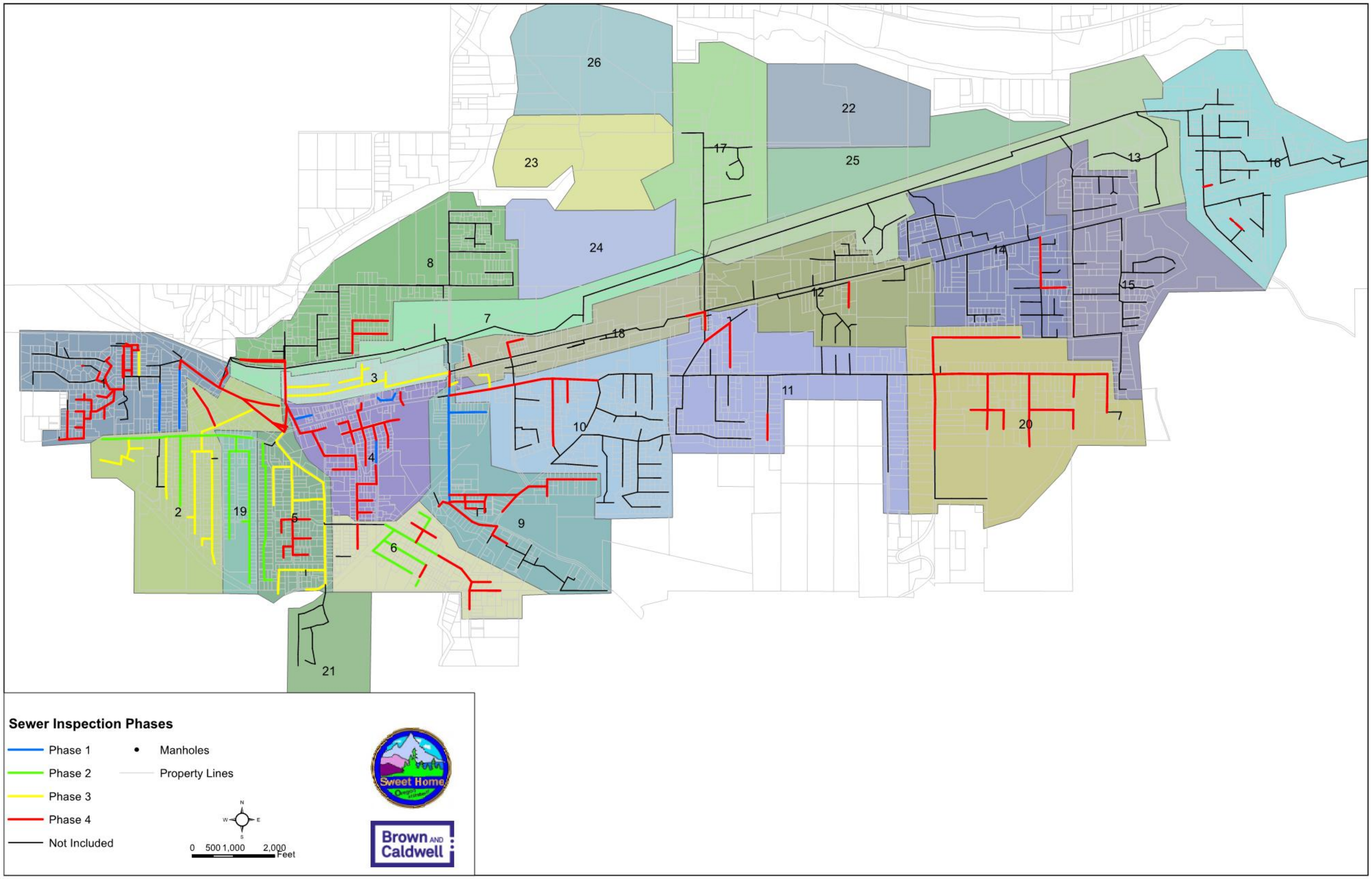


Figure ES-1. Phases 1, 2, 3, and 4 R&R work

Hydrologic Modeling Efforts

As part of the *Sanitary Sewer Master Plan* (BC, 2002), a hydrologic model was developed to simulate the maximum-hour, 1-in-5 year flows. Modeling was conducted under two scenarios: 1) using the existing population; and 2) using future population and also assuming future expansion of the City's wastewater service area.

Flow monitoring data, collected by the City from December 2000 to February 2002 at eight locations, was used to calibrate the model. As a result of the modeling effort, the peak-hour flow with a 5-year recurrence under existing population projections was modeled to be 22.0 mgd, while the peak-hour 5-year flow under future population projections was modeled to be 25.1 mgd.

After completion of the Phases 1 and 2 rehabilitation projects, a more comprehensive flow monitoring effort was conducted. The model was recalibrated using these flow data. The model projected that, under current population and service area conditions, the maximum-hour flow with a 5-year recurrence was 15.3 mgd with a system peaking factor of 15, or a 6.7 mgd reduction in peak-hour flow from the modeling effort conducted in 2002. BC postulates that this dramatic decrease in peak flow was the result of the Phases 1 and 2 projects as well as the more refined flow data leading to a more precise calibration of the model.

The metering/modeling results were also used to determine the most cost-effective methodology for rehabilitation and to focus the capital investments on the leakiest basins. Basins that underwent rehabilitation of the mains and laterals appeared to have the greatest reduction in I/I by a significant margin, as listed in Table ES-2.

Table ES-2. Post-Phase 1 and Phase 2 Rehabilitation Effectiveness Summary

I/I reduction method	Effectiveness at reducing I/I, percent
Sewer mains and manholes	11 to 16
Laterals only	7 to 11
Sewer mains, manholes, and laterals to building	60 to 88

Based on the understanding of the need to address the sewer system holistically in each basin, Phase 3 was designed and constructed with the goal of completing rehabilitation in areas that were only partially rehabilitated previously as well as adding full basins to the scope. After completion of the Phase 3 rehabilitation project, additional flow monitoring was conducted during the winter of 2008/2009 at eight locations to gauge the effectiveness of the Phase 3 work. The projected peak-hour flow with a 5-year recurrence, under existing population and service area conditions, was 13.6 mgd, or a 1.7 mgd reduction in peak flow from the 2006 modeling effort.

Phase 4 was aimed at continuing the holistic rehabilitation efforts. After completion of the Phase 4 rehabilitation project, additional flow monitoring was conducted during the winter of 2012-2013 at 15 locations to gauge the effectiveness of the Phase 4 work and project the future peak-hour flows to the Sweet Home Wastewater Treatment Plant (WWTP). The projected peak-hour flow with a 5-year recurrence, under existing population and service area conditions, is 11.5 mgd, or a 2.1 mgd reduction in peak flow from the 2009 modeling effort.

This reduction was less than expected during the predesign efforts of Phase 4; however there are two main contributing factors. First, the funding for Phase 4 had unacceptable constraints for any work conducted on private property, so many laterals that were slated for full rehabilitation were addressed either at the connection only or to the edge of the public right-of-way. Secondly, some rerouting and upsizing was added to Phase 4 to reduce the occurrences of overflows upstream in the system, particularly at the upstream end of the Ames Creek siphon. While this upsizing reduces overflows in the collection system, it has the consequence of allowing additional I/I that was previously restricted from entering the system because of hydraulically restricted pipes.

In total for all four phases, 10.5 mgd of peak-hour I/I has been removed from the system under existing conditions and nearly 11.8 mgd under future conditions. Table ES-3 summarizes modeling results for the phases.

Table ES-3. Modeling Results

Model run	Peak-hour flow, existing conditions, mgd	Peaking factor	Peak-hour flow, future conditions, mgd
Pre-Phase 1 and 2	22.0	22	25.11[a]
Post-Phase 1 and 2	15.3	15	17.92[b]
Post-Phase 3	13.6	14	15.42[b]
Post-Phase 4	11.5	12	13.32[b]
Total Flow Removed	10.5	-	11.8

[a]*Based on future population (2027) of 10,525 with no expansion of the City's wastewater service area (WWFP, 2002).*

[b]*Based on future population (2025) of 15,633 with expansion of the City's wastewater service area (WWFP, 2002).*

Figure ES-2 shows these predicted peak-hour flows after each modeling effort in graphical format.

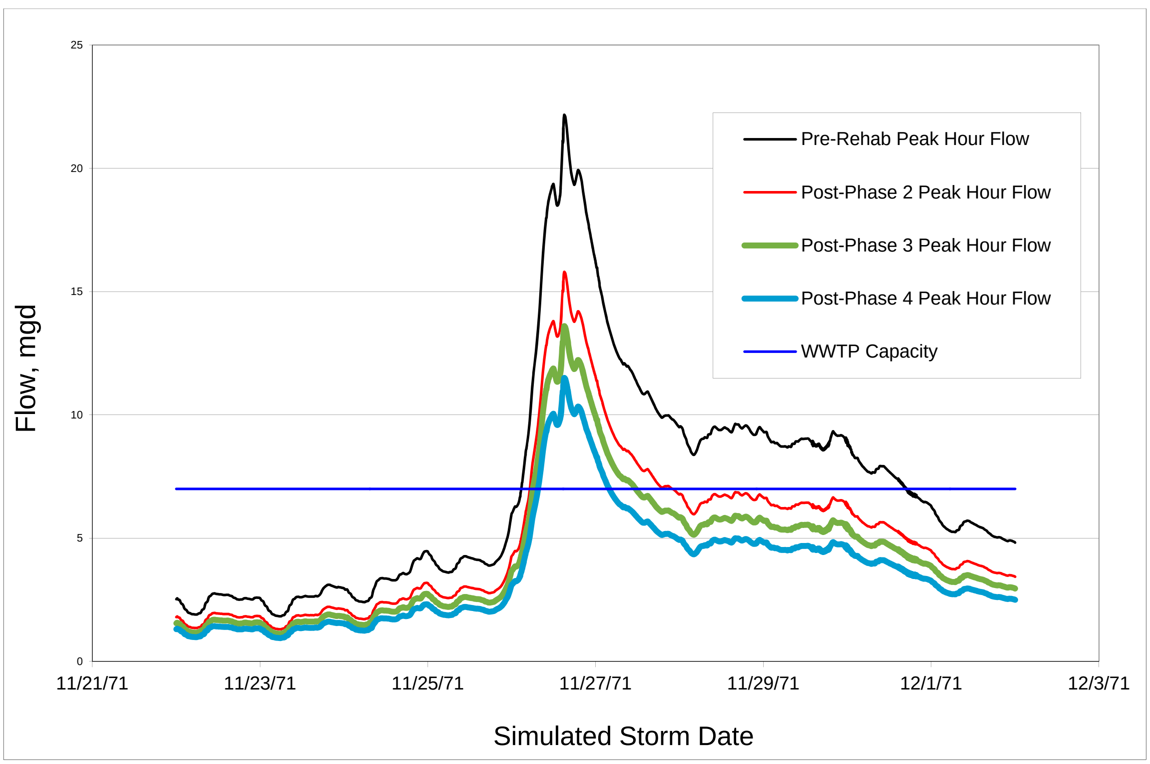


Figure ES-2 Predicted 1-in-5 peak-hour flow

Brown AND Caldwell

Future R&R work in the collection system should continue for the City, either to maintain the level of RDII entering the system or to further target RDII reductions while making structural improvements to the unaddressed sewers that are aging and deteriorating. However, the highest priority basins identified throughout the course of the I/I Abatement Program have been largely addressed and there is a diminishing rate of return on the dollars invested in the collection system. Table ES-4 lists the estimated rehabilitation costs for future R&R work, with the expected reduction in peak RDII.

Table ES-4. Future R&R Work Cost Effectiveness

Sanitary Basin(s)[a]	Type of R&R	Cost of remaining R&R work, dollars	Peak RDII removed[b], mgd	Cost-effectiveness, dollars per mgd RDII removed	Rank
1	Full rehabilitation, complete uppers	1,620,000	0.18	9,000,000	12
2, 19	Complete uppers	310,000	0.17	1,800,000	1
3	R&R work complete	0	0	0	NA
4	Complete uppers	820,000	0.14	5,700,000	7
5, 6, 21	Complete uppers	970,000	0.39	2,500,000	2
7,13,14,17	Full rehabilitation	7,350,000	1.55	4,700,000	6
8	Full rehabilitation, complete uppers	2,720,000	0.28	9,900,000	13
9	Full rehabilitation, complete uppers	910,000	0.29	3,100,000	4
10	Full rehabilitation, complete uppers	2,990,000	0.42	7,100,000	11
11,12	Full rehabilitation	3,770,000	0.53	7,100,000	10
15	Full rehabilitation	2,130,000	0.31	6,800,000	8
16	Full rehabilitation	2,520,000	0.58	4,400,000	5
18	Full rehabilitation	1,130,000	0.37	3,100,000	3
20	Complete uppers	630,000	0.09	7,000,000	9
	Total	27,900,000	5.30		

[a]*Basins grouped together due to flow monitoring locations and model calibration* methodology.
[b]*Assumes 65 percent reduction in RDII for full rehabilitation, 30 percent reduction for completing uppers.*

An estimated $28 million in construction costs would be required to remove an additional 5.3 mgd. Since $12 million was spent on the first four phases with over 10 mgd removed, the diminishing cost-effectiveness is apparent. However, future R&R work should focus on completing the upper laterals, particularly on Phase 4 sewers, with full rehabilitation efforts directed in Sanitary Basins 18, 9, and 16, in that order of priority.

Hydraulic Modeling Efforts

A hydraulic model was developed to determine the collection system's response to peak flows under the 5-year wet-weather condition event. Flows input into the hydraulic model were the current population dry weather flow as well as projected rainfall-derived I/I (RDII) peak-hour flows under the 1-in-5 year 24-hour event. Only the major trunk lines were modeled hydraulically and are shown in Figure ES-3. The scenario assumes no hydraulic restrictions or flow limitations at the Sweet Home WWTP, meaning the WWTP will be expanded to convey the peak 5-year flows. The scenario also assumes the pipes are maintained properly and are capable of reaching their hydraulic capacity.

Figure ES-3. Hydraulic modeling network

The hydraulic modeling effort reveals a number of locations where the collection system either surcharges or overflows. Figure ES-4 summarizes the hydraulic modeling results from the 1-in-5 year event under existing conditions. Red manholes indicate locations of projected overflows, yellow manholes indicate locations of surcharging between 0 to 3 feet below grade, and green manholes indicate either no surcharging or surcharging between 3 and 10 feet below grade. A number of locations where overflows were identified in the Post-Phase 3 modeling effort, particularly along the main trunk that parallels the railroad, are now projected not to overflow based on the rehabilitation work conducted as part of Phase 4. Under existing conditions, a single manhole at Long and 18th Streets is predicted to overflow in the 1-in-5 year event. The manhole and associated pipe segments were rehabilitated in Phase 4, but this location was not identified as a potential overflow location. It is possible that the slight reduction in inside diameter from the Phase 4 reconstruction work as well as refined flow data and model calibration since the 2009 modeling effort is contributing to the predicted overflows.

Figure ES-4. Hydraulic modeling results, projected surcharge and overflow locations under existing conditions

Figure ES-5 shows locations where the model predicts potential severe surcharging or overflows under future conditions. Under future conditions, three additional overflow locations on the east-west 24-inch trunk paralleling the railroad tracks are anticipated. However, raising or sealing these manholes will prevent overflows without creating additional overflow points anywhere else in the City.

Figure ES-5. Hydraulic modeling results, projected surcharge and overflow locations under future conditions

Conclusions

The following summarizes the conclusions BC has made based on modeling and hydraulic capacity evaluation.

- Post-rehabilitation and reconstruction flow monitoring and hydrologic modeling demonstrate that basin-wide work can remove approximately 65 percent of the projected 1-in-5 year event peak-hour RDII flow in that basin.
- Focusing efforts on rehabilitating sewer mains, manholes, and laterals to the private building has been found to be the most effective at removing peak-hour RDII. Focusing only on specific components such as mains or laterals offers some reduction but at a much lower cost-effectiveness.
- To date, over 50 percent of the peak-hour RDII has been removed from the system over four phases of R&R work.
- Approximately an additional 4.5 mgd of RDII will need to be removed or accommodated at the WWTP to pass the 1-in-5 peak-hour flow under existing conditions, and approximately 6.3 mgd will need to be removed to handle future conditions. These are conservative estimates based on the modeling work.
- Under existing conditions, a single manhole at Long and 18th streets is predicted to overflow in the 1-in-5 year event. The manhole and associated pipe segments were rehabilitated in Phase 4 but this manhole was not identified as a potential overflow location. It is possible that the slight reduction in inside diameter from the Phase 4 reconstruction work and refined flow data and model calibration since the 2009 modeling effort are contributing to the predicted overflows.
- The benefits of R&R work in select basins have not been realized fully due to partial lateral rehabilitation caused by funding agency constraints related to work on private property without a permanent easement and/or owner unwillingness to allow for the work to be completed. Completing the rehabilitation work on the uppers in these partially completed basins (see Table 8-2) is the most cost-effective way to remove additional RDII.
- Full replacement of sanitary basins 18 and 9 have the most cost-effective R&R remaining in the City, with an approximate cost of $2.04 million (2010 R&R costs) to remove approximately 0.66 mgd of

peak-hour RDII. Sanitary Basin 8, conversely, has an approximately $2.7 million R&R cost to remove an estimated 0.28 mgd of peak-hour RDII.

- Upsizing and rerouting of flows from Sanitary Basins 5 and 6 toward Sanitary Basin 2 has significantly reduced the potential for overflows at the upstream of the siphon under Ames Creek, but may have resulted in the negative effect of allowing previously restricted I/I to now enter the system.
- A number of locations where overflows were identified as overflow points in the Post-Phase 3 modeling effort, particularly along the 18- to 24-inch main trunk that parallels the railroad, are now no longer projected to overflow based on the rehabilitation work conducted as part of Phase 4.
- Whereas the *Sanitary Sewer Master Plan* identified approximately $1.4 million in upsizing pipes to pass the 1-in-5 peak-hour flows (2012 dollars), the R&R work under the last four phases has essentially eliminated the need for upsizing of pipes. This assumes that the rate of RDII does not increase over time and that the City finds surcharging up to the manhole rim but not overflowing acceptable during the 1-in-5 year event. The City should continue to address RDII in the system on an annual basis. Under existing conditions, there is one manhole in Sweet Home that is predicted to overflow during the 1-in-5 year peak-hour flow event.
- Under future conditions, there are three additional manholes that are predicted to overflow during the 1-in-5 peak-hour flow. Several additional manholes on or immediately adjacent to the 24-inch main trunk line just upstream of the WWTP experience increased surcharging to within 3 feet of the manhole rim.

Recommendations

BC recommends the City take the following steps to continue to manage I/I in the system with the goal of regulatory compliance:

- Closely monitor the single manhole at the downstream end of Sanitary Basin 10 on Long Street that is predicted to overflow during the 1-in-5 year peak-hour flow. Due to margin-of-error and compounding conservative assumptions within any modeling effort, it is possible the predicted overflow may be overly conservative. Therefore as a precaution, the City should clean and monitor this section of pipe annually and also prior to anticipated large wet-weather events. In addition, there is a significant portion of Sanitary Basin 10 that has not been addressed by the first four phases of the program. R&R work in Sanitary Basin 10 will likely greatly reduce the overflow potential, both in existing as well as future conditions. Additional flow monitoring at monitoring location 9.1 to validate the modeling predicted peak flows is also recommended.
- Evaluate sealing or raising the three manholes just east of 9th Avenue on the east-west 24-inch trunk paralleling the railroad tracks. These manholes are predicted to overflow under future conditions but sealing or raising these manholes will prevent overflows while also not creating any adverse affect elsewhere in the City's collection system.
- Prepare an update to the City's Wastewater Facility Plan to determine the feasibility and cost of an upgrade to the Sweet Home WWTP to accommodate additional flows and determine the break-even point between WWTP upgrades and RDII reduction through future R&R work. As part of this update, re-evaluate the future growth projections and timing of expansion of the City's wastewater service areas.
- Prioritize completion of the rehabilitation work on upper laterals to complete the holistic basin approach, per Table 8-2. Further R&R work in the collection system aimed at reducing peak-hour RDII has diminishing returns. **However, at a minimum the City must continue with additional R&R work to maintain the current level of RDII in the system.** Sanitary Basins 18 and 9 are the next

highest priority basins with the largest predicted RDII removal rates. Look for opportunities to remove I/I while also addressing the pipes with the worst structural ratings.

- Explore implementing a lateral rehabilitation program that can address the private laterals without the constraints of acquiring permanent easements.
- Update sewer condition maps that document the structural and operational condition of sewers. The last comprehensive update of sewer condition was completed in 2006.
- Evaluate the cost and feasibility for addressing Grade 5 sewers (as defined in Section 6 of the main report). Many Grade 5 sewers are likely rated so severely due to isolated point defects rather than full pipe issues. However, failure of point defects are as problematic as full length failures and the City should plan for the rehabilitation of these Grade 5 sewers.
- Begin preparing for and implementing a formal Capacity, Management, Operations, and Maintenance (CMOM) Program, in accordance with U.S. Environmental Protection Agency guidelines. The Oregon Department of Environmental Quality has guidance documents that indicate cities with compliant CMOM plans in place will receive greater leniency in cases of non-compliance (e.g., overflows during events less than the 1-in-5 year storm, see Appendix B).
- Install flow meters and increase the monitoring resolution in Sanitary Basins 7, 13, 14, and 17 to further delineate flows and determine if full basin rehabilitation would be effective. The City's post-Phase 4 flow monitoring was extremely successful, and the City can utilize their flow monitoring equipment and experience to identify and prioritize areas of additional RDII reduction.

Section 1

Introduction

1.1 Background

The City of Sweet Home (City) is located along the west slope of the Cascade Mountains at the edge of the Willamette Valley. The City limits encompass an area of approximately 6.5 square miles and the urban growth boundary is coincident with the city limits. The current population is 9,025, with the population expecting to increase to 9,800 in 2020 and 10,550 in 2027.

The South Santiam River runs east to west along the northern edge of the city and functions as the base of the watershed in which the city lies. Groundwater in the area is generally shallow and ranges from 8 to 25 feet below ground surface. Soils in the area are comprised of fluvial gravels near the Santiam River with silty clay and loam in the upland areas.

The City wastewater collection system is comprised of approximately 275,000 linear feet (LF) of sanitary sewers. Construction of the collection system began as early as 1910. The sewer pipe ranges from 6 to 24 inches in diameter with over 80 percent of the pipe sized at 8 inches. The majority of pipes are constructed of non-reinforced concrete pipe in 3-1/2 foot sections. Other pipe materials include reinforced concrete, cast iron, and poly-vinyl chloride. The collection system transports wastewater by gravity flow to the Sweet Home Wastewater Treatment Plant (WWTP) adjacent to the South Santiam River. The WWTP treats an average dry weather flow of 1 million gallons per day (mgd) with a treatment capacity of up to 7 mgd. Figure 1-1 shows an overview of City's collection system and location of the WWTP.

The City's sanitary sewer collection system experiences high levels of infiltration/inflow (I/I) during wet weather that can lead to overflows at the WWTP and within the collection system. For the past several years, the City has been under a Mutual Agreement and Order (MAO) from the Oregon Department of Environmental Quality (DEQ) to eliminate sanitary sewer overflows in accordance with Oregon Administrative Rule 341-041-0120. More specifically, the MAO requires the elimination of any overflows caused by less than the 1-in-5 year recurrence, 24-hour-duration storm during the winter (November 1 to May 21) and the 1-in-10 year, 24-hour storm during the summer (May 22 to October 31).

To meet DEQ requirements, the City had the choice in 2002 of either reducing I/I within the collection system at an estimated cost of $30 million or increasing capacity of the WWTP at an estimated cost of $17 million. Even though I/I reduction was more costly, the City recognized that its collection system was aging and not addressing the deterioration would likely lead to future problems and potentially higher I/I. The City decided to conduct an aggressive I/I abatement program.

This report summarizes the results of the multi-phased program to reduce I/I within the collection system between 2003 and 2012.

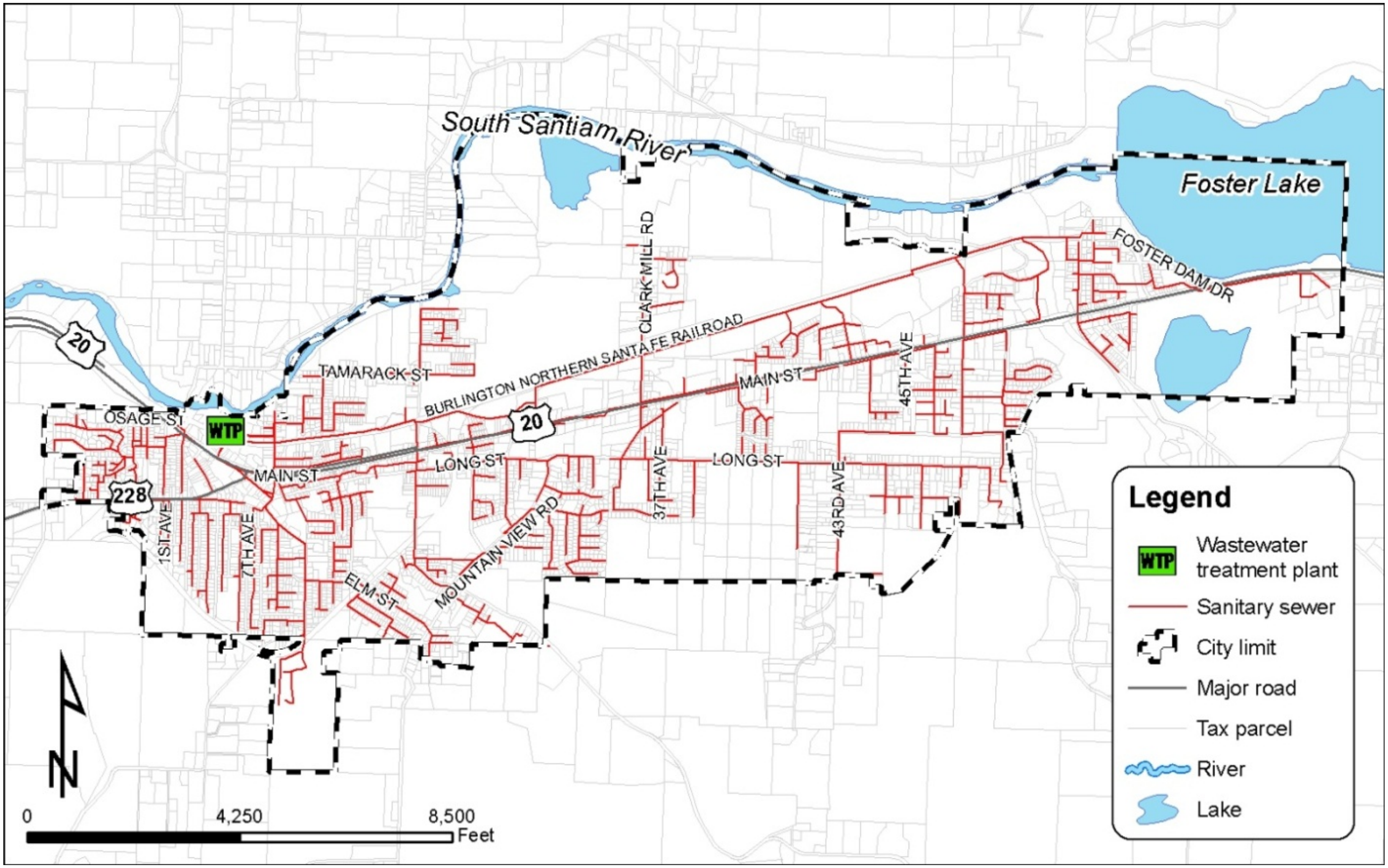


Figure 1-1. Overview of the City's collection system

1.2 Summary of Rehabilitation Work

The I/I abatement program has consisted of multiple phases. In 2003 and 2004, two separate I/I reduction demonstration projects (Phases 1 and 2), as well as pre- and post-rehabilitation flow monitoring and modeling were conducted in some of the leakiest basins to help determine the most cost-effective approach to I/I removal. Holistic basin-wide rehabilitation addressing manholes, sewer main, and laterals up to the private building was determined to be the most cost-effective method of removing I/I.

In 2007, Phase 3 addressed basins that did not yet complete this holistic approach and added other basins. Post-rehabilitation flow monitoring and modeling were conducted to measure results and target areas for future rehabilitation. In 2012, Phase 4 was completed and post-rehabilitation flow monitoring followed in the winter of 2012/2013 and the City's hydrologic and hydraulic models were recalibrated.

Work was focused either in entire City sanitary basins or smaller subbasins. Sweet Home is divided into 27 sanitary basins, 19 of which have residents within their boundaries connected to the public sewer system. Figure 1-2 shows a map of the City's sanitary basins.

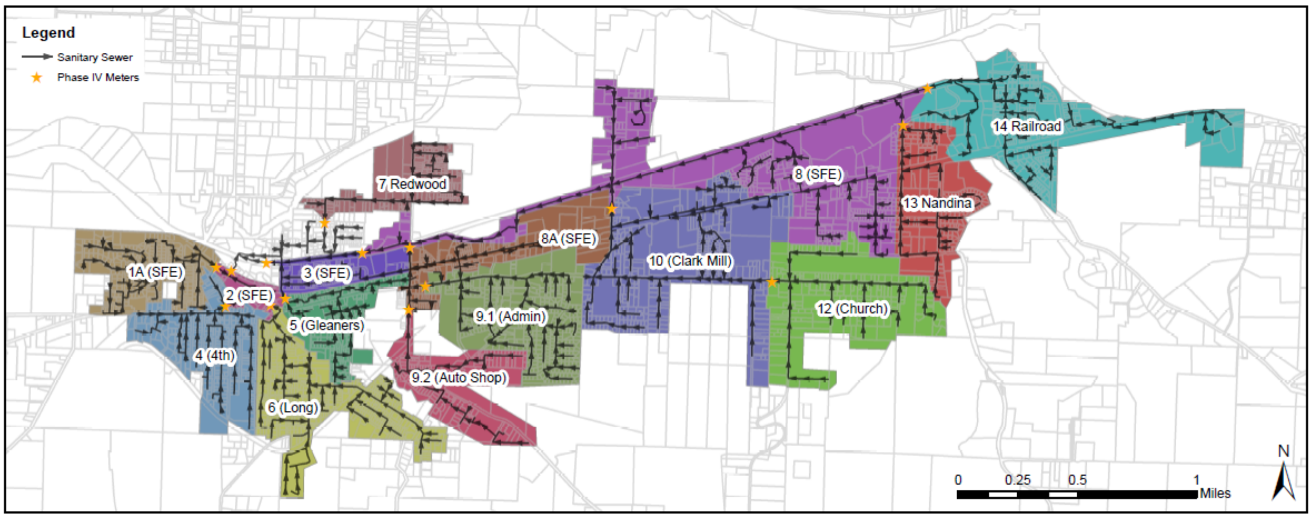


Figure 1-2. City's monitoring basins

1.2.1 Phases 1 and 2

Phase 1 work was performed in 2003 and focused predominately on the southern part of the city, in Sanitary Basins 1, 4, 5, 6, and 9. Phase 2 was performed in 2004 and focused on the southwestern portion of the city, with a focus on Sanitary Basins 1, 2, 5, and 19. Approximately $3 million was spent on these two phases, which included the rehabilitation or reconstruction of 18,500 LF of sewer main and 300 laterals. During Phases 1 and 2, rehabilitation technologies were separated into three categories to determine the most effect rehabilitation plan. In some basins, only sewer mains and manholes were addressed with existing laterals being reconnected to the new sewer main. In other basins, only service laterals were rehabilitated to the edge of the public right-of-way (ROW) while in other areas, service laterals on private property were rehabilitated with the goal of attempting to rehabilitate the lateral as close to the private building as possible. Lastly, some areas had a more holistic approach, with rehabilitation efforts focusing on sewer mains, manholes, and laterals up to the private building. Inactive lateral connections were plugged and cleanouts were installed on all active laterals.

The extent of the work in each Sanitary Basin is shown in Figure 1-4.

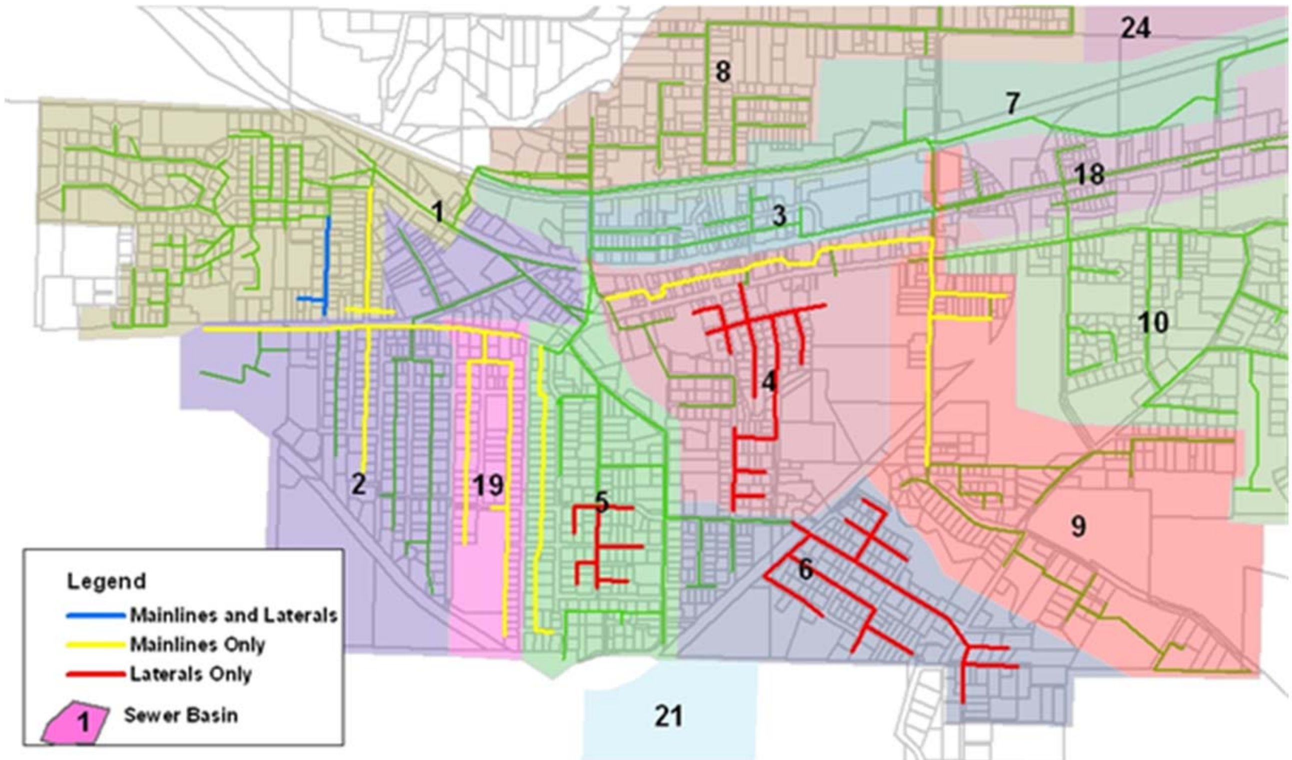


Figure 1-3. Phases 1 and 2 project extents

1.2.2 Post-Phases 1 and 2 Flow Monitoring and Modeling

After completion of Phases 1 and 2 construction, flow monitoring and modeling were conducted to quantify the benefits of the rehabilitation program and allow accurate I/I reduction estimates to be made. These estimates were used to determine the most cost-effective methodology for rehabilitation and to focus the capital investments on the leakiest basins. Basins that underwent rehabilitation of both mains and laterals have the greatest reduction in I/I by a significant margin, as listed in Table 1-1.

Table 1-1. Post-Phase 1 and Phase 2 Rehabilitation Effectiveness Summary

I/I reduction method	Effectiveness at reducing I/I, percent
Sewer mains and manholes	11 to 16
Laterals only	7 to 11
Sewer mains, manholes, and laterals to building	60 to 88

A more detailed discussion of the rehabilitation effectiveness analysis is described in Section 2.

1.2.3 Phase 3

Phase 3 work was performed in 2007 and focused on Sanitary Basins 1, 2, 3, and 5. Approximately $3 million was spent on Phase 3 work, which included the rehabilitation or reconstruction of 17,000 LF of sewer main and 415 laterals. After the Phases 1 and 2 post-rehabilitation I/I removal effectiveness analysis, Phase 3 focused on completing only those basins in Phases 1 and 2 that were partially completed and holistic rehabilitation in previously unaddressed high-priority basins.

The extent of the Phase 3 project is shown in Figure 1-4.

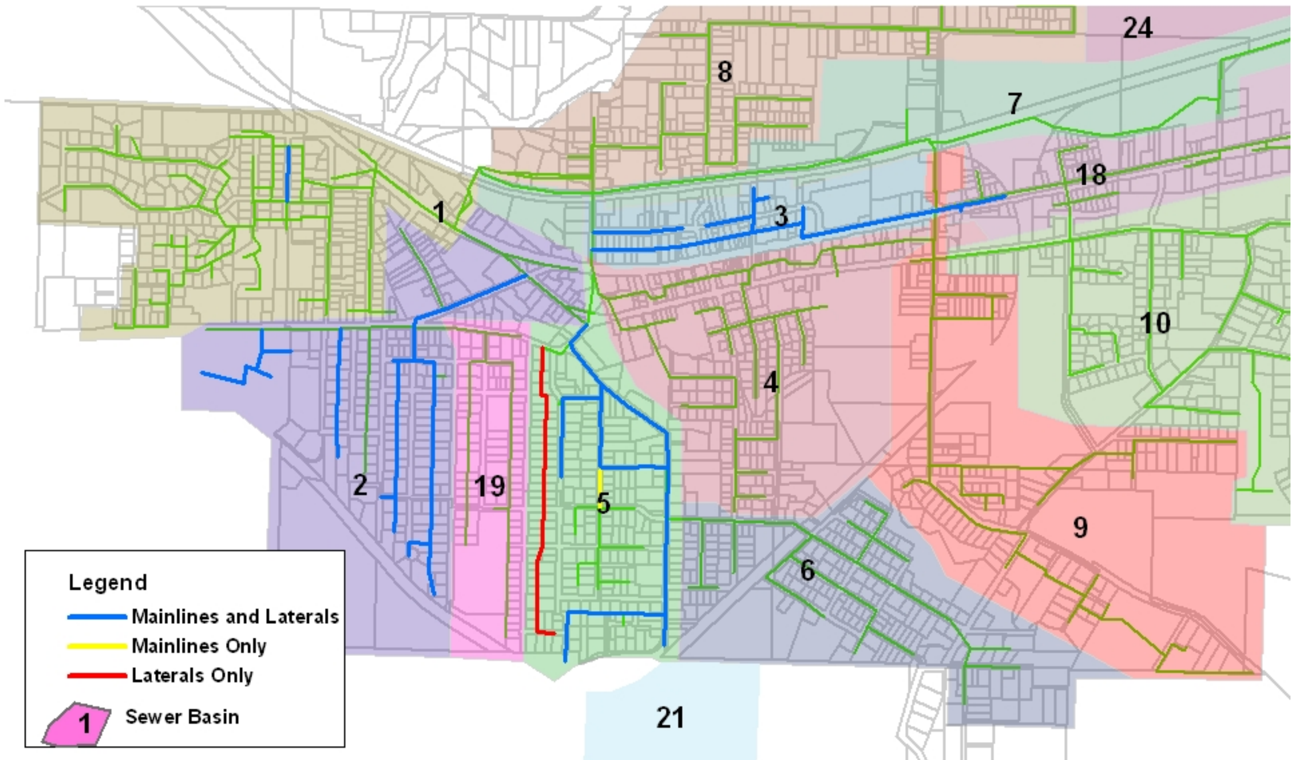


Figure 1-4. Phase 3 project extents

Combined, the first three phases have addressed 36,000 LF of sewer main, or approximately 15 percent of the sewers in the city. Approximately 700 laterals have been rehabilitated or replaced (R&R), or 20 percent of the laterals in the city. Figure 1-5 shows the extents of all R&R work from Phases 1, 2, and 3.

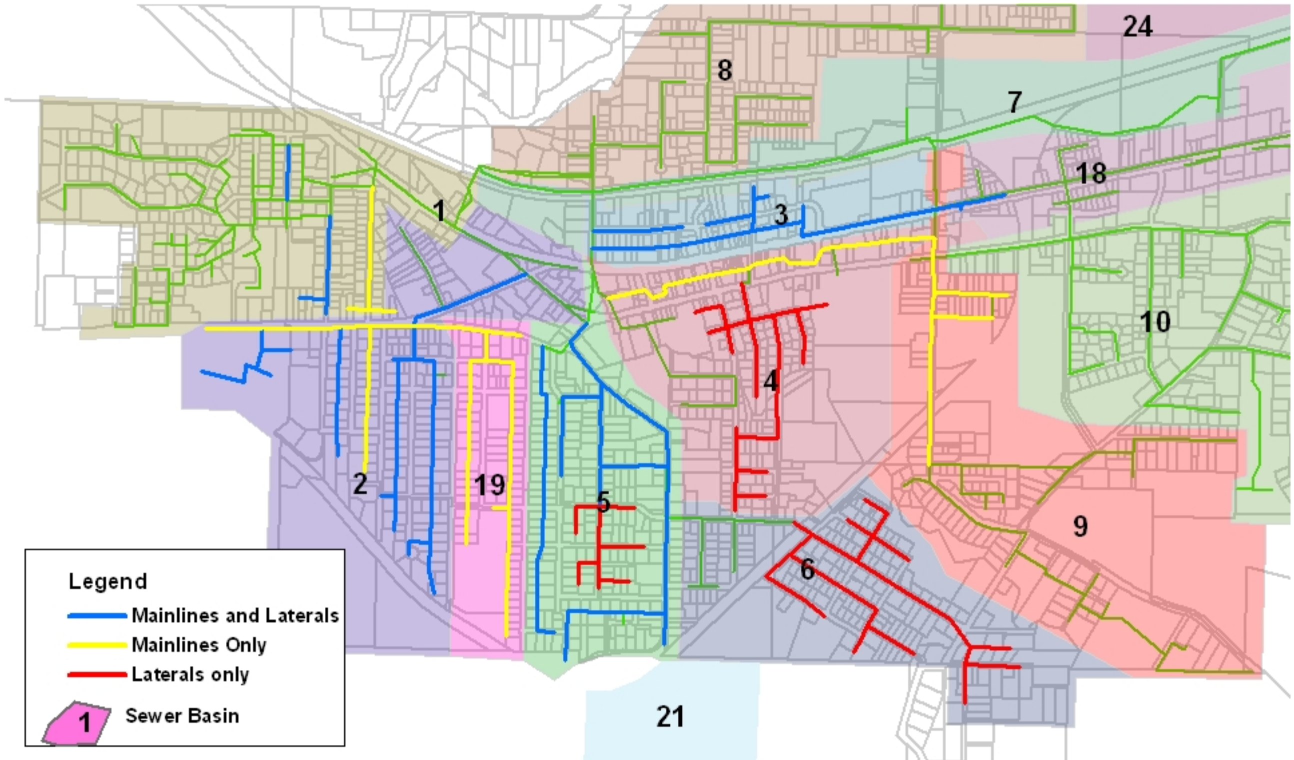


Figure 1-5. Phases 1, 2, and 3 R&R work

1.2.4 Phase 4

Phase 4, the largest of the four I/I abatement projects, was completed in 2012. The $6 million project covered 11 basins and was designed to rehabilitate or reconstruct 51,500 LF of sewer and 700 laterals. While all the sewer mains and manholes were addressed, the funding for Phase 4 had unacceptable conditions for any work conducted on private property. Many laterals slated for full rehabilitation were addressed either at the connection only or to the edge of the public ROW. Only 577 laterals were reconstructed, with most lateral rehabilitation being done in the public ROW. Upper laterals were inspected using closed-circuit television and only those that were clearly structurally deficient or actively leaking were rehabilitated. In addition, some rerouting and upsizing was conducted to reduce the occurrences of overflows upstream in the system, particularly at the upstream end of the Ames Creek siphon. The City also elected to have some additional grouting (non-structural) work performed to augment the RDII reductions. Figure 1-6 shows the extent of the Phase 4 project.

In total, the City's I/I abatement program has addressed 92,500 LF of sewer main, or approximately 35 percent of the sewers in the city. Approximately 1,250 laterals have been rehabilitated or replaced, or 30 percent of the laterals in the city. Table 1-2 shows the breakdown of existing sewers and rehabilitated sewers by sewer basin.

Table 1-2. Post-Phase 4 Summary of Work by Sewer Basin					
Sanitary Basin	Total pipe, LF	Rehabilitated pipe in Phases 1, 2, and 3, LF	Rehabilitated pipe in Phase 4, LF	Total rehabilitated pipe, lf	Remaining non-rehabilitated pipe, lf
1	17,920	2,053	7,320	9,373	8,547
2	14,030	10,821	2,851	13,672	358
3	5,220	4,444	719	5,163	57
4	12,500	1,325	8,434	9,759	2,741
5	13,500	9,405	2,280	11,685	1,815
6	9,700	3,567	3,500	7,067	2,633
7	9,400	0	0	0	9,400
8	17,600	0	2,550	2,550	15,050
9	12,230	2,053	5,558	7,611	4,619
10	21,350	0	4,167	4,167	17,183
11	16,000	0	1,768	1,768	14,232
12	12,700	0	438	438	12,262
13	14,150	0	0	0	14,150
14	14,050	0	1,314	1,314	12,736
15	15,100	0	0	0	15,100
16	19,400	0	367	367	19,033
17	5,300	0	0	0	5,300
18	8,800	0	763	763	8,037
19	5,436	5,144	0	5,144	292
20	15,220	0	11,757	11,757	3,463
21	3,200	0	0	0	3,200

Figure 1-7 shows the extent of all work from Phases 1, 2, 3, and 4.

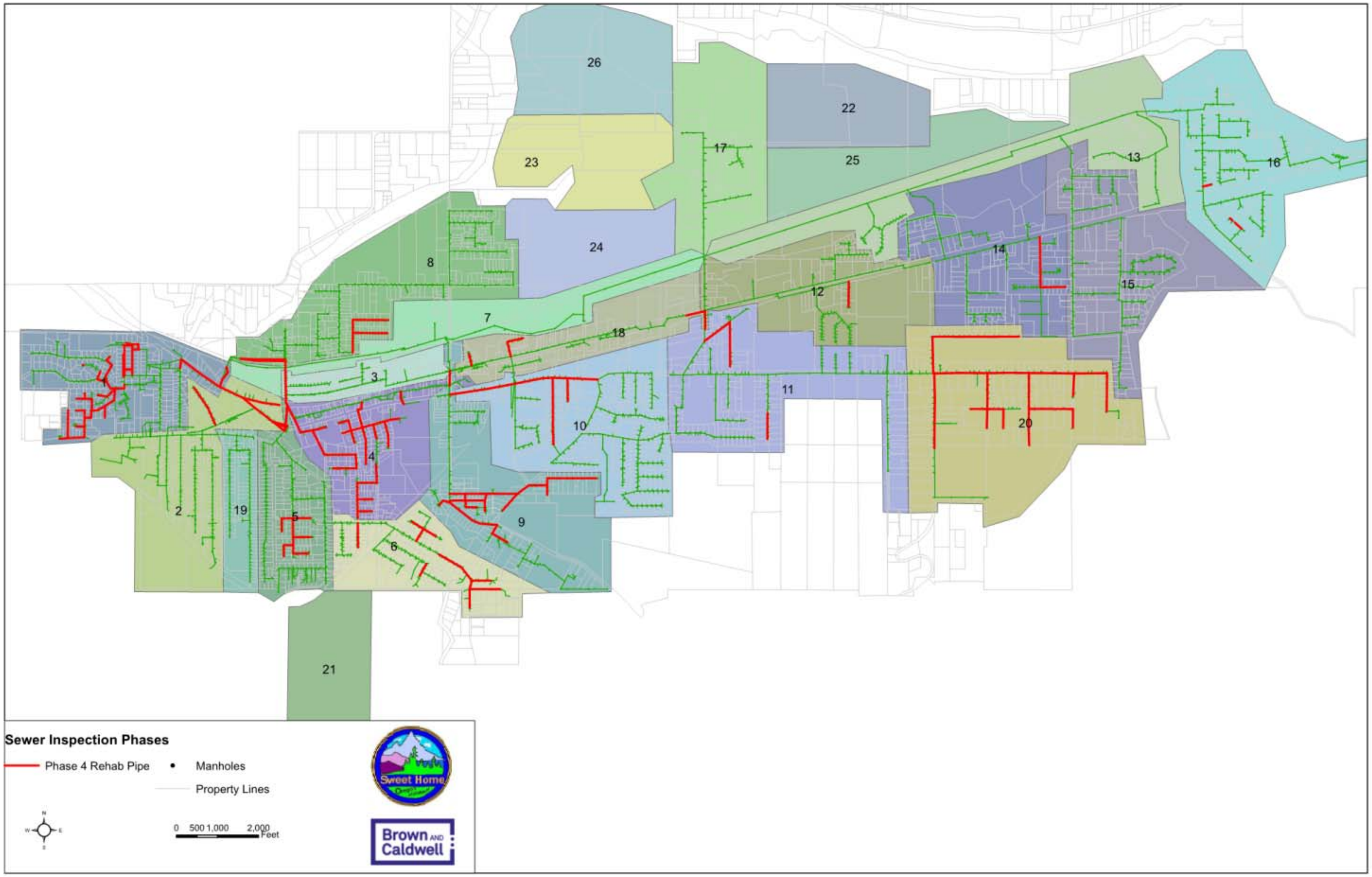


Figure 1-6. Phase 4 project extents

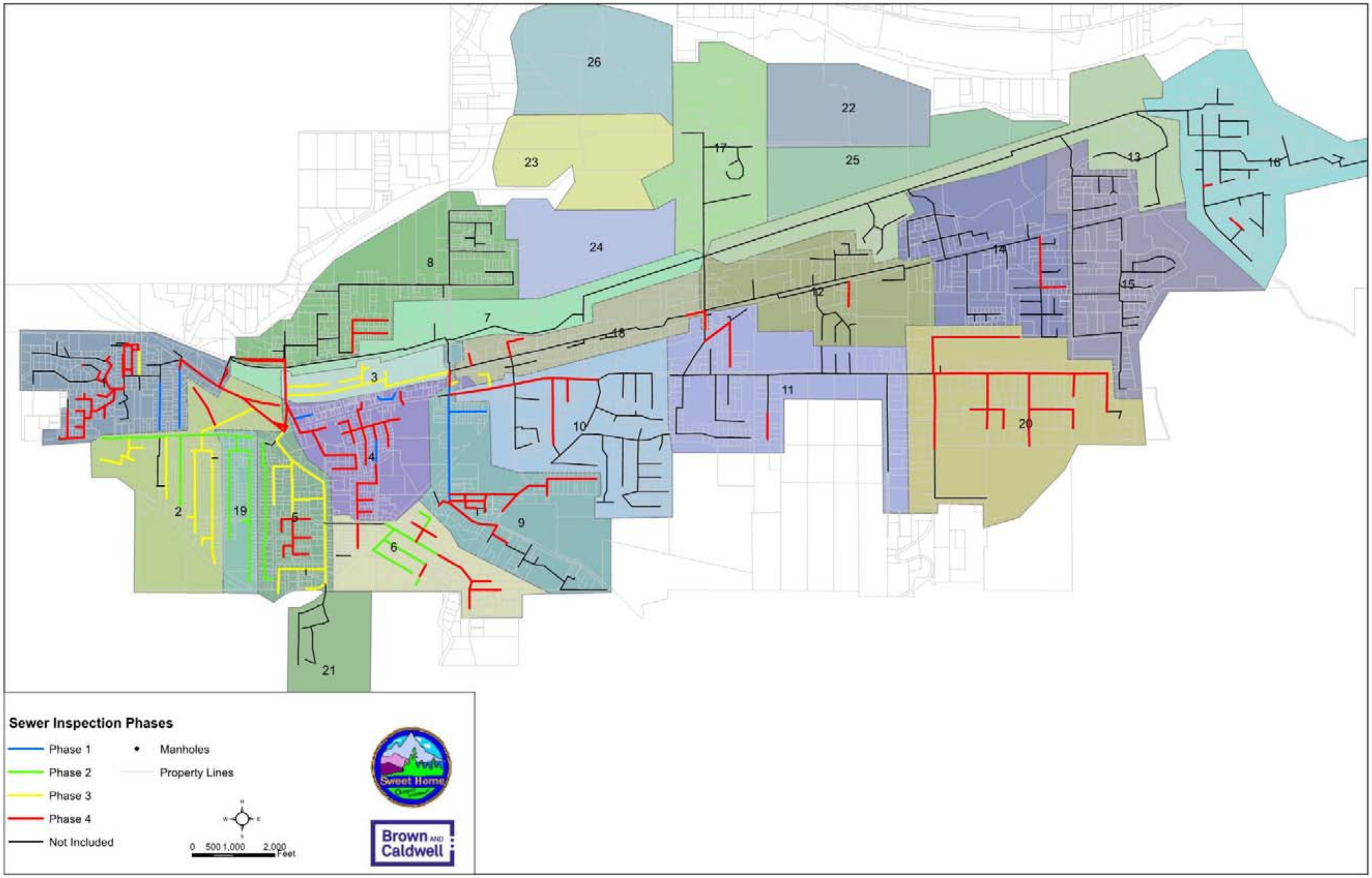


Figure 1-7. Phases 1, 2, 3, and 4 R&R work

Section 2

Rehabilitation Effectiveness

This section describes the hydrologic and hydraulic modeling effort undertaken in Phase 4 to represent the hydrologic response of the collection system to rainfall and to identify areas of limited conveyance capacity.

After completion of Phases 1 and 2 construction, flow monitoring and modeling were conducted to quantify the benefits of the rehabilitation program and allow accurate infiltration/inflow (I/I) reduction estimates to be made. These estimates were used to determine the most cost-effective methodology for rehabilitation and to focus the capital investments on the leakiest basins. Sanitary basins that underwent rehabilitation of both the mains and laterals have the greatest reduction in I/I.

2.1 I/I Reduction from Mainline Rehabilitation

In Phases 1 and 2, sewer mainlines only were rehabilitated in six smaller subbasins. However, only three subbasins had both pre-rehabilitation flow data and post-rehabilitation data of sufficient quality to assess the effectiveness of the work.

I/I reduction resulting from mainline rehabilitation ranges from 11 to 16 percent. This minimal reduction can be attributed to many factors including lateral connection quality, condition of laterals, manhole connection quality, and incomplete rehabilitation. A leaky sewer system can depress the groundwater in the surrounding area. When only mainlines are rehabilitated, the groundwater table rises and enters the sewer system at a higher defect. Figures 2-1 and 2-2 include a comparison of the Log-Pearson Type III plots for pre- and post-rehabilitation of the sewer mainlines in Sanitary Basins 2, 5, and 19.

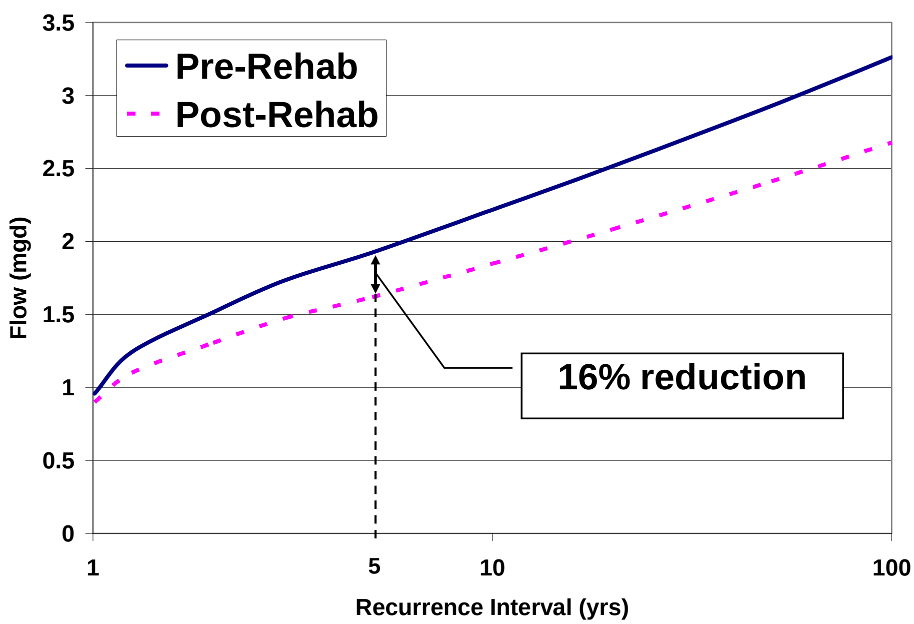


Figure 2-1. Pre- and Post- rehabilitation flow rates for portions of Sanitary Basins 2 and 19

(60 percent of mainlines rehabilitated)

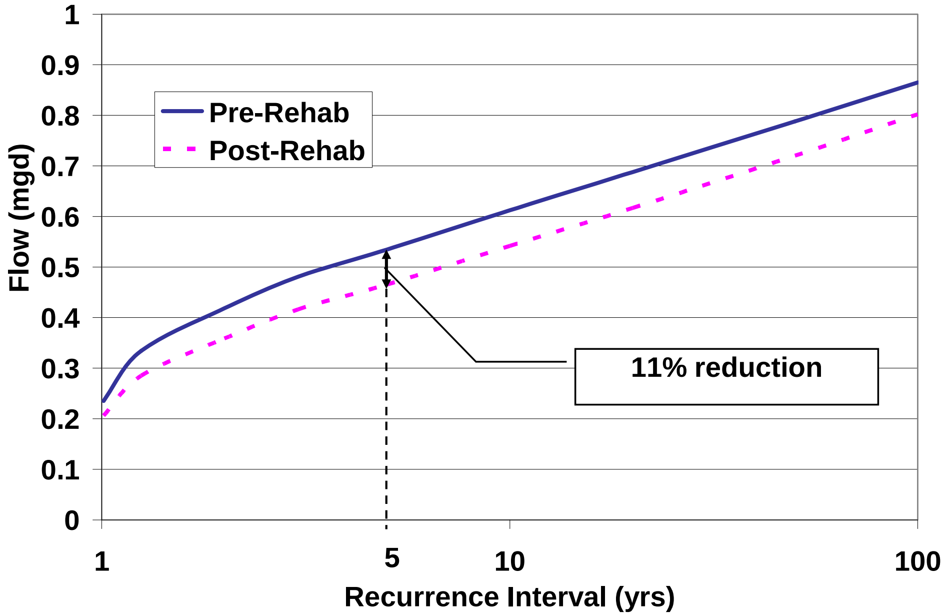


Figure 2-2. Pre- and post-rehabilitation flow rates for portions of Sanitary Basin 5

(100 percent of mainlines rehabilitated)

2.2 I/I Reduction from Lateral Rehabilitation

Service laterals were rehabilitated in Sanitary Basins 4, 5, and 6. In these basins, the work varied from complete lateral replacement to only the upper or only the lower laterals, depending on previous work, existing lateral condition, and property access. Lateral rehabilitation included 70 to 83 percent of laterals within any given subbasin.

Pre- and post-rehabilitation flows from Sanitary Basins 4, 5, and 6 are shown in Figures 2-3, 2-4, and 2-5, respectively. It can be seen that I/I reduction ranges from 7 to 40 percent.

The much higher reduction achieved in Sanitary Basin 6 was most likely due to the mainline rehabilitation work performed by the City of Sweet Home (City) in 1999 on the lower portion of this basin. In general, since groundwater levels influence when and how much I/I will enter a defect, if an upstream defect is repaired, the groundwater will simply enter a defect at a lower elevation in the same basin. However, since the mainlines at the bottom of the basin already had been rehabilitated, there were fewer defects for the I/I to enter, thus the significantly higher removal rate of I/I.

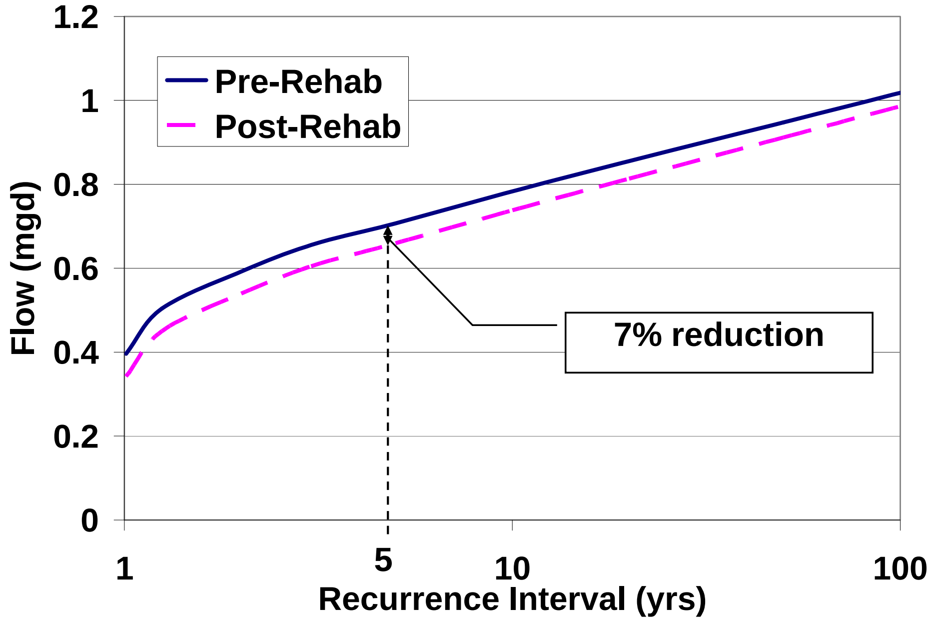


Figure 2-3. Pre- and post-rehabilitation flow rates for laterals in Sanitary Basin 4

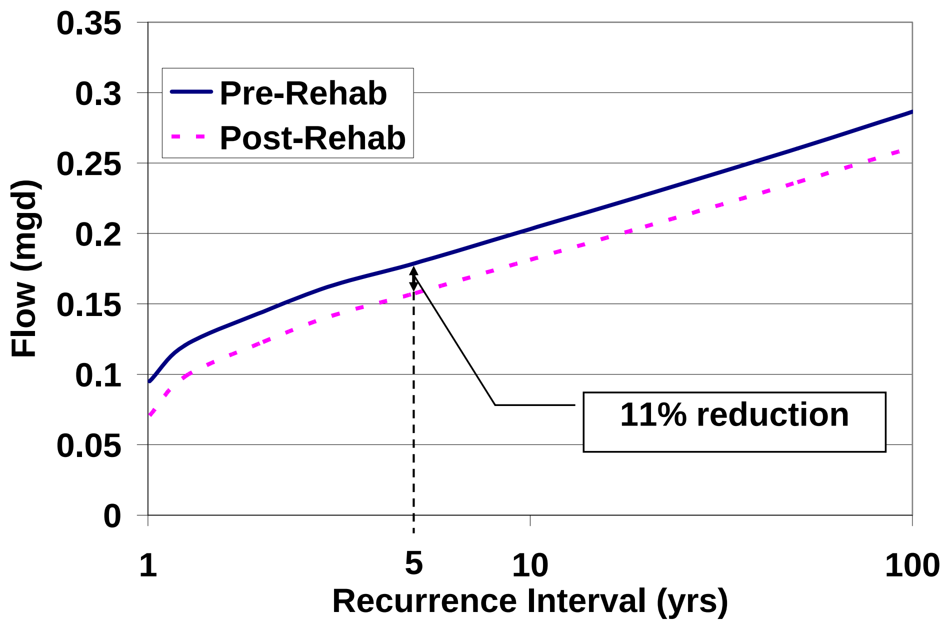


Figure 2-4. Pre- and post-rehabilitation flow rates for laterals in Sanitary Basin 5

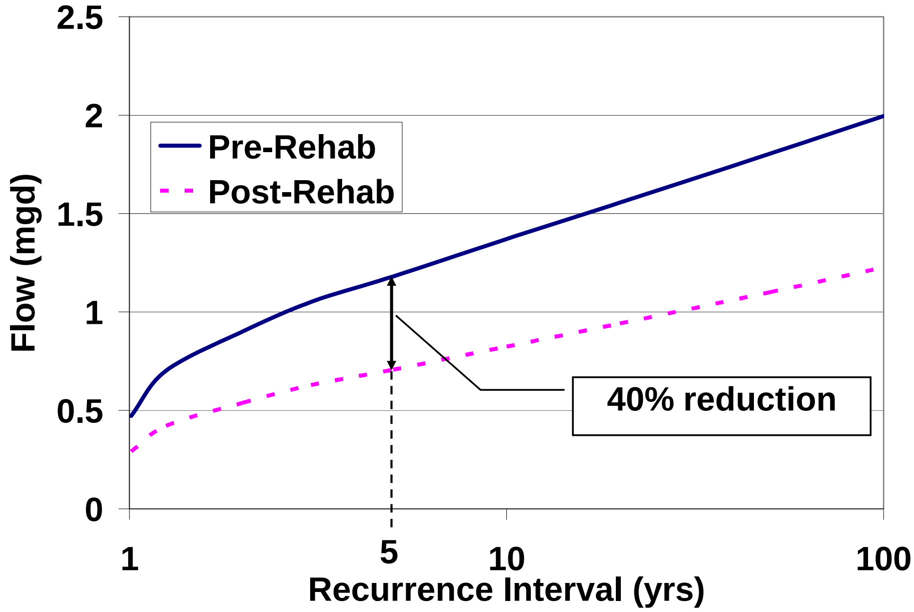


Figure 2-5. Pre- and post-rehabilitation flow rates for laterals in Sanitary Basin 6

2.3 I/I Reduction from Full (Sewer Mains and Laterals) Rehabilitation

Full rehabilitation of mains and laterals was completed in a subbasin in Sanitary Basin 1. Pre- and post-rehabilitation flows are shown in Figure 2-6. It can be seen that approximately 88 percent of the peak I/I was removed through 100 percent rehabilitation of the mains and nearly 95 percent rehabilitation of the laterals.

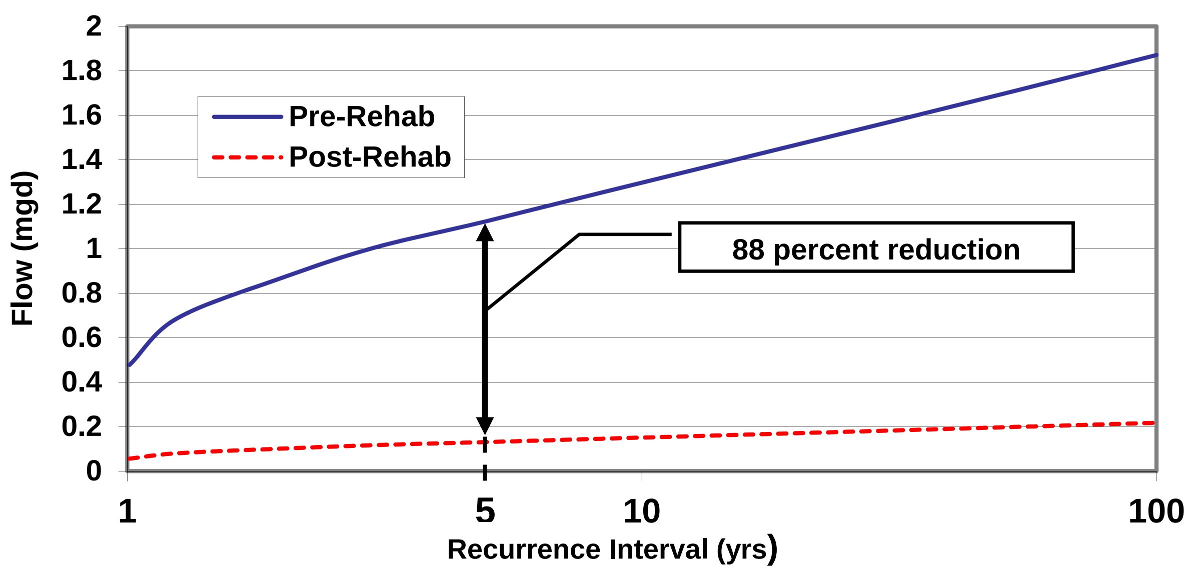


Figure 2-6. Pre- and post-rehabilitation flow rates after full rehabilitation in Sanitary Basin 1

2.4 Cost-Effectiveness of Rehabilitation Strategies

The bottom line for many communities is how much money they have to spend to achieve the desired level of I/I reduction. The cost-effectiveness of the 2003 and 2004 rehabilitation projects in Sweet Home is summarized in Table 2-1. Items such as mobilization, bypass pumping, and traffic control were evenly distributed between rehabilitation basins without weighting for basin size or type of work performed. Construction costs were escalated to approximate 2008 costs. It can be seen that rehabilitating an entire basin (mains and laterals) was, in these examples, 60 to 70 times more effective than doing either mains or laterals alone.

Table 2-1. Cost-Effectiveness of Various Rehabilitation Strategies

Rehabilitation method	Footage or quantity	Construction cost, dollars	I/I reduction, gallons	Dollars per gallons removed
Full rehabilitation	1,200 linear feet (LF) and 15 laterals	398,308	970,000	0.41
Mainlines only	20,000 LF	1,000,502	36,000	27.79
Laterals only	330	1,425,718	54,000	26.40

Prior to this analysis, City policy was to work on the public portion of the sewers and service laterals only. As a result of the rehabilitation effectiveness analysis, holistic rehabilitation efforts were targeted in Phases 3 and 4.

Section 3

Flow Monitoring

The City has engaged in pre- and post-rehabilitation flow monitoring for each phase of its infiltration/inflow (I/I) reduction improvements over the last decade. The purpose of the flow monitoring is to collect flow data from isolated sanitary basins that could then be used to calibrate a hydrologic model. The hydrologic and hydraulic modeling effort then aims to predict theoretical peak-hour flows, determine collection system capacity, and ultimately identify future improvements needed.

3.1 Past Monitoring Efforts

City-owned Isco Model No. 2150 flow monitors were used for the pre-rehabilitation flow monitoring studies in the winters of 2001/2002 through 2004/2005. Isco 2150 flow monitors use continuous wave Doppler technology to measure mean velocity. The sensor transmits a continuous ultrasonic wave, and then measures the frequency shift of returned echoes reflected by air bubbles or particles in the flow. Additional flow monitoring was performed in additional years before and after additional rehabilitation and replacement projects. Figure 3-1 shows the location of the monitors used between 2001 and 2005.

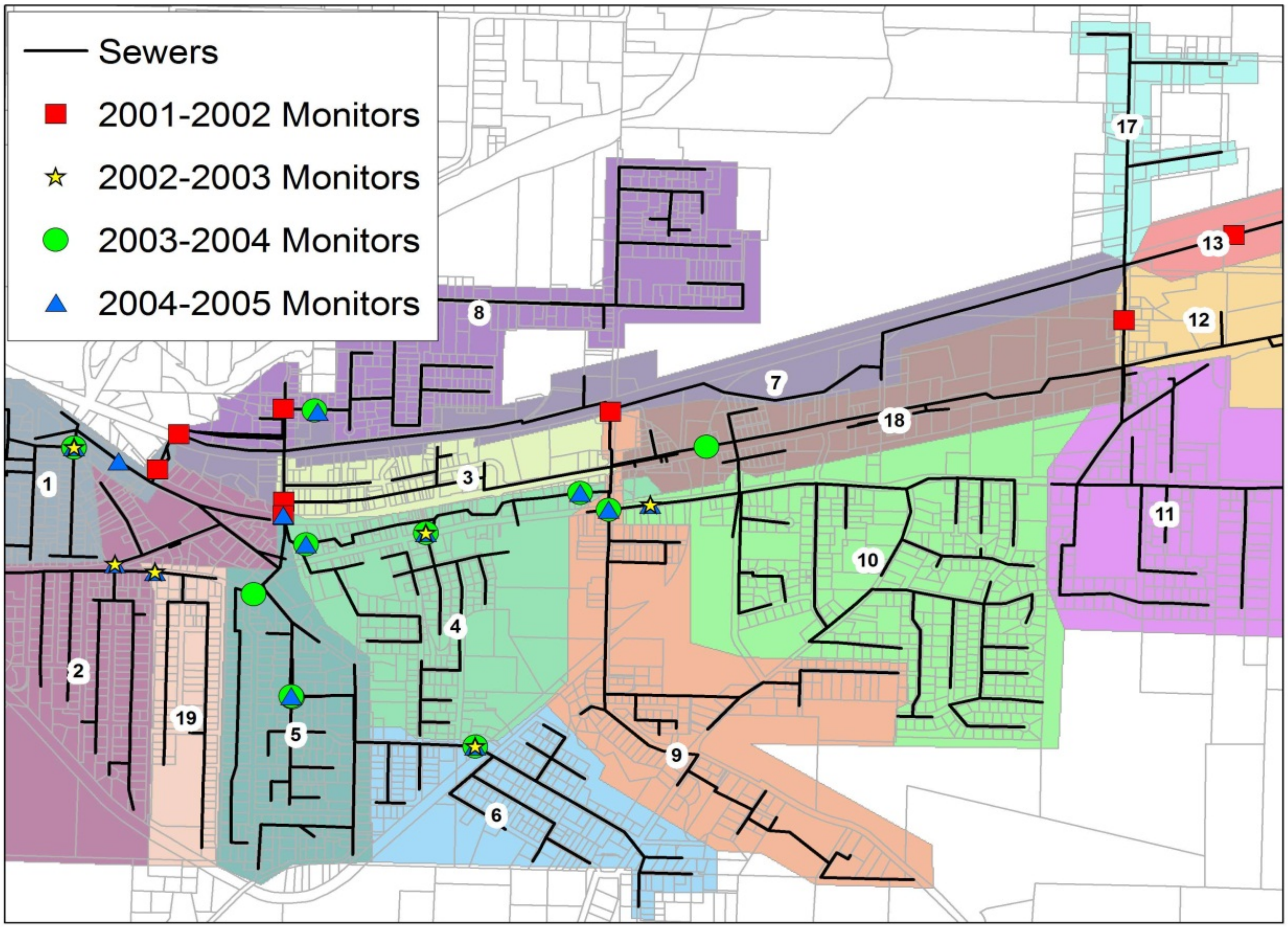


Figure 3-1. Locations of flow monitors from 2001 to 2005

The 2005/2006 winter flow monitoring was performed by SFE Global (SFE), a flow monitoring company based out of British Columbia, Canada. SFE was responsible for installation, download, maintenance, and removal of the flow monitors. Brown and Caldwell (BC) provided oversight, assistance with site and flow monitor selections, and data quality assurance and quality control (QA/QC). Two different types of flow monitors were employed by SFE. The first was a custom compound, sharp-crested weir manufactured by SFE with an Isco Model No. 2150 providing backup flow measurement. The depth of water behind each weir was measured by the Isco 2150 and a custom rating table was used to translate the water depth into a flow rate for each site. The second type of flow monitor was a Datagator® venturi flow meter, manufactured by Renaissance Instruments. These meters combine a modified Venturi flow tube design with pressure transducers at the inlet, throat, and outlet to measure flow under all conditions, including transitional periods between open channel and full pipe. The Datagators translate pressure directly into flow using the continuity and Bernoulli equations. A total of 22 meters were installed by SFE, 11 weirs and 11 Datagators.

Eight of these meters monitored flows from sanitary basins that were rehabilitated in 2003 and 2004 as part of Phases 1 and 2 projects. The other 14 were installed in strategic locations around the collection system to allow a comparison of all major basins in the system and to help guide future I/I rehabilitation work. The locations of all of the monitors and basins are shown in Figure 3-2.

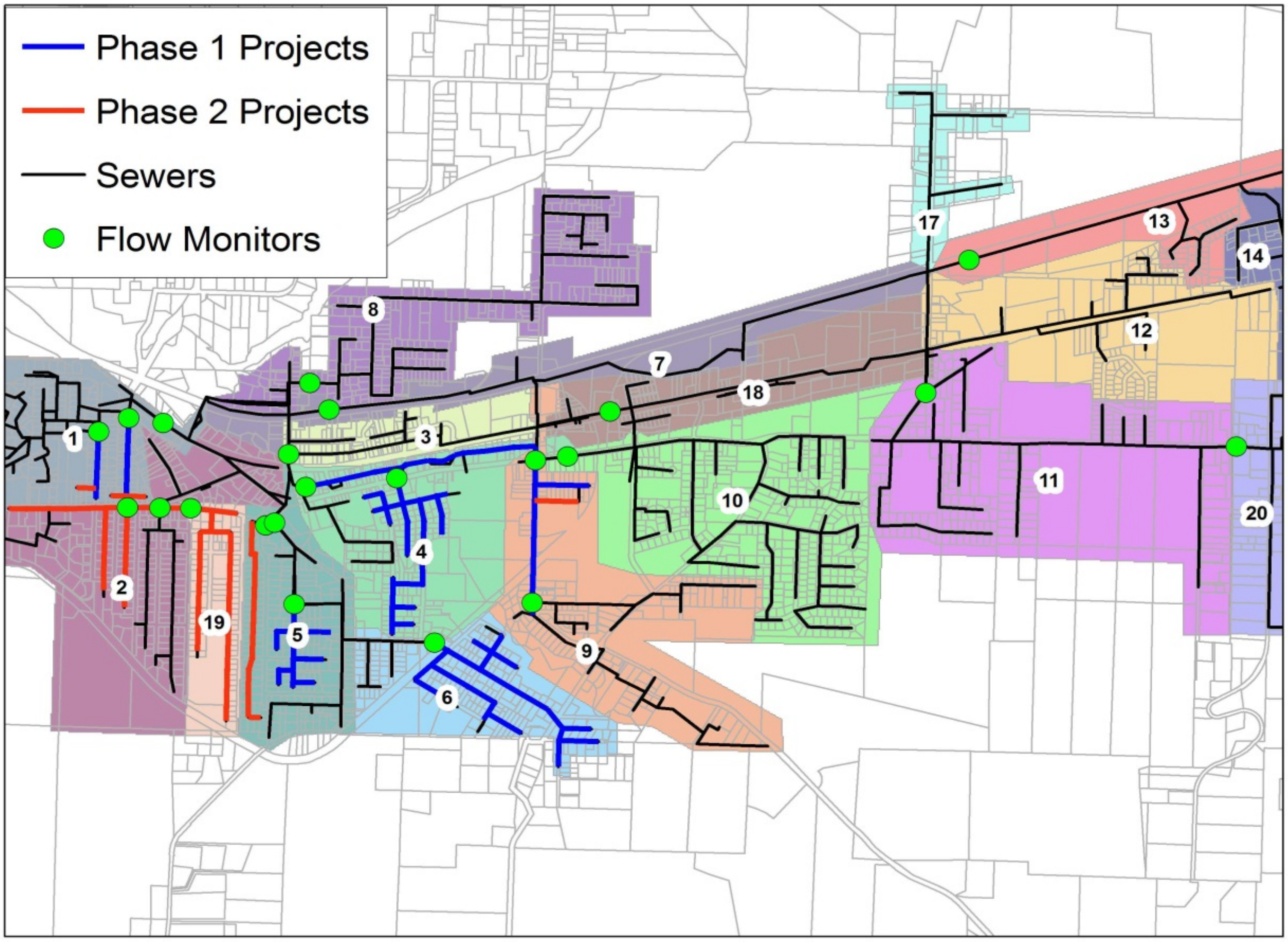


Figure 3-2. Location of 2005-2006 flow monitors

After Phase 3 was constructed, additional flow monitoring was conducted in the winter of 2008/2009 to determine the impact of prior projects and to recalibrate the model. SFE was retained again to conduct the flow monitoring using custom weir flow monitors. The location of the additional eight flow monitors is shown in Figure 3-3.

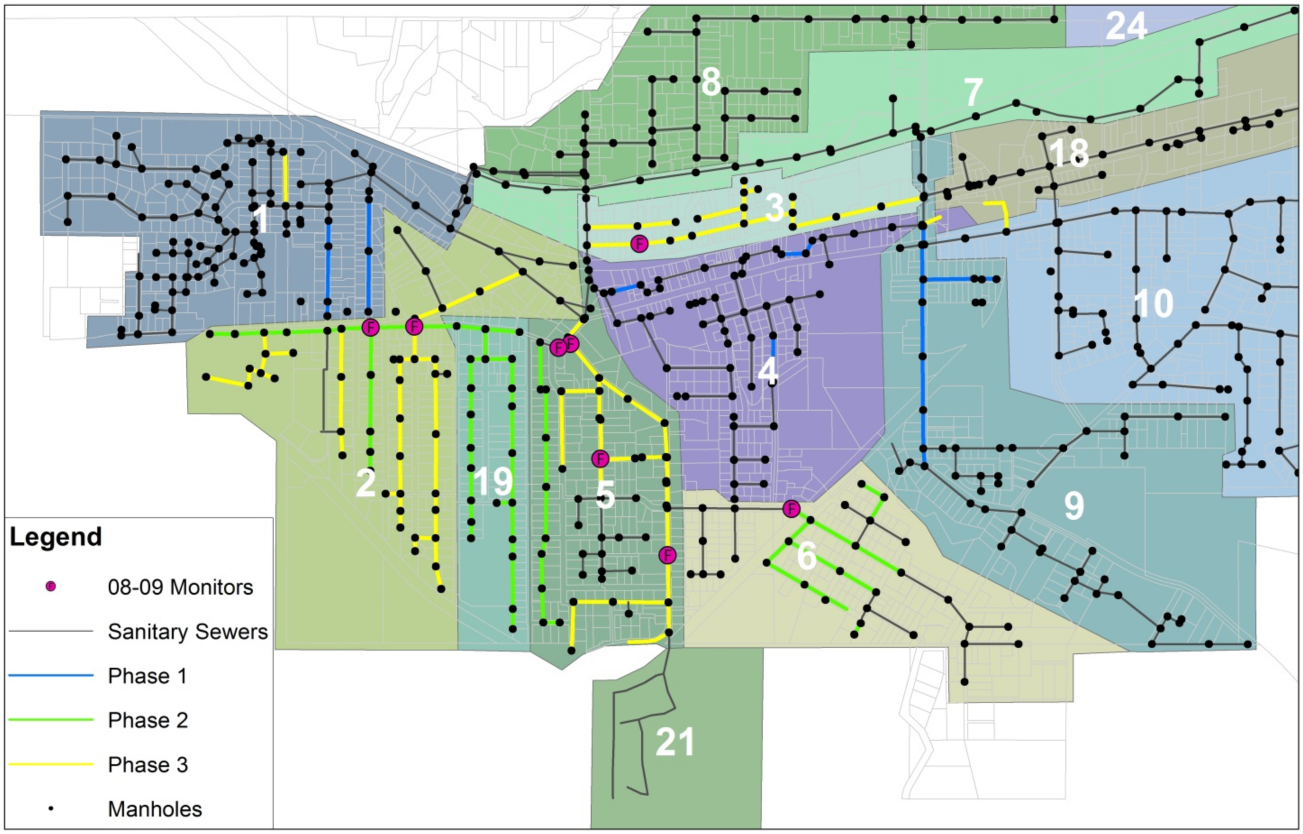


Figure 3-3. Location of 2008/2009 flow monitors

After the conclusion of Phase 4 construction, the City engaged in a post-rehabilitation flow monitoring program during the wet season of 2012/2013. The flow monitoring period extended from November 2012 to March 2013. Monitoring sites were selected based on the ability to isolate portions of the sanitary sewer system for analysis. Site evaluation criteria included site hydraulics, surcharge potential, manhole invert configuration, and pipe diameter.

The flow monitoring was conducted using a combination of ten City-owned ISCO 2150 area-velocity (AV) meters and five SFE-owned weirs. City staff and SFE were each responsible for installation, weekly download and site maintenance, and removal of their respective flow monitors. In addition, effluent flow data from the Sweet Home Wastewater Treatment Plant (WWTP) were collected weekly by City staff. Rainfall data were obtained from the City's total weather station located at the WWTP until it experienced a maintenance failure in February 2013. SFE installed a rain gauge at the City's maintenance yard and continued to collect rainfall data in March.

Figure 3-4 and Table 3-1 highlight the meters used in the post-Phase 4 flow monitoring effort.

Table 3-1. Phase 4 Post-Rehabilitation Flow Monitoring Locations				
Monitoring basin	Location of flow monitor	Meter type and owner	Upstream basins	Corresponding Sanitary basins
All	Sweet Home WWTP		2,3,7	
1A	4th and Main Street	Weir - SFE		1
2	490 Main Street	Weir - SFE	1A, 4, 6	19 (partial)
3	8th Avenue West of 9th Avenue	Weir - SFE	5,8	3, 8 (partial)
4	4th Street	AV – City		2, 19 (partial)
5	Gleaners	AV – City		4
6	Long	AV – City		5, 6, 21
7	Redwood	AV – City		8 (partial)
8	15th Avenue	Weir - SFE	8A, 10, 13, 14	7, 12, 14, 17
8A	18th Avenue at Rail Road	Weir - SFE	9.1, 9.2	18
9.1	Admin	AV – City		10
9.2	Auto Shop	AV – City		9
10	Clark Mill	AV – City	12	11
12	Church	AV – City		20
13	Nandina	AV – City		15
14	Rail Road	AV – City		16

3.1.1 Quality Control Oversight

As part of the post-Phase 4 monitoring, BC provided oversight, assistance with site and flow monitor selections, and QA/QC. Raw data and field maintenance notes from each site were sent weekly in electronic format from both City staff and SFE to BC for validation and verification. Field notes accompanying the data exchange included a digital photograph of each installation, all observations, field verifications, calibrations, and adjustments for each site.

At different times throughout all of the flow monitoring periods, short gaps and inconsistencies in the data were observed due to lost power, faulty calibration, debris, computer malfunction, limitations of the flow monitor, etc. Appendix A contains a detailed analysis of all flow monitoring data collected during the monitoring period.

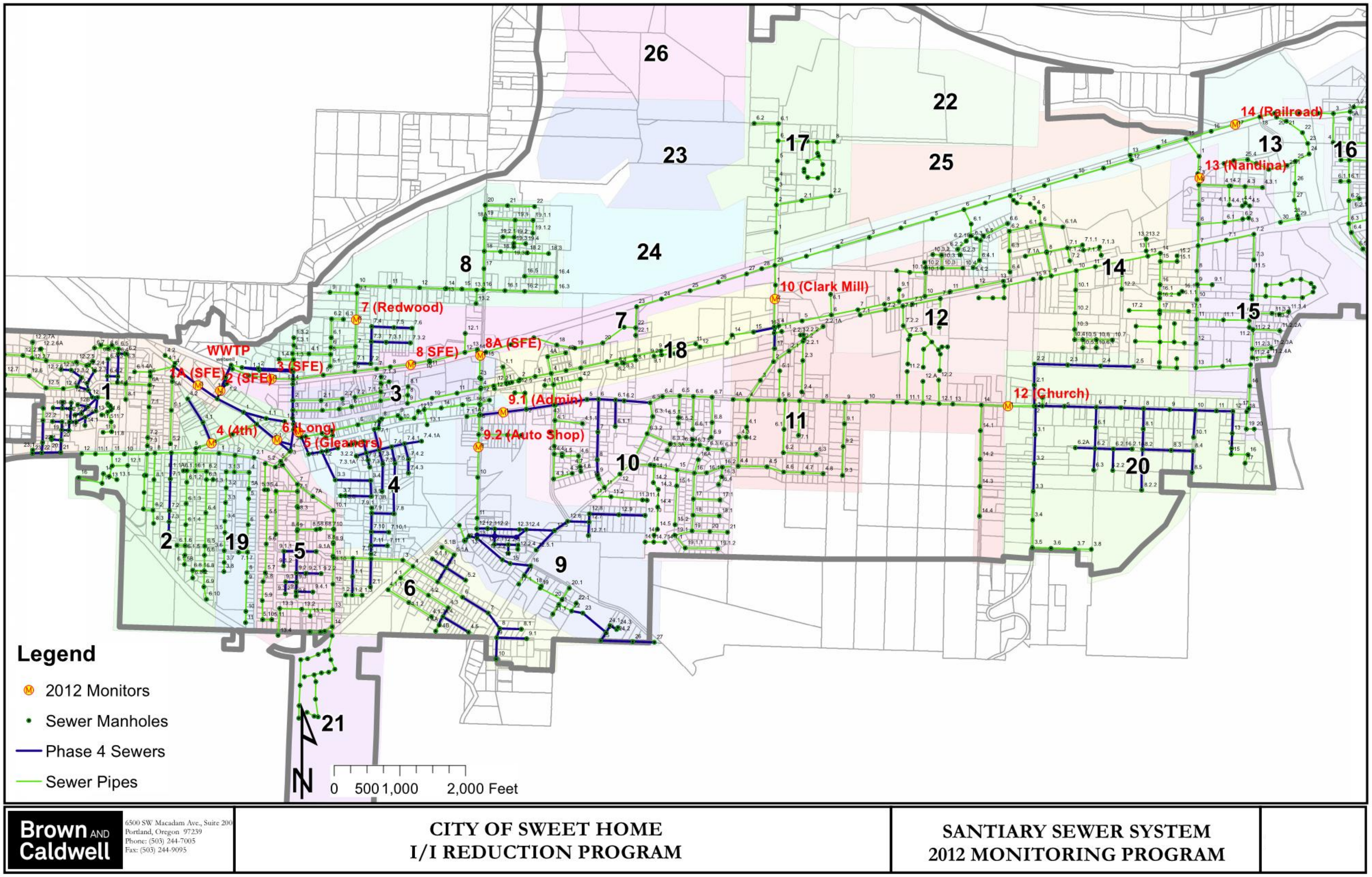


Figure 3-4. Location of 2012/2013 flow monitors

Section 4

Modeling

This section describes the hydrologic and hydraulic modeling effort undertaken in Phase 4 to represent the hydrologic response of the collection system to rainfall, evaluate the effectiveness of infiltration and inflow reduction projects, and to identify areas of limited conveyance capacity and system flooding.

4.1 Hydrologic Modeling

As part of the *Sanitary Sewer Master Plan* (Brown and Caldwell [BC], 2002), hydrologic models were developed in BC's modeling platform, Capacity Assurance Planning Environment (CAPE), to simulate the peak–hour, 5-year recurrence flows at each flow meter in the Sweet Home collection system. The hydrologic engine selected to simulate rainfall-derived infiltration/inflow (RDII) at each flow meter is the Stanford Watershed I/I (SWII) model, which simulates impervious runoff as well as subsurface rapid and long-term infiltration. Throughout the four phases of sewer system rehabilitation, flow monitoring data have been collected to calibrate the SWII models.. As a result of the modeling efforts, the 5-year peak-hour flow under existing and future population projections can be estimated at each point in the rehabilitation process and the effectiveness of sewer rehabilitation projects can be quantified between rehabilitation phases.

4.1.1 Hydrologic Data Sources

The following subsections describe the inputs to the hydrologic models necessary for model calibration ands long-term simulations for recurrence statistics.

4.1.1.1 Basin Delineations

Sweet Home is divided into 27 sanitary basins that do not necessarily share the same borders as the flow monitoring basins. Phase 4 flow monitoring generally was coarser spatially than flow monitoring efforts in previous phases. Some flow monitoring basins, such as Monitoring Basin 6, used to have multiple flow monitors installed for purposes of refining I/I abatement activities. Now, these areas are represented by a single flow monitor. Because of this, the monitoring basins had to be delineated before the hydrologic models could be set up because the basin area tributary to each flow meter is a necessary model input. Some basins, such as Sanitary Basin 12 (Church) were unchanged from previous monitoring efforts (apart from the meter ID). Figure 4-1 shows the delineated monitoring basins for the Phase 4 meters. The area of these basins were calculated using ArcGIS 10.1.

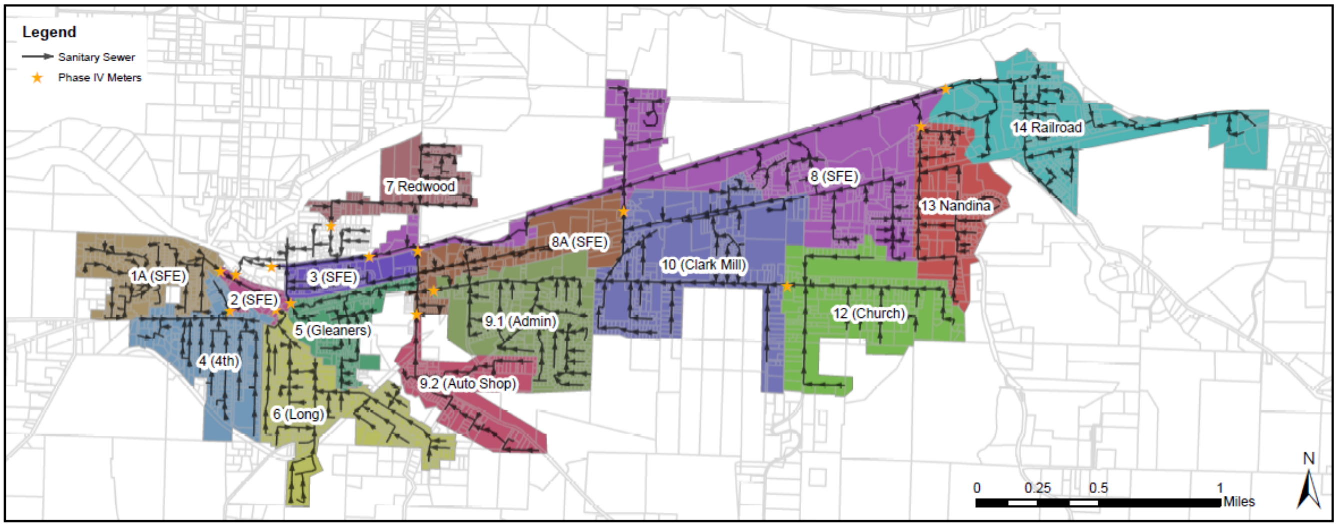


Figure 4-1. Phase 4 monitoring basins

4.1.1.2 Local Rainfall

A local representative rainfall dataset is necessary to calibrate the hydrologic models to the observed flow data. The closer in proximity the rain gauge is to the flow meters, the more likely the rainfall data will be representative of the rainfall that fell on the monitoring basins when the flow was monitored. The City operates a rain gauge at the Sweet Home Wastewater Treatment Plant (WWTP) that collects rainfall at a 15 minute interval. The City's gauge failed twice during the Phase 4 monitoring period, which created gaps in the record. These gaps were patched to create a complete rainfall record that was used to maintain the water balance of the hydrologic models through the wet season and to appropriately match late season storms.

To patch the rainfall record, data from weather stations in Sweet Home available on WeatherUnderground.com were analyzed for correlation to the WWTP rain gauge during concurrent periods. A National Oceanic and Atmospheric Administration (NOAA)/National Weather Service weather station located at Foster Dam on the eastern edge of town collects rainfall data; however, at the time of this analysis, finalized rainfall data were not available for the Foster Dam gauge operated by NOAA, so it was not considered for record patching. Four available gauges within Sweet Home were analyzed for daily rainfall totals as they compared to daily totals at the WWTP gauge. The flow monitoring firm retained by Brown and Caldwell (BC) installed a rain gauge, KORSWEET4, at the City's Maintenance Yard. The gauge KORSWEET4 correlated best and was selected to be used in patching the local rainfall record. As the calibration process began, it was noted that the models consistently over-predicted flows on the March 19th event. The WeatherUnderground gauges showed lower rainfall totals for the storm than the WWTP gauge read, so the WWTP rainfall data were replaced with KORSWEET4 data for the March 19th event.

Table 4-4 summarizes the rainfall sources used to create a composite local rainfall record for use in calibrating the 15 hydrologic models.

Table 4-1. Local Rainfall Record Sources			
Source	Start date	End date	Reason
WWTP	10/14/2012 00:00	12/26/2012 12:15	WWTP rainfall available
KORSWEET4	12/26/2012 12:30	01/02/2013 15:00	WWTP rainfall data gap
WWTP	01/02/2013 15:15	01/31/2013 13:45	WWTP rainfall available
KORSWEET4	01/31/2013 14:00	03/06/2013 04:00	WWTP rainfall data gap
WWTP	03/06/2013 04:15	03/19/2013 15:45	WWTP rainfall available
KORSWEET4	03/19/2013 16:00	03/21/2013 00:00	WWTP rainfall not representative
WWTP	03/21/2013 00:15	03/27/2013 08:00	WWTP rainfall available

The local rainfall dataset, shown in green in Figure 4-2, is plotted against the flow monitoring data for meter 8. As the graph shows, the high flows correspond to periods of rainfall.

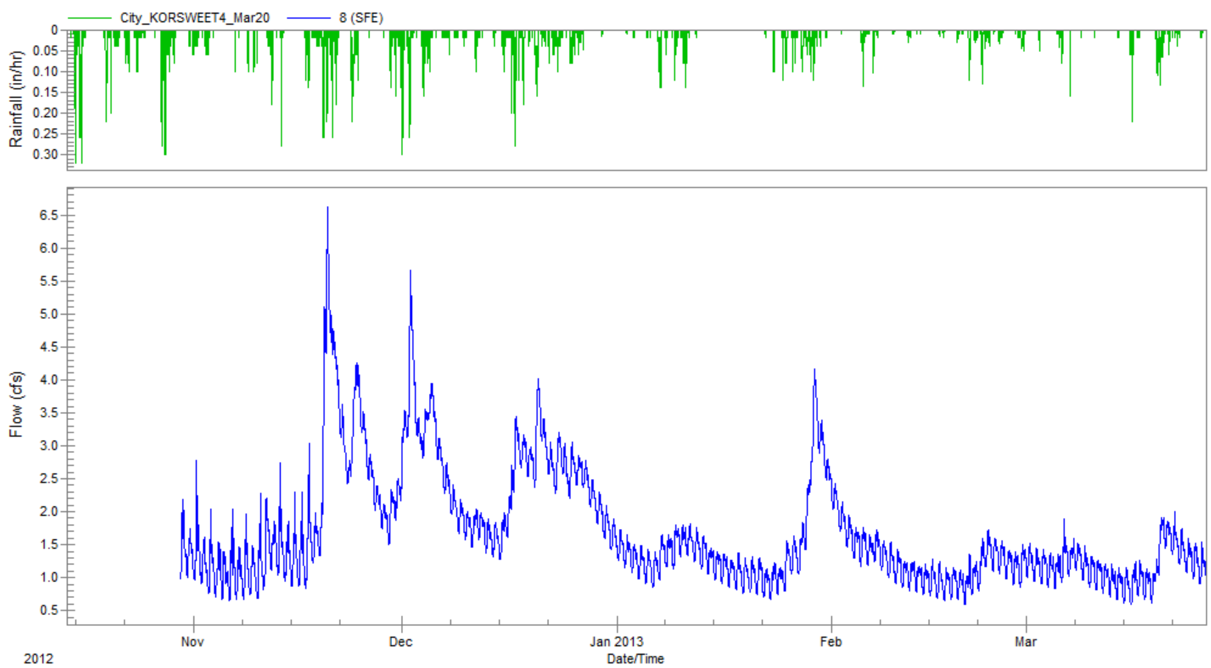


Figure 4-2. Local rainfall plot

4.1.1.3 Long-Term Rainfall

Long-term rainfall datasets allow hydrologic models to be run over the course of many years. The predicted long-term flow datasets are used to calculate recurrence statistics on peak flows by looking into the large events of the past which were not monitored. The City's gauge at the WWTP has not been in service long enough to run the hydrologic models for a period sufficient to calculate recurrence statistics.

The rain gauge at Foster Dam (operated by NOAA) on the eastern edge of Sweet Home has been in service since November 1969, collecting rainfall at an hourly interval. In previous analyses of RDII rehabilitation effectiveness for the City, BC has used the rainfall record from Foster Dam that spans from

November 1, 1969 through April 14, 2009. For consistency, we have used this same long-term rainfall dataset in Phase 4 modeling for long-term simulations. This provides an apples-to-apples comparison of post-Phase 3 and post-Phase 4 model statistics to determine the change in the 5-year peak-hour RDII flow due to rehabilitation alone. Figure 4-3 shows the spatial relationship of the Foster Dam gauge to the gauges used in creation of the local record.

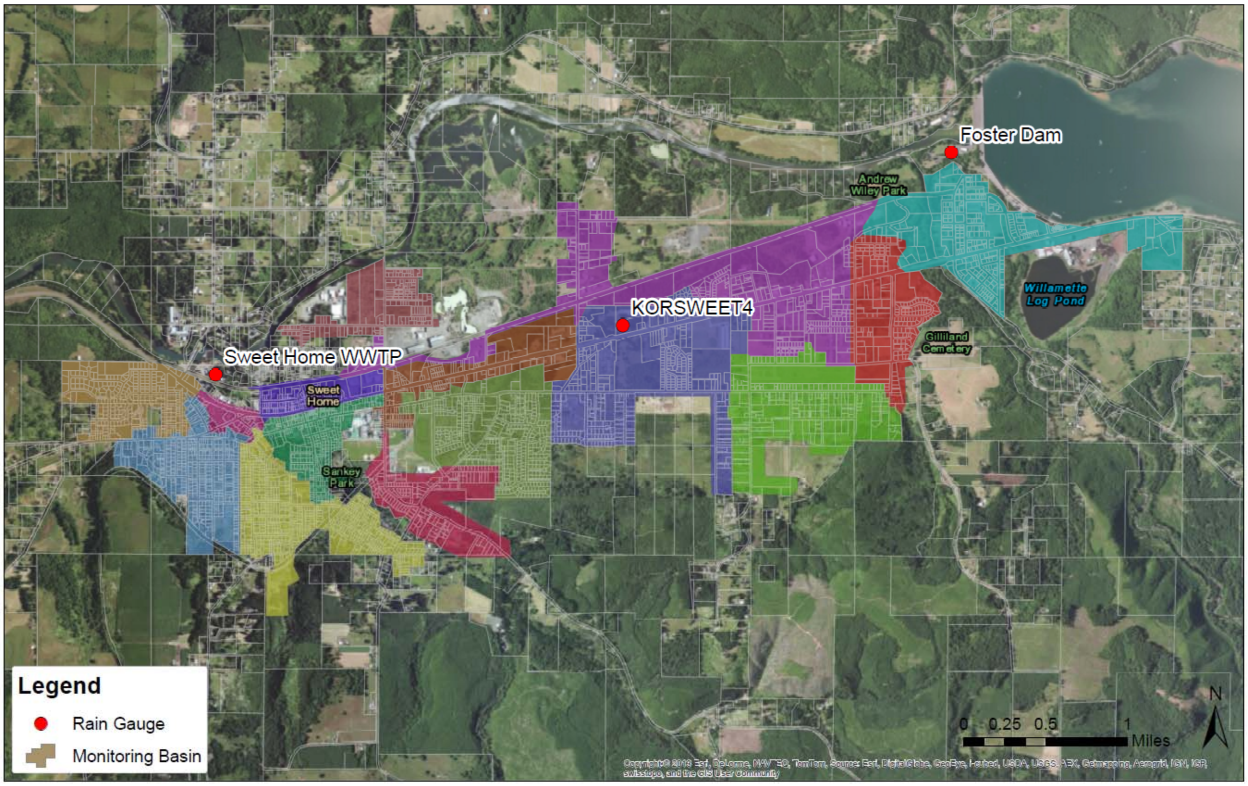


Figure 4-3. Rain gauge spatial relationship

At the time of this analysis, finalized Foster Dam rainfall data were not available for the period concurrent with the monitoring data. However, NOAA did provide raw rainfall data that had not yet been put through the agency's internal QC process. A comparison of the raw Foster Dam rainfall data to the local record was made to determine how well the two gauges likely correlate each other. Event separation was performed on the local gauge using a 24-hour event duration with an additional 6 hours of duration on each side of the event. Rainfall from the two gauges were summed for the events and plotted against each other. Figure 4-4 shows points with a linear best fit line in blue. The red line is the 1:1 plot upon which the points would lie if the gauges correlated perfectly.

The best fit line equation was forced through the origin to prevent a Y intercept from being calculated as both gauges should read zero rainfall on a dry day. The slope of 0.75 indicates that the local gauge may tend to read lower total rainfall for concurrent events than does the Foster Dam Gauge by approximately 25 percent. Although the Foster Dam rainfall data are not finalized, this analysis suggests that the long-term rainfall record may produce more water in the hydrologic models than the calibration with the local gauge would intend. The consequence of this additional rainfall is an element of conservatism in the magnitudes of estimated flows from long-term simulations.

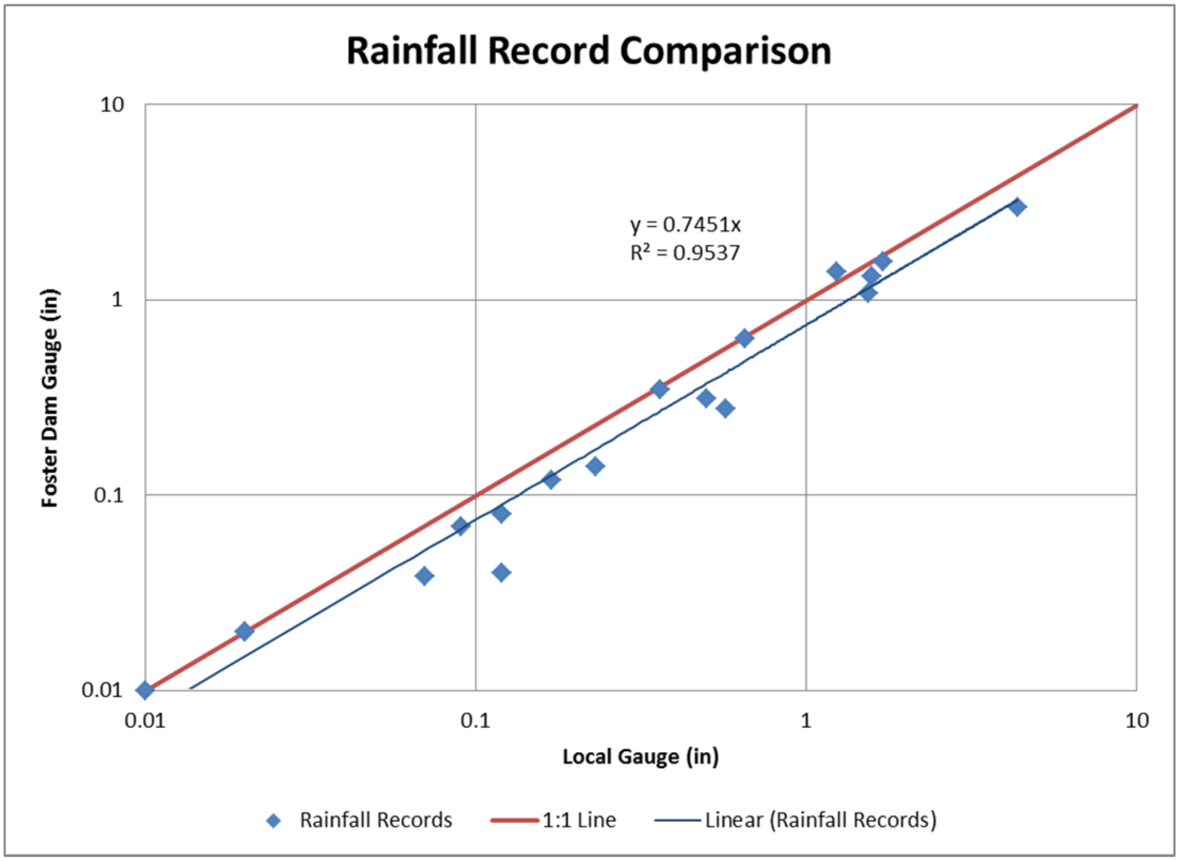


Figure 4-4. Local gauge to Foster dam gauge comparison

4.1.2 Model Calibration and Long-Term Simulation

A hydrologic model was constructed for each of the 15 flow meters deployed in the winter of 2012/2013 as part of Phase 4 rehabilitation monitoring. Calibration to observed flow data was first performed on the most upstream basins which did not have any flow inputs (meters 1A, 4, 6, 7, 9.1, 9.2, 12, 13, and 14). Upon calibration of the models, model flows were input into downstream basin models, which were then calibrated to the downstream meters (meters 2, 3, 8A, and 10). The following sections describe the dry and wet weather flow calibration.

4.1.2.1 Dry Weather Flow (DWF) Calibration

DWF refers to the flow in the sewer system independent of rainfall. It is the wastewater flow produced from household discharges. Over time, a diurnal pattern can be seen due to periods of high and low water use throughout the day. Calibration to this flow is necessary to capture the wastewater component of sewer flow which is not subject to rainfall.

DWF calibration was performed in PCSWMM, using built in tools to develop the factors and flow magnitudes necessary to replicate the observed dry weather diurnal flows in the model. Figure 4-5 shows an example of a DWF calibration for the flow meter for Monitoring Basin 2 flow meter. The red line is the calculated diurnal pattern for the observed (blue) flow monitoring data. The pattern was calculated from the mostly dry period (as there was not a completely dry period in the monitoring record) between November 4 and November 9, 2012. Pattern development periods are chosen on a meter specific basis and the dates used reflect periods where the flows appear to be uninfluenced by rainfall or meter error.

The magnitude of the flow as well as the factors for each hour produced by PCSWMM were then entered into CAPE to produce the diurnal flow pattern in the hydrologic model.

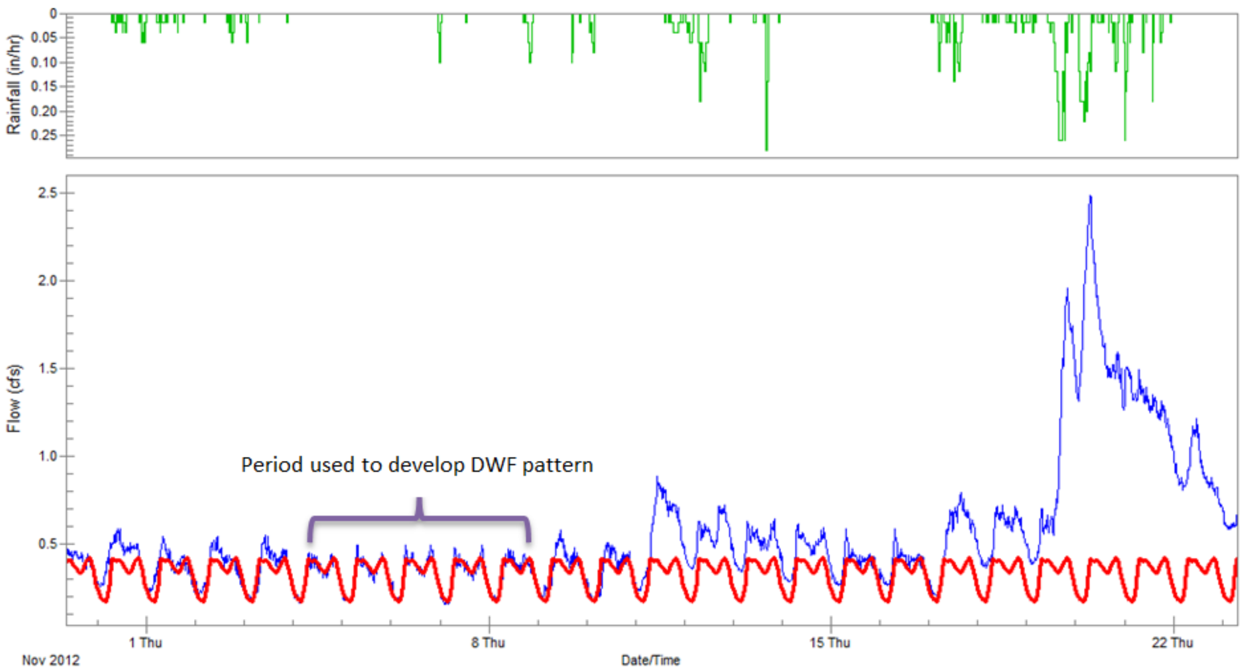


Figure 4-5. DWF pattern development

For the most upstream basins, the calculated DWF magnitude was placed directly into CAPE (as there are no upstream basins contributing to the DWF observed at the meter. The DWF value entered into the CAPE models for downstream basins is calculated as the difference between the calculated magnitude for the downstream meter and the magnitude for the upstream meter (or meters). For example, if the calculated DWF magnitude for meter 12 (upstream) is 0.03 million gallons per day (mgd) and the calculated magnitude for meter 10 (downstream) is 0.2 cubic feet per second (cfs), then the value entered into the CAPE for the Monitoring Basin 10 model is 0.17 mgd (the difference between the two). This contributing flow represents the DWF produced by the downstream basin alone independent of any upstream basins.

The lower most Phase IV flow monitors in the City (2, 3, and 7) were sufficiently upstream from the WWTP that about 7600 feet of pipe went unmonitored. To account for this, scaling factors based on pipe length for monitoring basins 2, 3, and 7 were developed based on the fractional difference in monitored to unmonitored pipe length and are shown in Table 4-2. These factors are necessary to scale flows that are likely created in the unmonitored areas to estimate the full contribution of the City's collection system to the WWTP.

Table 4-2. Unmonitored Area Adjustment Factors

Monitoring Basin	Monitored Pipe Length (ft)	Unmonitored Pipe Length	Factor
2 (SFE)	3,125	450	1.14
3 (SFE)	7,768	680	1.09
7 (SFE)	10,538	6433	1.61

Brown AND Caldwell

Table 4-3 summarizes the DWF values calculated from the observed flow data and the contributing DWF values entered into the hydrology models. The total calculated DWF to the Sweet Home Wastewater Treatment Plant (WWTP) is 1.22 mgd. This number appears to be high compared to anecdotal information about summer time dry weather flows which have been recorded near 0.7 to 0.8 mgd. This high estimation is likely due to the fact that flow meters were not deployed until November, when groundwater is likely to begin infiltrating the sewers and elevating the average low flows. The consequence of this high DWF estimation is that the future growth scenarios will be inherently conservative analyses as this number is used to extrapolate out future DWFs for a larger population.

Table 4-3. DWF Calibration

Flow meter	DWF magnitude, mgd	Upstream basins	Contributing DWF, mgd
1A (SFE)	0.080	N/A	0.080
2 (SFE)	0.217	1A, 4, 6	0.011
3 (SFE)	0.871	5, 8, 8A, 9.1, 9.2, 10, 12, 13, 14	0.217
4 (4th)	0.032	N/A	0.032
5 (Gleaners)	0.075	N/A	0.075
6 (Long)	0.095	N/A	0.095
7 (Redwood)	0.069	N/A	0.069
8 (SFE)	0.597	8A, 9.1, 9.2, 10, 12, 13, 14	0.111
8A (SFE)	0.215	9.1, 9.2	0.089
9.1 (Admin)	0.078	N/A	0.078
9.2 (Auto Shop)	0.048	N/A	0.048
10 (Clark Mill)	0.197	12	0.197
12 (Church)	0.030	N/A	0.030
13 (Nandina)	0.052	N/A	0.052
14 (Railroad)	0.040	N/A	0.04
Total			1.22

4.1.2.2 Wet Weather Flow (WWF) Calibration

WWF calibration is an iterative process of hydrologic model parameter adjustment which seeks to isolate parameters that force the model to respond to rainfall volumes and intensities with the same hydrologic behavior seen in the observed flow dataset. In the case of the SWII model, four main components of the model need to be calibrated to represent the different components of the hydrograph of each monitoring basin accurately. When calibrated correctly, these four components work in tandem to represent directly-connected impervious area, rapid infiltration, interflow infiltration, and long-term groundwater infiltration.

Figure 4-6 shows the four different components of the Monitoring Basin 9.1 calibrated model for three large storms of the calibration period (rainfall is not shown for figure clarity). This model shows that during these storms, long-term groundwater accounted for approximately 0.03 mgd of the total flow. Interflow infiltration was the largest component of the flow with well-defined peaks and a majority of the total RDII volume. Virtual inflow (rapid infiltration) as well as direct inflow (connected impervious area) accounted for moderate portions of the peaks. The sum of the component flows creates the RDII time series.

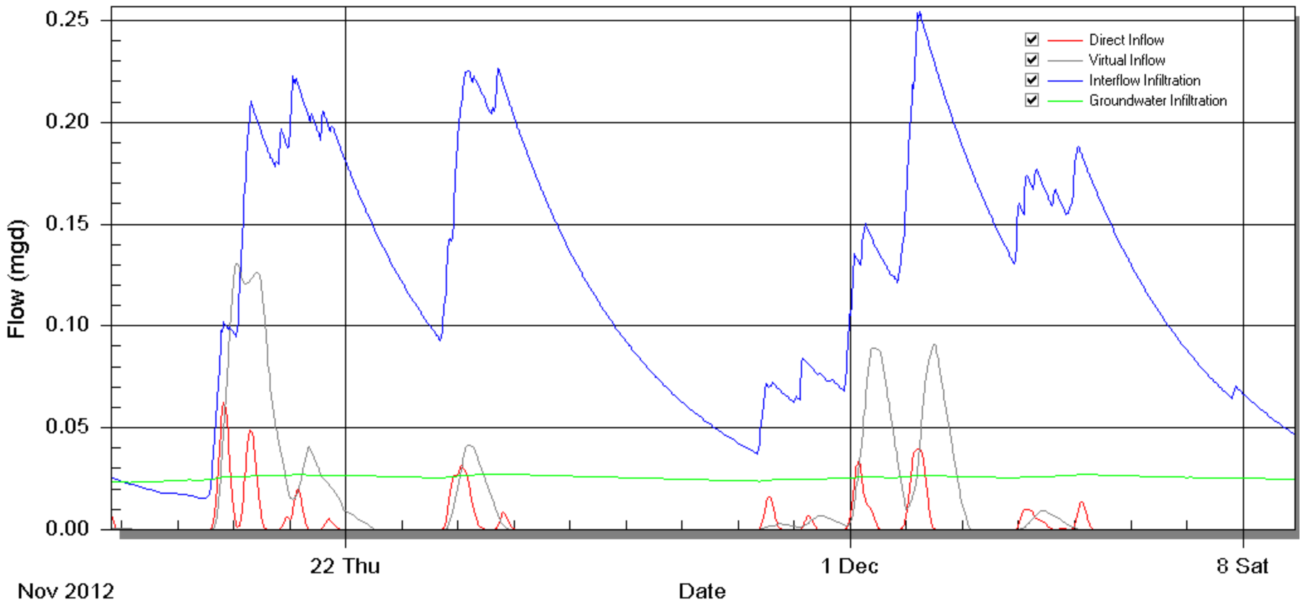


Figure 4-6. Monitoring Basin 9.1 Component RDII Flows

The sum of the RDII and the DWF creates the total flow, which can be seen plotted in red against the observed flow monitoring data (blue) in Figure 4-7. This figure is taken from the calibrated monitoring Basin 9.1 model. A model is determined to be calibrated when further adjustments of the model parameters no longer produce a better fit of the model flows to the observed flow data. In general, a calibrated model will do an accurate job of matching peak flows, rising and falling limbs, and long-term infiltration. A calibrated determination is fundamentally a subjective one because the model will never match observed flow data perfectly (as can be seen in the figure).

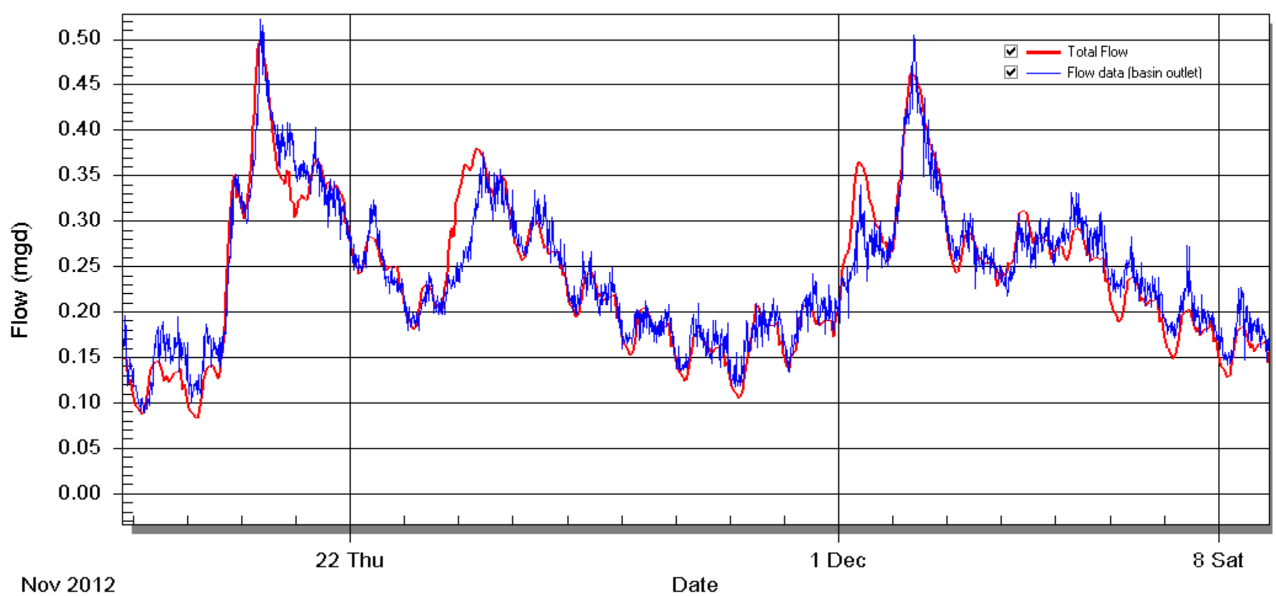


Figure 4-7. Monitoring Basin 9.1 peak storm calibration plot

Zoomed out with the local rainfall data visible (green), the calibrated model can be seen rising and falling along with the flow meter 9.1 monitoring data, as shown in Figure 4-8. The calibration rainfall data (green) is shown for reference.

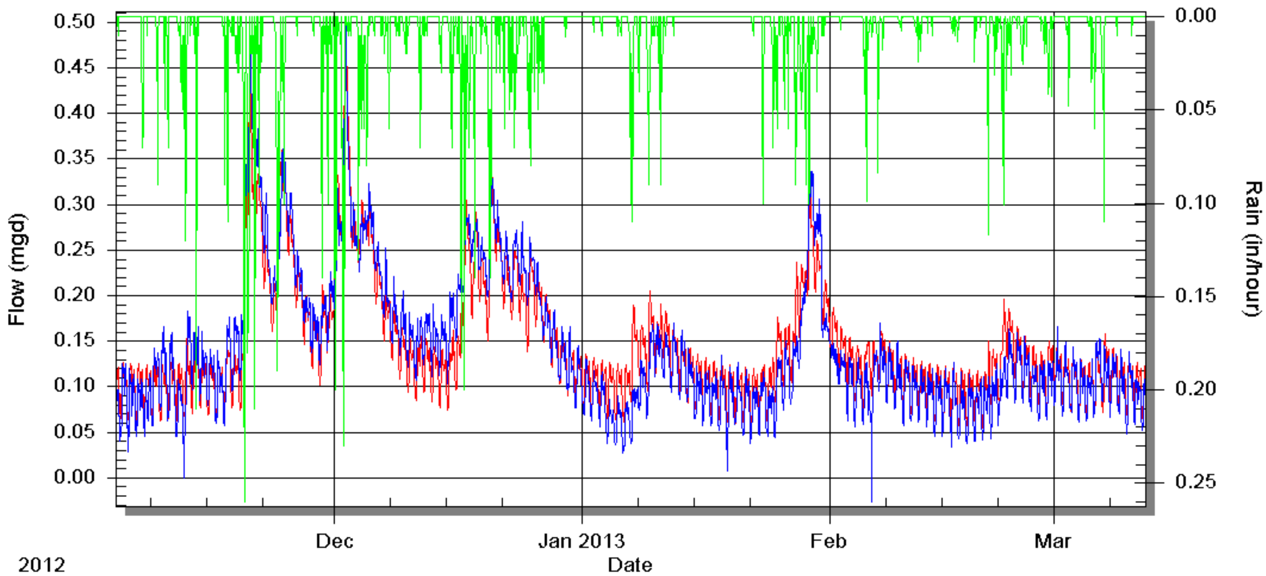


Figure 4-8. Monitoring Basin 9.1 seasonal calibration plot

Once calibrated, the model can be used with the long-term rainfall record to simulate RDII and total flows for 39 years. The long-term flow record can be used to generate flow statistics that describe the hydrologic performance of the basin over time.

4.1.2.3 Long-Term Simulation Statistics

After the model is run through the 39-year Foster Dam rainfall record, the peak hourly RDII discharge values for each year of the record are extracted to create the annual maxima series. The annual maxima series is fit to a Log Pearson Type III (LPIII) distribution to allow for estimation of the peak hourly RDII for any desired return period (also referred to as recurrence interval). This analysis is necessary to estimate the 5-year peak-hourly RDII between rehabilitation phases, which is the statistic used in calculating rehabilitation effectiveness. Figure 4-9 below shows the RDII annual maxima series (points) for monitoring basin 1A plotted on top of the fitted LPIII curve (black). To the right are the estimated flow magnitudes at different recurrence intervals. For monitoring basin 1A, the 5-Year peak hourly RDII is estimated at being 0.57 cfs (0.37 mgd).

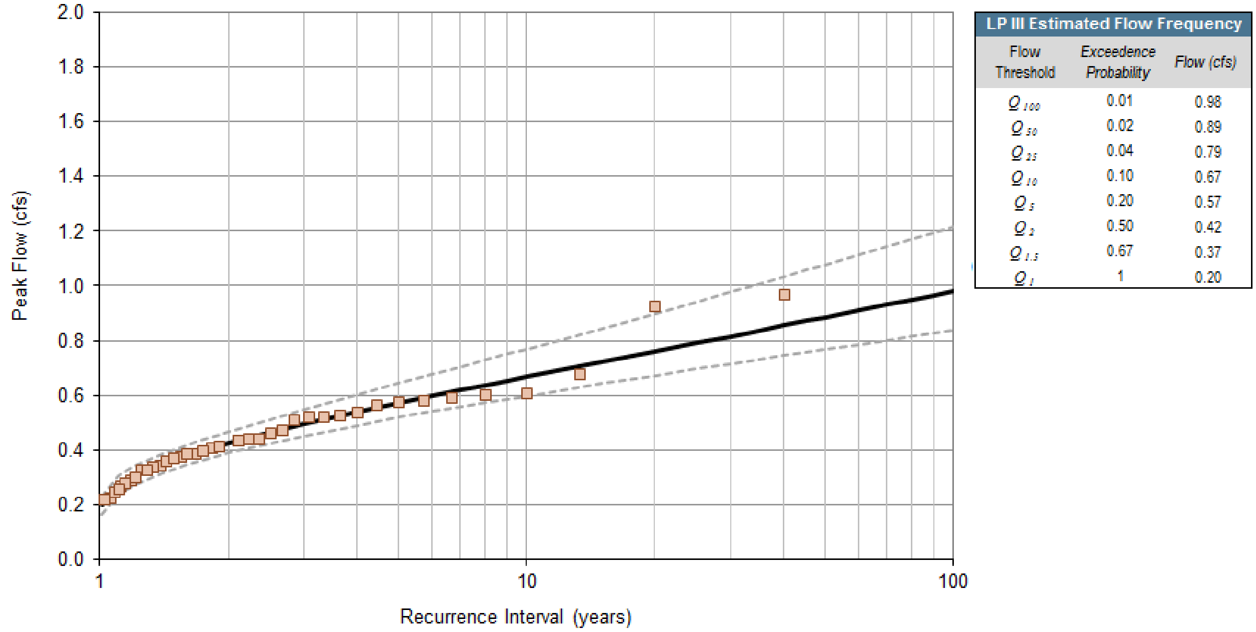


Figure 4-9. Fitted LPIII Curve with 90% Confidence Bounds

Table 4-4 provides the 5-year peak-hour RDII for each of the 15 Phase 4 monitoring basins.

Table 4-4. Post Phase 4 5-Year Peak-Hour RDII

Flow meter	RDII, mgd	Unit area, gpad[a]	Pipe length, gpm[b] per foot
1A (SFE)	0.37	3,647	0.015
2 (SFE)	0.05	2,682	0.007
3 (SFE)	0.66	13,998	0.051
4 (4th)	0.52	4,047	0.020
5 (Gleaners)	0.49	7,006	0.029
6 (Long)	1.32	8,403	0.034
7 (Redwood)	0.35	4,644	0.022
8 (SFE)	2.07	21,148	0.113
8A (SFE)	0.58	1,750	0.011
9.1 (Admin)	0.86	5,124	0.027
9.2 (Auto Shop)	0.59	5,629	0.037
10 (Clark Mill)	0.84	3,156	0.020
12 (Church)	0.31	1,616	0.014
13 (Nandina)	0.49	3,891	0.023
14 (Railroad)	1.09	5,160	0.032

[a] *gpad = gallons per acre per day*

[b] *gpm = gallons per minute*

The model results show that the highest RDII after completion of Phase 4 is being monitored by meter 8. The next highest RDII rates are being monitored by meter 3. Another way to interpret the results is to show the 1-in-5 peak-hour RDII being contributed by each sanitary basin, as shown in Table 4-4.

The highest RDII contributions are coming from sanitary basins 7, 13, 14, and 17, followed by sanitary basin 8.

Table 4-5. Post Phase 4 5-Year Peak-Hour RDII	
Sanitary Basin	RDII, mgd
1	0.36
2, 19	0.57
3	0.46
4	0.48
5, 6, 21	1.30
7, 13, 14, 17	2.38
8	0.55
9	0.58
10	0.84
11, 12	0.82
15	0.48
16	0.89
18	0.57
20	0.30

4.1.3 Rehabilitation Effectiveness

As discussed in Section 2, post-Phase 1/2 modeling revealed a marked decrease in predicted peak RDII when holistic basin rehabilitation is employed. Phase 3 post-construction modeling validated the need for full basin rehabilitation.

As part of the post-Phase 4 flow monitoring effort, five of the 15 flow monitors deployed in the winter of 2012/2013 measured flow from basins which had been rehabilitated as part of Phase 4. Rehabilitation effectiveness for this project is measured by the change in the 5-year peak-hour RDII flow between the pre-retrofit model (post-Phase 3 model) and the post-retrofit model (post-Phase 4 model). The following sections describe how the recurrence statistics are calculated, the calculation method for each rehabilitated basin, and a summary of the RDII removal effectiveness for Phase 4.

4.1.3.1 Phase 4 to Phase 3 Monitoring Basin Crosswalk

The flow meters deployed in Phase 4 below the rehabilitated basins were not always in the exact same location as the meters deployed during Phase 3 monitoring. Furthermore, there may have been multiple Phase 3 meters within the Phase 4 basin, in which case the sum of the RDII time series is necessary. Phase 4 meters 1A, 9.1, and 12 were placed in the same location as their Phase 3 counterparts. Thus, the long-term simulation results from the post-Phase 3 and post-Phase 4 models can be compared without alteration. For meters 5 and 9.2, the Phase 4 meter locations were adjusted from their original Phase 3 locations. Thus, an adjustment to the post-Phase 3 RDII time series was necessary to account for the additional or decreased tributary area upstream of the meter. A factor based on the change in upstream pipe length was applied to the RDII time series of the post-Phase 3 RDII time series so that it can be comparable to the post-Phase 4 RDII. Table 4-6 summarizes Phase 4 to Phase 3 meter crosswalk.

Table 4-6. Phase 4 to Phase 3 Monitoring Basin Crosswalk

Phase 4		Phase 3		Phase 3 RDII scaling factor
Monitoring basin	Total upstream pipe length, feet	Monitoring basin(s)	Total upstream pipe length, feet	
1A	16,747	1, 2, 3	16,747	1.00
5	11,567	9, 10	8,947	1.29
9.1	20,998	19	20,998	1.00
9.2	10,690	17, 18	12,643	0.85
12	14,818	23	14,818	1.00

4.1.3.2 Monitoring Basin 1A (Sanitary Basin 1)

Monitoring Basin 1A monitored the flows from Sanitary Basin 1. Approximately 9,400 linear feet (LF) or 52 percent of the sewers or were addressed in Sanitary Basin 1 since the inception of the program. Since a majority of the rehabilitation took place during Phase 4, the laterals were addressed on an as-needed basis only.

Figure 4-10 shows the extent of rehabilitation and replacement (R&R) work done since the beginning of the I/I Abatement Program.

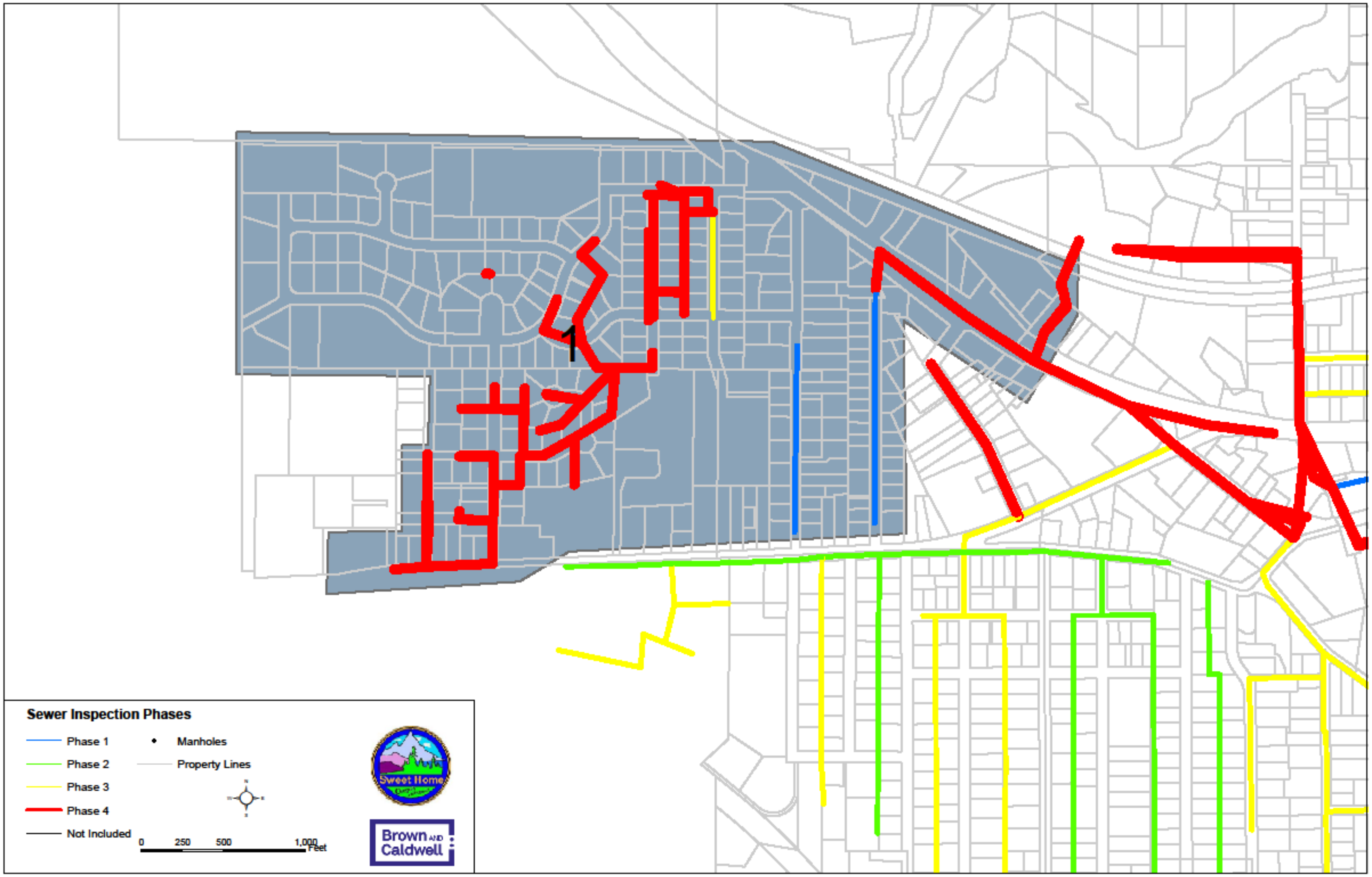


Figure 4-10. Extent of R&R work in Sanitary Basin 1

Phase 3 meters 1, 2, and 3 sum together to comprise the flow seen at the Phase 4 meter 1A, which was metering the flows coming from Sanitary Basin 1. Phase 3 meter 3 is located in the same manhole as Phase 4 meter 1A, thus, no scaling is necessary. The change in the LPIII curve between the two phases can be seen in Figure 4-11. The total reduction in 5-year peak hourly RDII is 0.78 mgd, which represents a 68 percent reduction in RDII since Phase 3. This large reduction in RDII indicates the rehabilitation within this monitoring basin was highly effective in reducing peak flows.

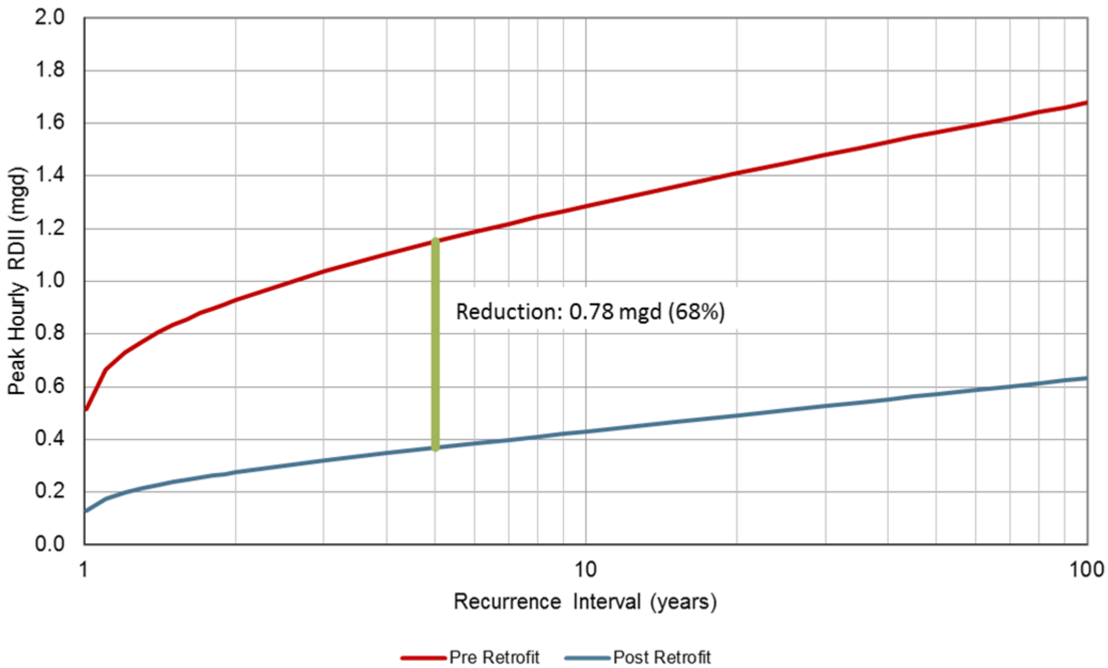


Figure 4-11. Phase 4 monitoring Basin 1A LPIII analysis

4.1.3.3 Monitoring Basin 5 (Sanitary Basins 4 and 6)

Monitoring Basin 5 monitored the flows from Sanitary Basins 4 and 6. Approximately 16,800 LF or 75 percent of the sewers were rehabilitated since the inception of the I/I Abatement Program.

Figure 4-12 shows the extent of work completed in the first four phases.

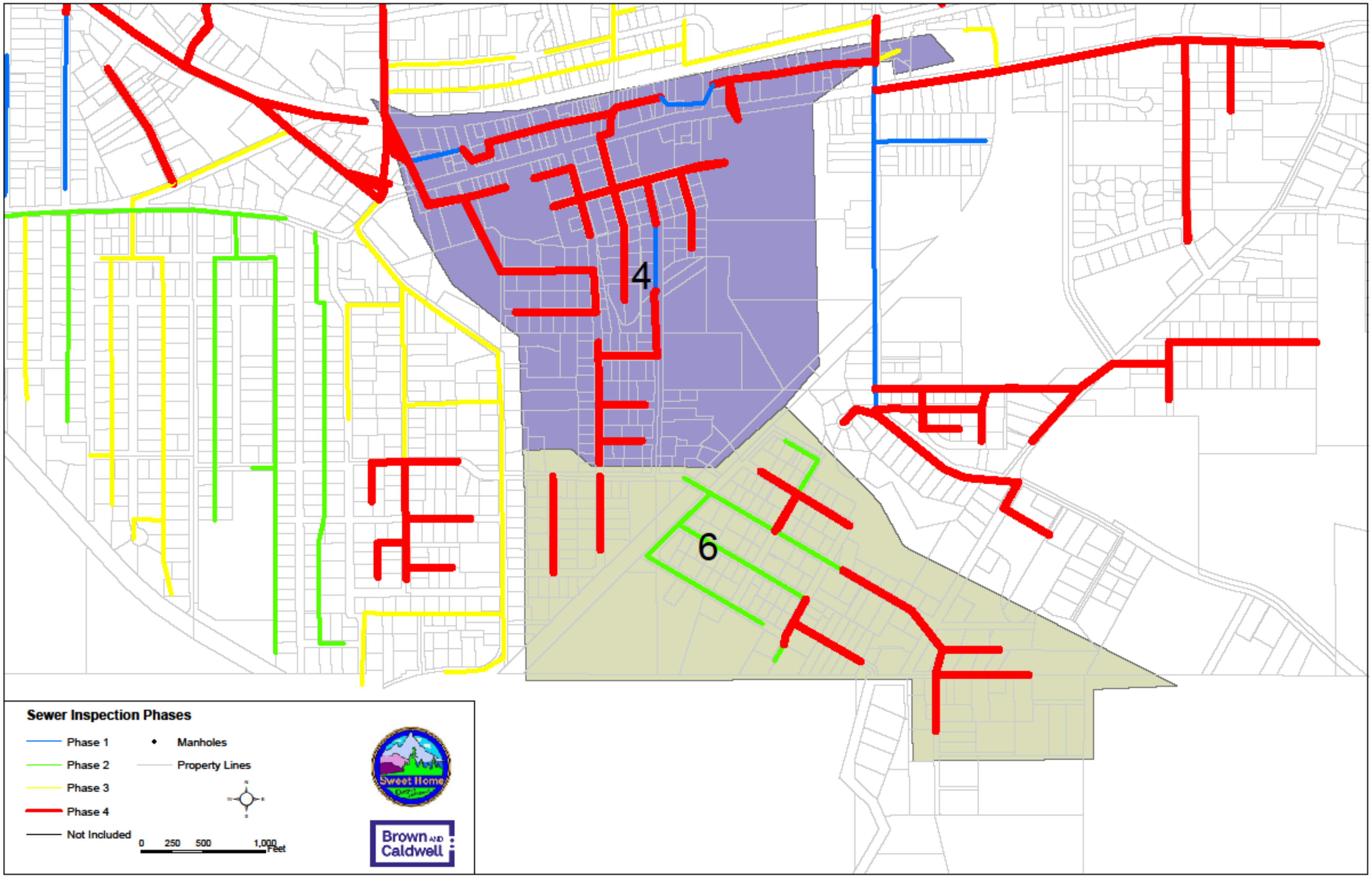


Figure 4-12. Extent of R&R work in Sanitary Basins 4 and 6

Phase 3 monitors 9 and 10 monitored 77 percent of the total pipe length monitored by Phase 4 meter 5. To account for the missing pipe lengths in Phase 3 that were monitored in Phase 4, the sum of the RDII time series from the two Phase 3 meters were scaled by 1.29 to be more comparable with Phase 4 meter 5. The total reduction in 5-year peak-hour RDII is 1.16 mgd, which represents a 70 percent reduction in RDII, as shown in Figure 4-13.

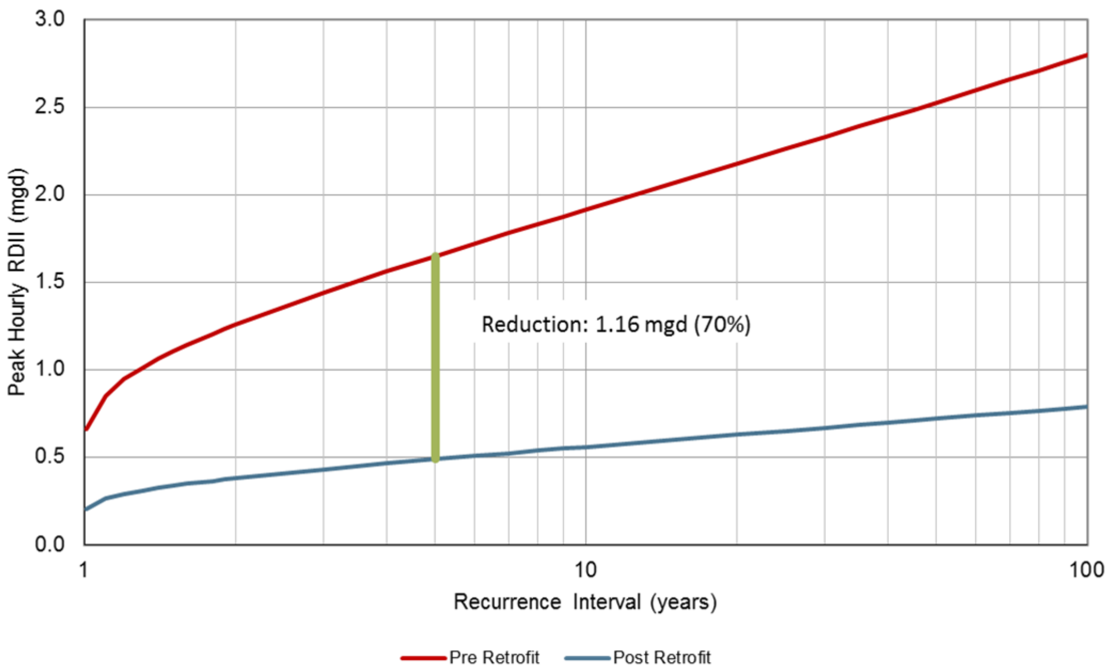


Figure 4-13. Phase 4 monitoring Basin 5 LPIII analysis

4.1.3.4 Monitoring Basin 9.1 (Sanitary Basin 10)

Monitoring Basin 9.1 monitored flows from Sanitary Basin 10. Phase 4 was the only phase to conduct work in this basin. Input from the City's engineering and maintenance staff indicated some localized issues along Long Street, and due to budget restrictions and lower predicted I/I removal rates, the scope of the work was limited to these areas. Approximately 4,200 LF or 20 percent of the sewers were addressed since the inception of the I/I Abatement Program.

Figure 4-14 shows the extent of work completed in the first four phases.

Phase 3 meter 19 was located in the same manhole as Phase 4 meter 9.1; therefore, no scaling was necessary to make the two time series comparable. The change in 5-year peak-hour RDII between the two models is negative, indicating the peak-hour RDII may have increased between phases. Closer inspection of the Phase 3 meter 19 flow data indicates that the meter may have had trouble accurately measuring peak flows, which in turn would make peak calibration difficult to achieve. The peaks in the blue observed time series shown in Figure 4-15 appear to be cropped at a relatively consistent value of around 0.55 to 0.6 mgd. These cropped peaks may indicate that the meter likely was unable to measure flow values greater than the 0.55- to 0.6-mgd threshold. The model calibrated to these flow data is likely underrepresenting peak flows, which would underestimate the post-Phase 3, 5-year peak-hour RDII. In other words, the pre-Phase 4 model likely underreported peak-hour flows due to underreported flows from the pre-Phase 4 monitoring effort.

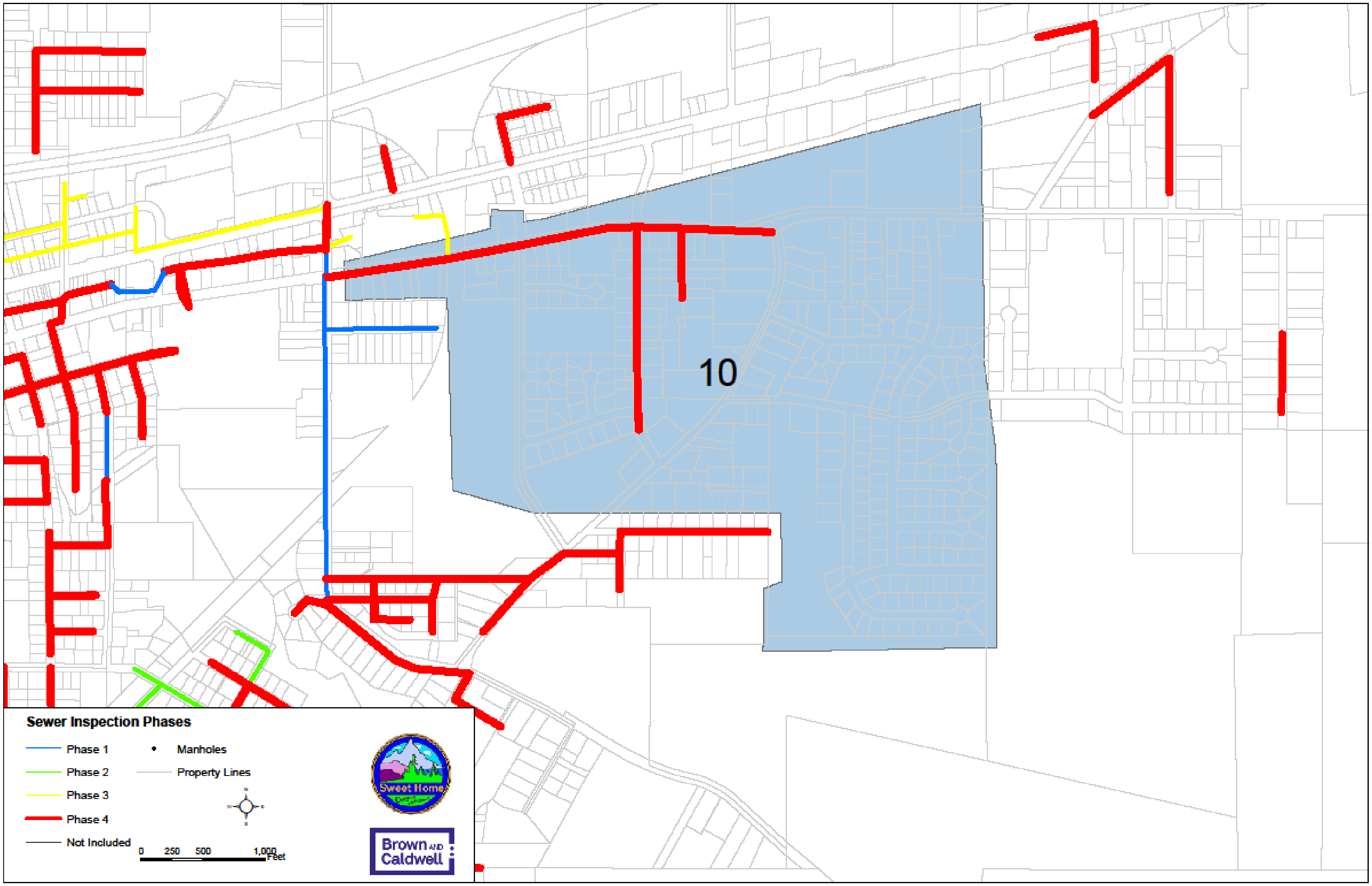


Figure 4-14. Extent of R&R work in Sanitary Basin 10

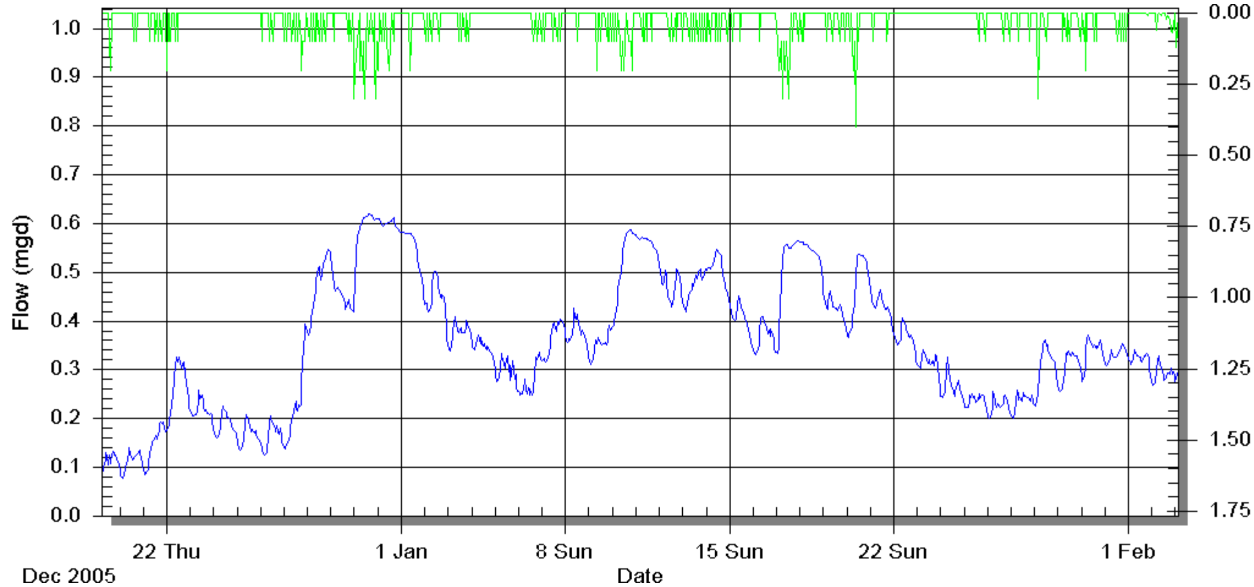


Figure 4-15. Cropped peaks in pre-Phase 3 flow monitoring data

Figure 4-16 presents the LPIII curves for both phases.

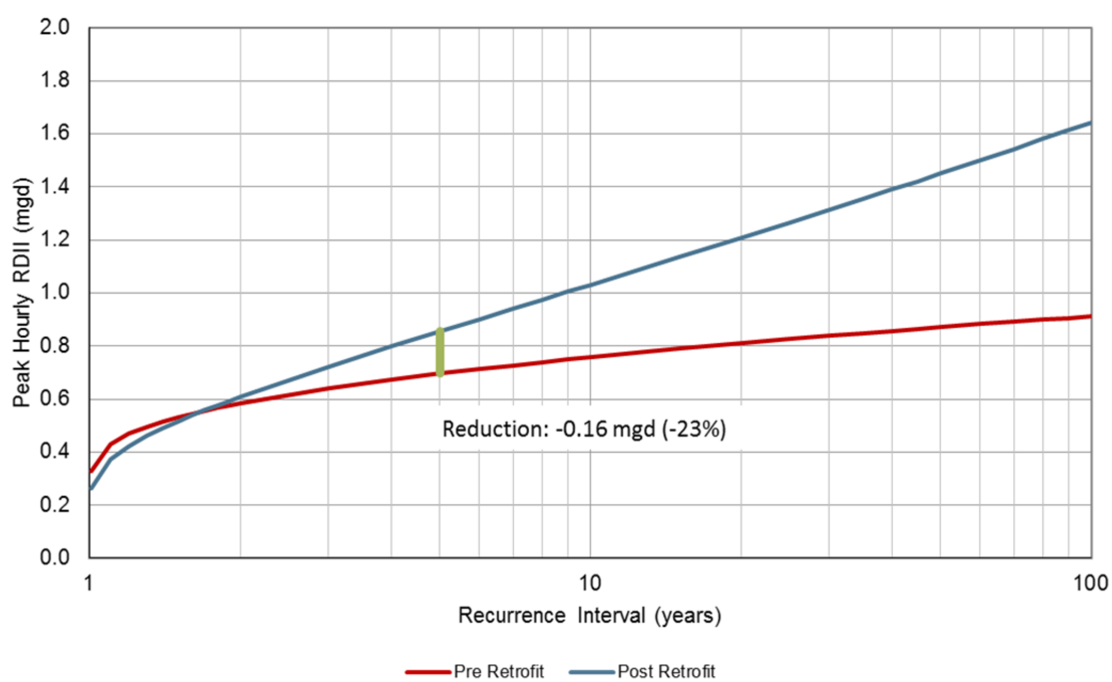


Figure 4-16. Phase 4 Monitoring Basin 9.1 LPIII analysis

4.1.3.5 Monitoring Basin 9.2 (Sanitary Basin 9)

Monitoring Basin 9.2 monitored flows from Sanitary Basin 9. Approximately 7,600 LF or 62 percent of the sewers have been addressed in this basin since the inception of the I/I Abatement Program. Figure 4-17 shows the extent of work completed in the first four phases.

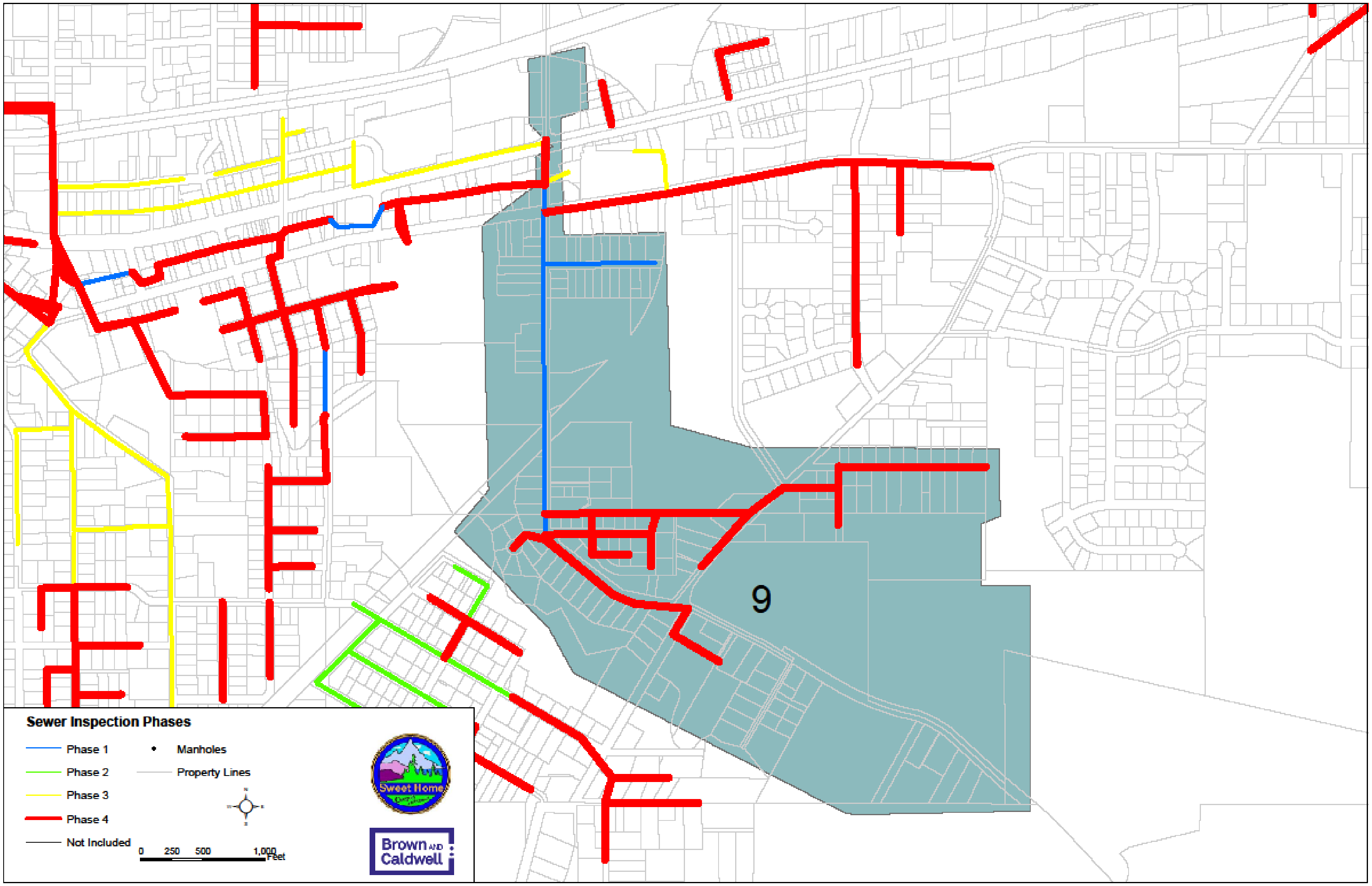


Figure 4-17. Extent of R&R work in Sanitary Basin 9

Phase 3 meters 17 and 18 monitored slightly more pipes than Phase 4 meter 9.2. Phase 3 meter 17, which is the downstream of the two meters, was prone to surcharge conditions during Phase 3 monitoring. To prevent the monitoring issues surrounding surcharge conditions, the meter was moved upstream to the Phase 4 meter 9.2 location. To account for this loss of monitored pipe, a factor of 0.85 was applied to the sum of the RDII time series of the Phase 3 basin 17 and 18 models to be comparable with the RDII time series of the Phase 4 Basin 9.2 model. Figure 4-18 shows the reduction of peak-hour RDII flows since the completion of Phase 3 in this basin. Overall, the reductions are consistent with what is expected in a basin where a portion of the mains and manholes have been rehabilitated but the laterals have been partially addressed.

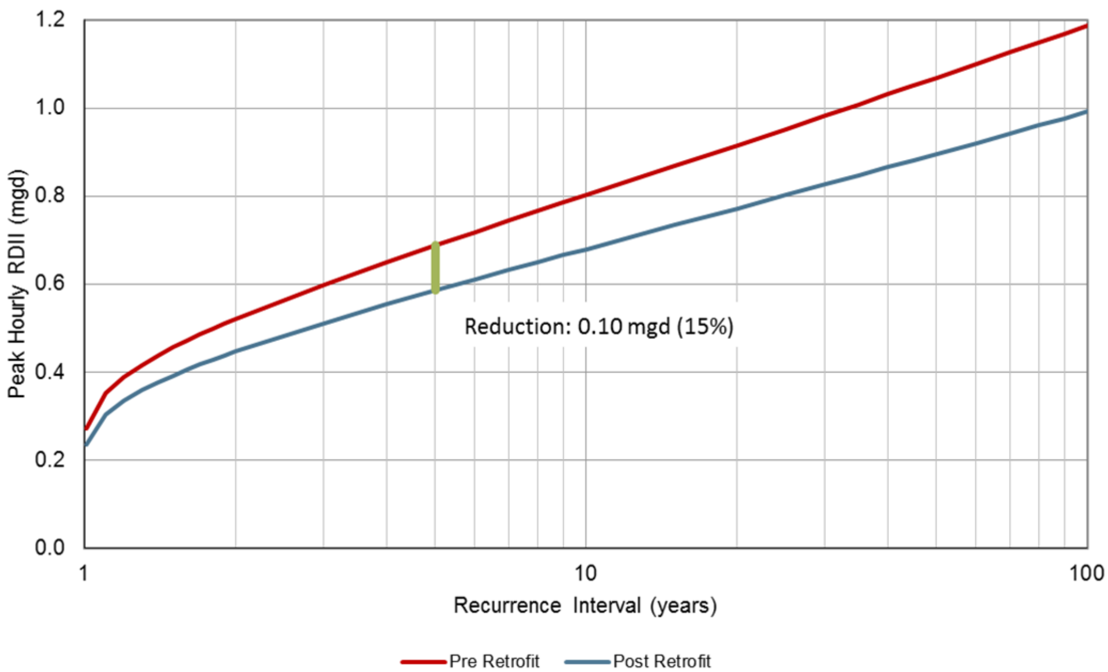


Figure 4-18. Phase 4 Monitoring Basin 9.2 LPIII analysis

4.1.3.6 Monitoring Basin 12 (Sanitary Basin 20)

Monitoring Basin 12 monitored flows from Sanitary Basin 20. Approximately 11,800 LF or 77 percent of the sewers have been addressed in this basin since the inception of the I/I Abatement Program, all in Phase 4. As discussed previously, Phase 4 funding constraints limited the amount of lateral work that could be done on private property, so laterals have not been addressed to the extent recommended for maximum I/I reduction. Figure 4-19 shows the extent of work completed in this basin.

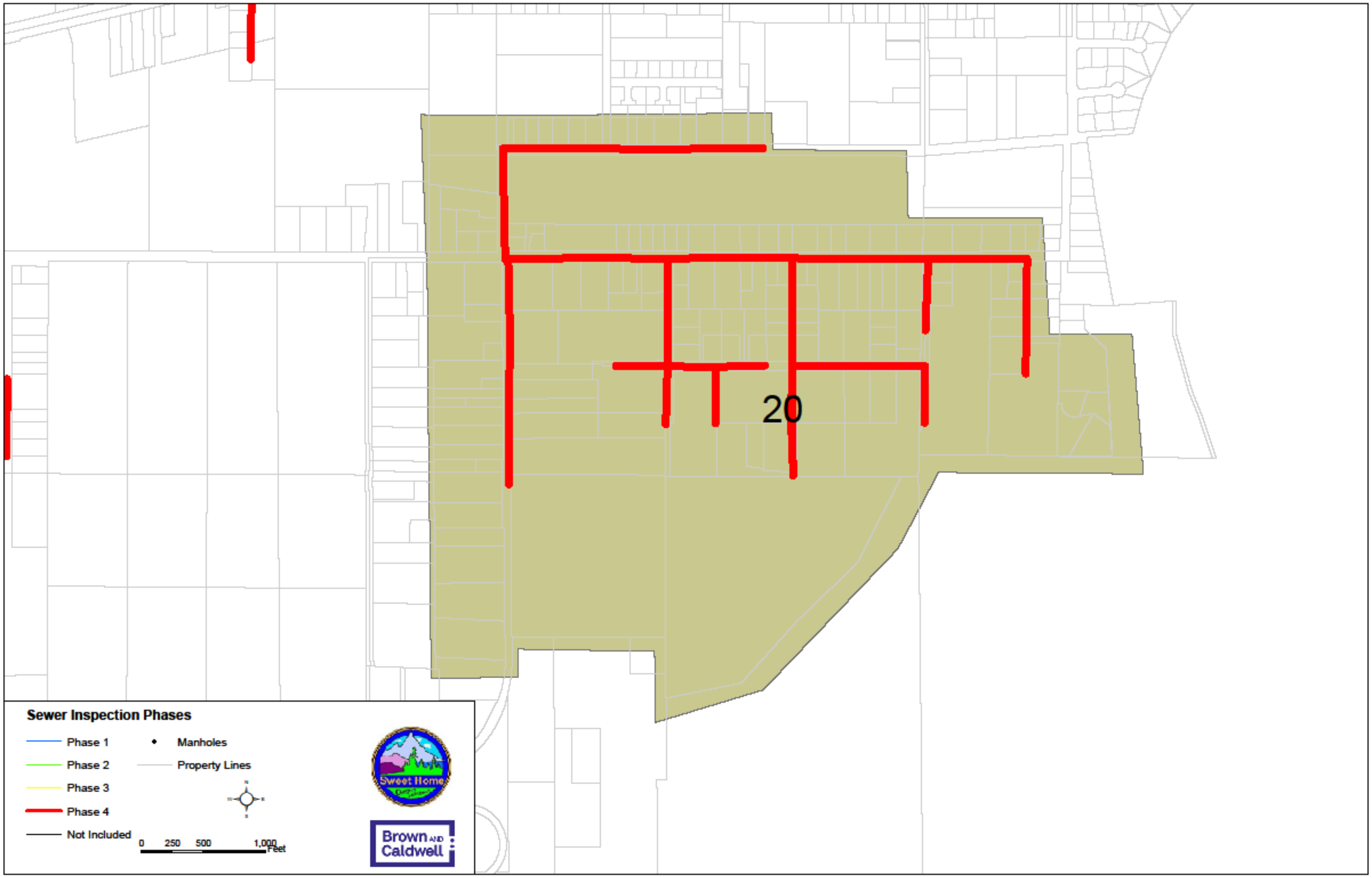


Figure 4-19. Extent of R&R work in Sanitary Basin 20

Phase 3 meter 23 was located in the same manhole as Phase 4 meter 12, allowing for a direct comparison of the RDII time series from each basin's calibrated models. The change in 5-year peak-hour RDII between the two phases is 0.16 mgd, representing a 35 percent change in RDII, as shown in Figure 4-20. Inspection of the LPIII curves indicates that the rehabilitation was most effective between the 2-year and 7-year return periods as the difference between the two curves is the greatest. Above approximately the 7-year return period, the effectiveness is reduced as the post-retrofit curve climbs steeply toward the pre-retrofit curve. This indicates that for low-frequency high-magnitude storms, the rehabilitation has less of an impact on RDII removal. For the more frequent low-magnitude storms, the rehabilitation appears to be more effective. One possible reason is that the unaddressed laterals at slightly higher elevation are contributing more RDII during those high-magnitude storms when the ground is extremely saturated and the groundwater table is temporarily elevated. Overall, the reductions are consistent with what is expected for a basin where mains and manholes have been rehabilitated but the laterals have been partially addressed.

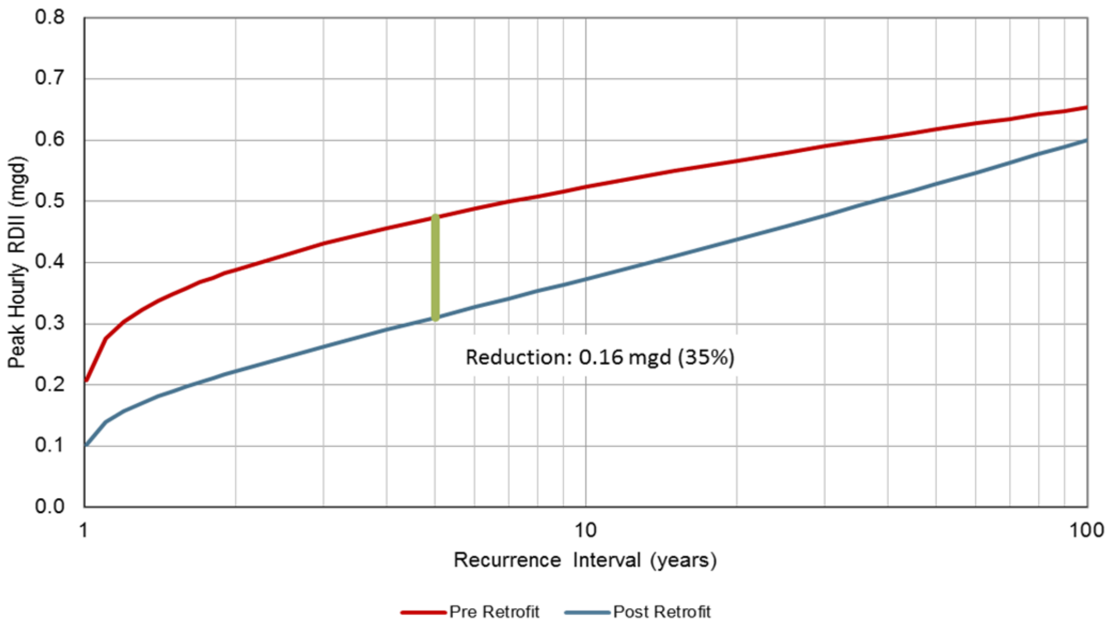


Figure 4-20. Phase 4 Monitoring Basin 12 LPIII analysis

4.1.3.7 Summary and Conclusions

Table 4-7 summarizes the RDII removal for the Phase 4 monitoring basins.

Table 4-7. Post Phase 4 5-Year Peak Hourly RDII				
Monitoring basin	Post-Phase 3 5-year peak-hour RDII, mgd	Post Phase 4 5-year peak-hour RDII, mgd	RDII removed, mgd	Percent reduction
1A (SFE)	1.15	0.37	0.78	68
5 (Gleaners)	1.65	0.49	1.16	70
9.1 (Admin)	0.70	0.86	-0.16	-23
9.2 (Auto Shop)	0.69	0.59	0.10	15
12 (Church)	0.47	0.31	0.16	35
Total	4.66	2.61	2.05	44

The total RDII removed during Phase 4 is at least 2.05 mgd in these five basins, representing a 44 percent reduction in available RDII. Considering the modeling complications associated with the post-Phase 3 model of Monitoring Basin 9.1 and recognizing additional rehabilitation work in spot areas outside of the basins, the actual Phase 4 RDII removal is likely higher than 2.05 mgd.

4.1.4 Future Conditions

The following subsections describe the methods used to model future sanitary system hydrologic conditions for Sweet Home.

4.1.4.1 Future Service Areas and Population

Sweet Home is divided into 27 sanitary basins that do not necessarily share the same borders as the flow monitoring basins. Currently, 19 of the sanitary basins have residents living within their boundaries. The remaining eight basins are not yet developed.

Figure 4-21 shows the existing and future sewer service areas.

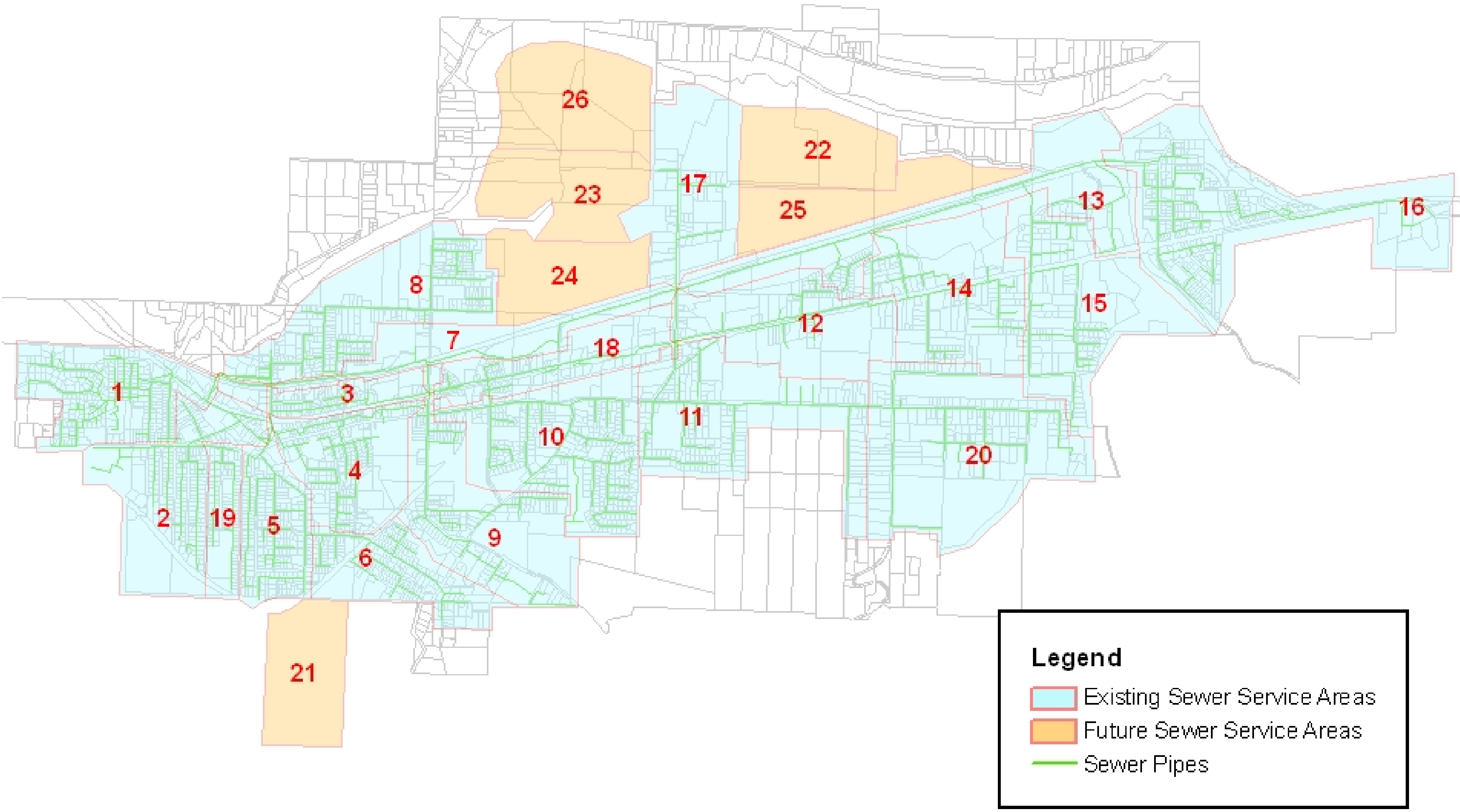


Figure 4-21. Existing and future service areas

Service area and population expansion data for the year 2025 as developed in the Sweet Home Wastewater Facility Plan were used to project wastewater and RDII loads in Sweet Home. The data provided are listed in Table 4-8.

Table 4-8. Future Service Areas and Populations

Sanitary basin	Existing service areas		Additional future (2025) service areas	
	Area, acres	Population	Area, acres	Population
1	118	687	0	0
2	108	690	33	162
3	36	258	0	0
4	103	637	0	0
5	78	605	0	0
6	77	505	13	81
7	94	475	0	0
8	159	778	0	0
9	124	444	33	54
10	166	858	0	54
11	100	673	75	81
12	82	259	49	216
13	115	177	42	81
14	104	390	49	189
15	111	400	56	162
16	230	430	16	135
17	0	0	74	81
18	65	227	12	81
19	40	272	0	0
20	170	356	65	405
21	0	0	85	637
22	0	0	94	448
23	0	0	105	1,215
24	0	0	97	0
25	0	0	108	810
26	0	0	122	270
27	0	0	100	1,350
Total	2,080	9,121	1,228	6,512

4.1.4.2 Future DWF

By 2025, an additional 6,512 people are predicted to live in Sweet Home. Current demand patterns can provide an estimate of what the demand patterns of the future may be. To estimate the wastewater demand of the future population, the current DWF of 1.22 mgd (see Section 4.1.2.1) was divided by the current population to give a per capita wastewater demand. This per capita demand was multiplied by projected future additional populations to estimate the future additional wastewater demand of each sanitary basin.

To estimate the DWF pattern of the future areas, a representative DWF pattern was created by averaging the DWF patterns of all 15 monitoring basins created during hydrologic modeling. This average DWF pattern, as shown in Figure 4-22 is the best guess of what the DWF pattern of any future population in Sweet Home may look like.

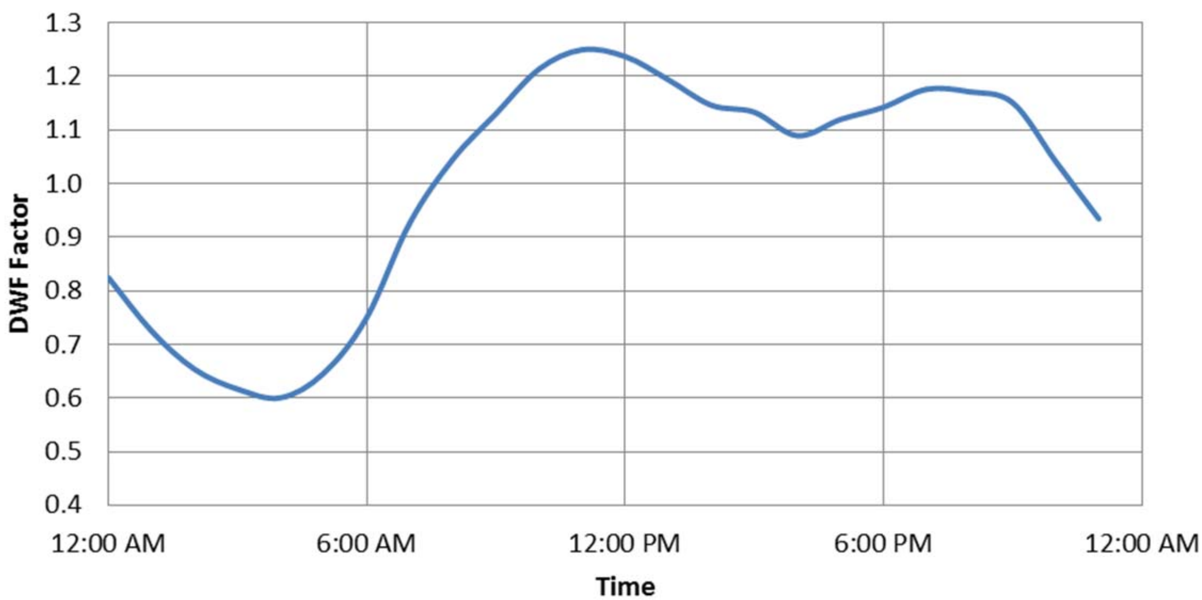


Figure 4-22. Sweet Home average DWF pattern

4.1.4.3 Future WWF

WWF projections were also performed on a sanitary basin basis. Projections for wet weather flow are based on basin size instead of basin population. For each sanitary basin, a peak RDII was calculated using an assumption of 1,500 gpad (Earth Tech Team, 2005), which is lower than the 2,000 gpad maximum allowable groundwater infiltration rate dictated by OAR 340 Division 52. To load these peak flows into the model, a similar approach to that which was taken in DWF projections was applied to WWFs. A characteristic RDII curve was created by taking the area weighted average (monitoring basin area) of the January 1976 event from the 15 calibrated model RDII time series. This average time series was scaled such that the peak-hour RDII would be equal to 1 mgd. When loaded into the hydraulic model, a factor could be applied to scale this characteristic RDII curve which in turn will produce the desired peak-hour RDII for a given sanitary basin. For example, Sanitary Basin 2 will have 33 additional acres of area in the future. Using the 1,500 gpad assumption, the projected future peak-hour RDII for this basin is 0.0495 mgd. Using a factor of 0.0495 on the characteristic RDII curve will produce a peak of 0.0495 mgd in the hydraulic model from this sanitary basin. .

The characteristic RDII time series is shown in Figure 4-23. Note that the peak RDII is 1 mgd.

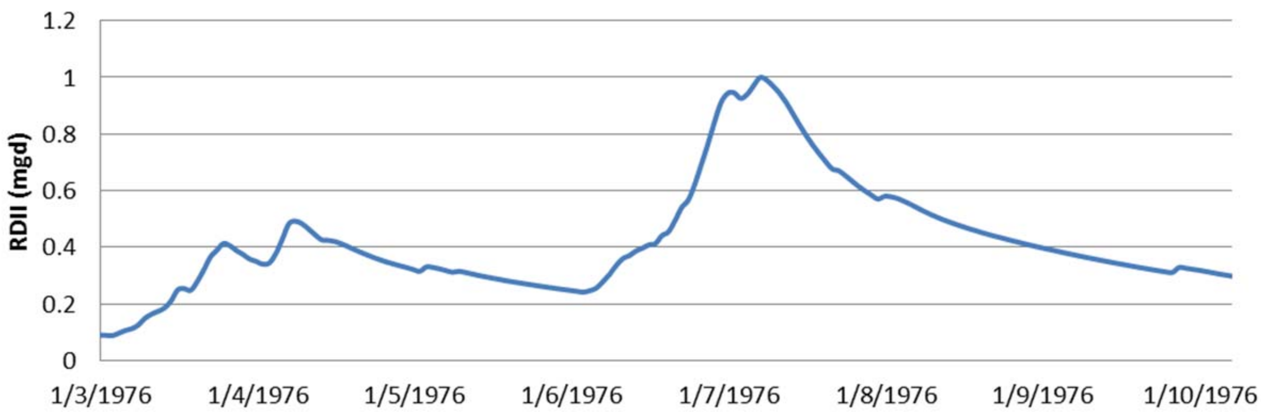


Figure 4-23. Characteristic RDII time series

4.1.4.4 Summary of Future Flow Inputs

Table 4-9 summarizes the future flow projections. These flows will be loaded into the hydraulic model based on their respective sanitary basins' outlet. For future sanitary basins not currently serviced by a sewer line, the closest existing node was chosen as the loading point.

Table 4-9. Future Service Areas and Populations

Sanitary basin	Additional DWF, mgd	Additional RDII, mgd	Hydraulic model loading node
1	0.000	0.000	1-4
2	0.022	0.050	2-4
3	0.000	0.000	3-3
4	0.000	0.000	4-1
5	0.000	0.000	5-3
6	0.011	0.020	6-1
7	0.000	0.000	7-1
8	0.000	0.000	8-3
9	0.007	0.050	9-7
10	0.007	0.000	10-1
11	0.011	0.113	11-2
12	0.0297	0.074	12-1
13	0.011	0.063	13-2
14	0.025	0.074	13-2
15	0.022	0.084	13-2
16	0.018	0.024	13-2
17	0.011	0.111	7-29
18	0.011	0.018	18-1

Table 4-9. Future Service Areas and Populations			
Sanitary basin	Additional DWF, mgd	Additional RDII, mgd	Hydraulic model loading node
19	0.000	0.000	19-1
20	0.054	0.098	20-2
21	0.085	0.128	5-14
22	0.060	0.141	13-2
23	0.162	0.158	7-23
24	0.000	0.146	7-23
25	0.108	0.162	13-2
26	0.036	0.183	7-23
27	0.18	0.150	13-2
Total	0.870	1.842	n/a

By the year 2025, an additional 0.87 mgd of DWF is projected year-round. An additional 1.84 mgd of peak-hour RDII is projected during the 5-year storm.

4.1.4.5 Hydraulic Modeling Results Summary

Table 4-10 provides a historical look at the 5-year peak-hour existing and future flow rates to the WWTP through the four phases of rehabilitation.

Table 4-10. Hydraulic Modeling Results			
Model phase	Existing 5-year peak-hour flow, mgd	Peaking factor	Future 5-year peak-hour flow, mgd
Pre-Phases 1 and 2	22.0	22	25.1[a]
Post-Phases 1 and 2	15.3	15	17.9[b]
Post-Phase 3	13.6	14	15.4[b]
Post-Phase 4	11.5	12	13.3[b]

[a]*Based on future population (2027) of 10,525 with no expansion of the City's wastewater service area.*

[b]*Based on future population (2025) of 15,633 with expansion of the City's wastewater service area.*

Figure 4-24 shows these predicted peak-hour flows after each modeling effort in graphical format.

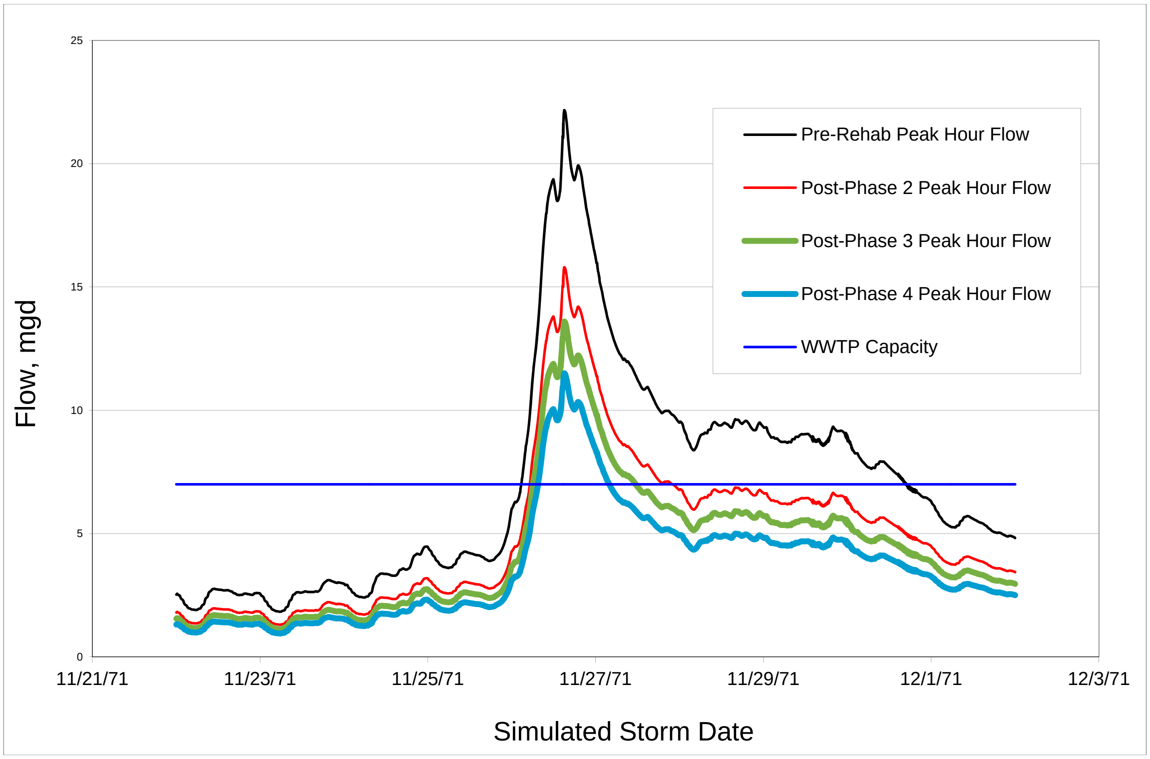


Figure 4-24. Predicted 1-in-5 peak-hour flow

Section 5

Capacity Evaluation

A hydraulic model was developed to determine the collection system's response to peak flows under the 5-year wet-weather event. The hydraulic model platform chosen was MIKE URBAN, a product of DHI, Inc. When the hydraulic model was originally developed, only the major trunk lines were included below the upper most monitoring basin outlets (meter locations). The purpose of the hydraulic model is to assess areas of capacity limitations within the collection system as well as prediction of the peak flow to the Sweet Home WWTP. This section describes hydraulic model modifications, flow loading, and results.

5.1.1 Model Modifications

As-built surveys created during Phase 4 rehabilitation guided the updating of the hydraulic model. In some locations, invert elevations, rim elevations, and pipe diameters needed to be updated. Hydraulic retrofits constructed during the Phase 4 rehabilitation needed to be reflected in the hydraulic model to reflect changes to the flow paths in the collection system accurately.

Three of the Phase 4 flow meters (meters 1A, 4, and 6) in the southwest portion of the city were placed farther down the collection system than their Phase 3 counterparts. This provides a more coarse view of the flows coming from these basins. Because the hydrologic models were built to reflect the flows at points farther downstream (where the existing conditions flows are loaded), the pipes upstream of these meters are not to be analyzed for surcharging because they do not see any flow in the existing condition model. Although the pipes remained in the model, their results are not presented here because they do not provide any useful information. Figure 5-1 shows the extents of the hydraulic model with reportable results.

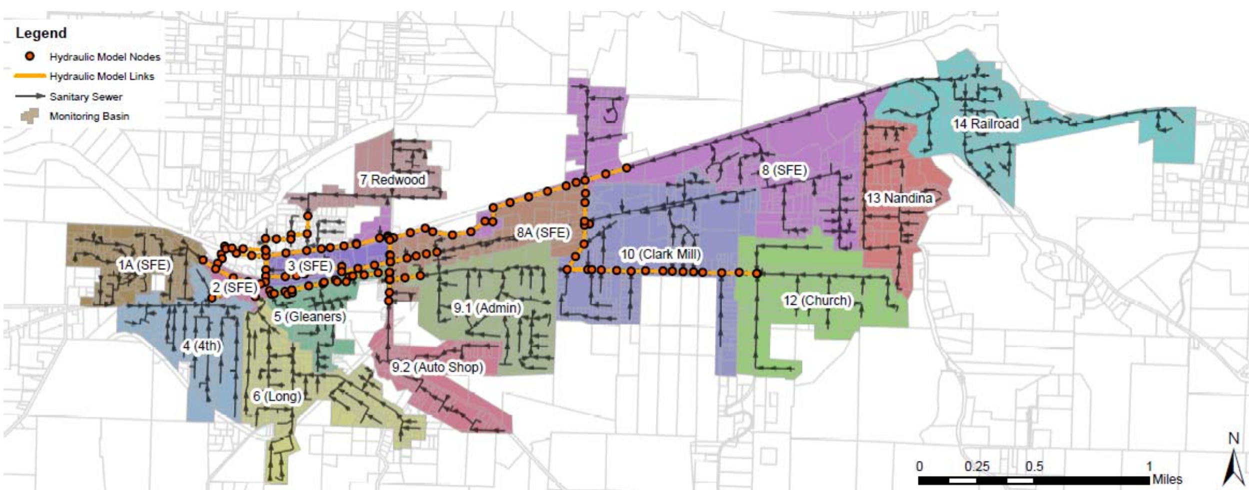


Figure 5-1. Hydraulic modeling network

Phase 4 flow meters 13 and 14 were placed in the far northeast corner of the city. These two meters break up Phase 3 Monitoring Basin 21 into smaller portions, giving a closer look at the contributions of flow from this part of the collection system. An attempt was made to extend the hydraulic model trunk line to this part of the city along the railroad tracks. The geographic information system data available to build this extension of the model were in error and prevented the extension from being built. Because of this, flows from Monitoring Basins 13 and 14 were loaded at the most northeastern node of the hydraulic model.

The meter for Phase 4 monitoring Basin 7 was located four manholes upstream of its Phase 3 counterpart. To account for the increased travel distance, the hydraulic model was updated to include these four additional links and nodes.

5.1.2 5 Year Recurrence Event Selection

An additional purpose of long term simulations is to isolate a 5-year storm to be routed through the hydraulic model. To identify this storm, the total flow timeseries of all 15 basin models were summed together, the annual maxima series of this summed data was extracted, and that series was fit to an LPIII distribution. The sum of the individual total flow timeseries is a quick way to simulate the total flow to the WWTP without having to route all 15 timeseries through the hydraulic model. The limitation of this method is that it does not take into account routing delays associated with flow conveyance from different points in the collection system. However, at an hourly timestep, these conveyance delays have a minimal effect on the analysis.

The LPIII curve provides an approximate 5-year peak-hourly total flow rate to the WWTP which can be used to select a storm in the long term simulation record that comes close to matching that peak value. The January 1976 storm used in previous (i.e. Phase 3) analyses for this project still ranks at nearly a 5-year peak-hourly flow recurrence. Therefore, this storm was chosen again for routing through the hydraulic model. Because the sum of the 15 individual total flow timeseries does not account for unmetered areas near the WWTP, the estimated 5-Year peak hourly total flow is considered an underestimate of the actual 5-year peak hourly total flow and is therefore not reported in this section. This underestimation is not a concern in selecting a 5-year storm since accounting for unmonitored area would not alter the ranking of storms against each other, which in turn would not affect the determination of a 5-year storm. This analysis is primarily intended to isolate a storm with a shape characteristic of a 5-year storm as the storm hydrograph will be scaled on a monitoring basin by basin basis. A discussion of this scaling can be found in Section 5.1.3.2.

5.1.3 Flow Loading

The following subsections describe the loading of flows into the hydraulic model.

5.1.3.1 Dry Weather Flow (DWF)

In all cases, loading DWF is done by giving the hydraulic model an average DWF magnitude and an associated diurnal pattern by which to scale the average values over time. For the 15 existing conditions monitoring basin models, 15 different DWF patterns were entered in the model and were associated with their respective average flow magnitudes (See Section 4.1.4.2) to provide 1.16 million gallons per day (mgd) to the WWTP. For consistency, these existing conditions DWFs were loaded into the hydraulic model at the same nodes that the flow meters used to calibrate the DWFs was placed within. For monitoring basins 2, 3, and 7, scaling factors were applied to the average DWF value to account for unmonitored downstream areas.

Future DWFs were loaded in much the same way except only one diurnal pattern was used for all additional DWF loads. Future DWFs were loaded with respect to sanitary basin loading points. These locations can be found in Section 4.

5.1.3.2 Wet Weather Flow (WWF)

Depending on the characteristics of the rainfall for a given storm, different subbasins basins will react in different ways due to factors such as soil condition, pipe condition, land surface conditions, etc. To illustrate this, consider two identically sized and sloped subbasins where subbasin A is entirely impervious and subbasin B has no impervious area. Subbasin A will be most sensitive to rainfall intensities as impervious surfaces wash off rainfall nearly immediately and have little initial abstractions to fill. Subbasin B will be more sensitive to total rainfall volume and duration as initial abstractions will need to be filled and the effects of subsurface interflow and groundwater buildup can add additional peak discharge later in high volume storms. The sum of the discharges from the subbasins will rank at some recurrence interval for each storm. However, since the two subbasins have dissimilar hydrologies, the recurrence interval for the sum of the two discharges will not be necessarily indicative of the recurrence interval of the flows from the individual subbasins. This is the same situation as can be found in Sweet Home. A rainfall event that produces a 5-year peak flow to the WWTP does not guarantee that all upstream subbasins are discharging at their individual 5-year peak flows due to dissimilar hydrologic conditions throughout the city.

To accurately represent the hydraulic performance of the collection system to 5-year peak flows, the modeled RDII timeseries loaded into the hydraulic model for the January 1976 storm were scaled to statistical 5-year peak hourly values for each of the monitoring basins. This scaling exercise prevents some pipes from having to pass 25-year flows while others only need to pass 2-year flows (and in turn, this prevents the mislabeling of undersized pipes). This method allows each basin to flow at a 5-year recurrence and therefore provide representative information about capacity restrictions in the collection system during statistical 5-year frequency conditions. Table 5-1 provides the scaling factors used on the existing conditions RDII as well as calculated composite factors which take into account adjustments necessary for unmonitored pipe lengths.

Table 5-1. Existing Conditions RDII Factors

Monitoring Basin	January 1976 peak-hour RDII, mgd	5-year peak-hourly RDII, mgd	RDII factor	Unmonitored pipe length factor	Composite factor	Calculated peak-hour RDII, mgd
1A (SFE)	0.367	0.368	1.004	n/a	1.004	0.370
2 (SFE)	0.061	0.054	0.883	1.144	1.010	0.054
3 (SFE)	0.737	0.658	0.893	1.088	0.971	0.639
4 (4th)	0.465	0.522	1.123	n/a	1.123	0.586
5 (Gleaners)	0.556	0.490	0.882	n/a	0.882	0.432
6 (Long)	1.303	1.319	1.012	n/a	1.012	1.336
7 (Redwood)	0.405	0.348	0.860	1.610	1.385	0.482
8A (SFE)	0.590	0.579	0.982	n/a	0.982	0.569
8 (SFE)	1.732	2.073	1.196	n/a	1.196	2.480
9.1 (Admin)	0.959	0.856	0.893	n/a	0.893	0.764
9.2 (Auto Shop)	0.661	0.585	0.886	n/a	0.886	0.519
10 (Clark Mill)	0.926	0.839	0.907	n/a	0.907	0.761
12 (Church)	0.318	0.310	0.975	n/a	0.975	0.302
13 (Nandina)	0.492	0.486	0.989	n/a	0.989	0.481
14 (Railroad)	1.110	1.094	0.985	n/a	0.985	1.078

Future conditions wet weather flows were loaded into the model with respect to sanitary basin outlets and were routed through the hydraulic network along with the existing conditions dry and wet weather flows. For a discussion of future condition hydrology, see Section 5.1.4.2.

5.1.4 Hydraulic Model Results

The hydraulic modeling effort reveals a number of locations where the collection system either surcharges or overflows. The following sections present the results of four modeling scenarios to assess flooding nodes and hydraulic capacity limitations in both existing and future conditions. Figure 5-2 below shows the results of the hydraulic model results as it relates to manholes; red manholes indicate locations of projected overflows, yellow manholes indicate locations of surcharging from 0 to 3 feet below grade, and green manholes indicate either no surcharging or surcharging less than 3 feet below grade.

5.1.4.1 Existing Conditions

The locations of pipe with the highest surcharge potential result from Sanitary Basins 9 and 10. Due to flow data complications in Phase 3, the projected 5-year peak-hour flow rate from Sanitary Basin 10 (Monitoring Basin 9.1) is higher in the Phase 4 analysis than calculated in Phase 3. This results in the only flooding manhole predicted in the existing condition model. A few manholes at the west ends of Sanitary Basins 7 and 8 show high surcharge as well.

Figure 5-2. Hydraulic modeling results, projected surcharge, and overflow locations under existing conditions

To determine which pipes are hydraulically restricted, the hydraulic model was rerun with sealed manholes that prevent the manholes from overflowing. This change projects where the 5-year event peak flow results in a hydraulic grade line higher than the rim of manholes and ultimately demonstrates which pipe segments are undersized. The pipes in red indicate capacity limitations that cause surcharging above 3 feet of freeboard in the upstream manhole, as shown in Figure 5-3. However, the undersized pipes are based on the criterion that surcharging with less than 3 feet of freeboard is unacceptable during the 1-in-5 peak-hour flow; applying a less conservative criterion would result in fewer undersized segments.

Figure 5-3. Hydraulic modeling results and undersized pipes under existing conditions

5.1.4.2 Future Conditions

Future population and service area expansion adds additional DWF as well as projected RDII. This adds flow to a system that already is hydraulically restricted in some areas. Flooding risks appear to be elevated in a future flow scenario as three manholes near the WWTP along the railroad tracks are predicted to overflow as shown in Figure 5-4.

Figure 5-4. Hydraulic modeling results, projected surcharge, and overflow locations under future conditions

The hydraulic model was rerun with sealed manholes under future growth conditions to show where pipes are undersized. These results are shown in Figure 5-5. Only two additional links are determined to be capacity-limited between the existing and future conditions. Again, the undersized pipes are based on the criterion that surcharging with less than 3-feet of freeboard is unacceptable during the 1-in-5 peak-hour flow; applying a less conservative criterion would result in fewer undersized segments.

Figure 5-5. Hydraulic modeling results and undersized pipes under future conditions

Section 6

Condition Assessment

In 2005, the City conducted a closed-circuit television (CCTV) inspection and condition assessment on the entire publicly-owned collection system. The assessment was intended to supplement the flow metering and modeling data to increase the effectiveness of the Phases 3 and 4 Infiltration/Inflow (I/I) Abatement Projects. Approximately 221,000 linear feet (LF) or 88 percent of the sewer system was inspected, as shown in Figure 6-1. The project provided the City with baseline digital inspections, updated inspection software capable of digital inspections and consistent with nationally accepted condition assessment protocols, and condition assessment certification for the City's inspectors. The City adopted the standards developed by the National Association of Sewer Service Companies (NASSCO) for the sewer defect identification and defect rating system. The ratings are according to the NASSCO Pipeline Assessment Certification Program grading system.

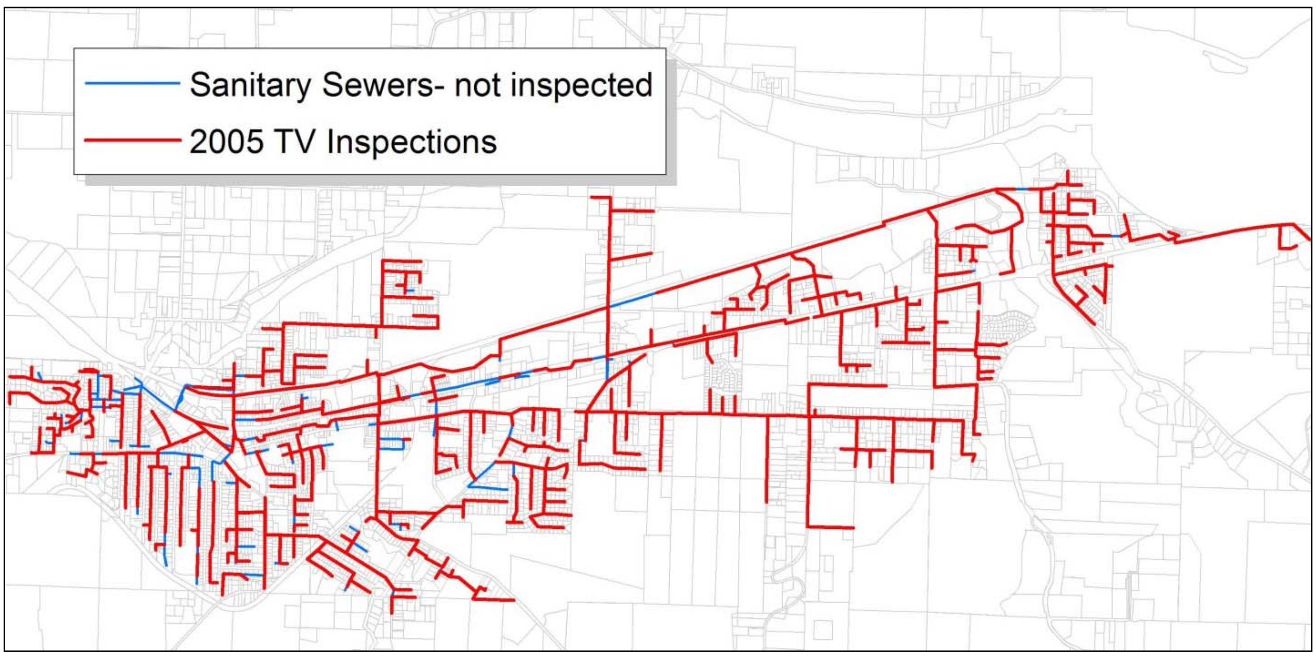


Figure 6-1. Extent of 2005 CCTV inspections

The 2005 results revealed the majority of the public sewer mains to be in fair to good condition with no apparent high risk structural (e.g., broken pipe) or operational (e.g., debris, roots) defects. Approximately 9 percent of pipe had structural defects requiring immediate attention (i.e. holes) and approximately 16 percent of pipe had defects that were recommended for monitoring that should be addressed in the next 5 to 10 years. A summary of the results is listed in Table 6-1.

Table 6-1. Summary of 2005 CCTV Inspections						
Condition grade	Structural		Operational		Structural and operational	
	LF	Percent of total inspections	LF	Percent of total inspections	LF	Percent of total inspections
5 (Failed)	14,714	6.7	7,883	3.6	20,411	9.3
4 (Poor)	27,729	12.6	13,738	6.2	35,733	16.2
3 (Fair)	51,170	23.2	16,443	7.5	47,653	21.6
2 (Good)	127,043	57.6	182,592	82.7	116,859	53.0
1 (Excellent)	0	0[a]	0	0[a]	0	0[a]

[a]A structural or operational grade of 1 was reserved for new sewers only.

6.1.1 Current Conditions

Since the 2005 inspections, approximately 69,000 LF of sewers have been rehabilitated as part of the Phases 3 and 4 I/I Abatement Projects. The main rehabilitation technologies included some cured-in-place pipe (CIPP and open-cut construction with a majority of pipe being rehabilitated using pipe bursting. Sewers that were addressed during Phases 3 and 4 had a decrease in the sewer structural/operational rating.

The condition improvement was based on the technology used for rehabilitation as listed in Table 6-2.

Table 6-2. Condition Grade Based on Rehabilitation Technology	
Rehabilitation technology	Condition grade
CIPP	2
Pipe bursting	1
Open-cut replacement	1

The 2005 inspections were conducted during the seasonally dry summer months when operational defects such as infiltration may not be visible. This should be considered when evaluating the operational condition of the sewers.

Figure 6-2 displays a map of the current structural ratings for the City's sewer system. Figure 6-3 displays a map of the current operational ratings for the City's sewer system.

Brown AND Caldwell

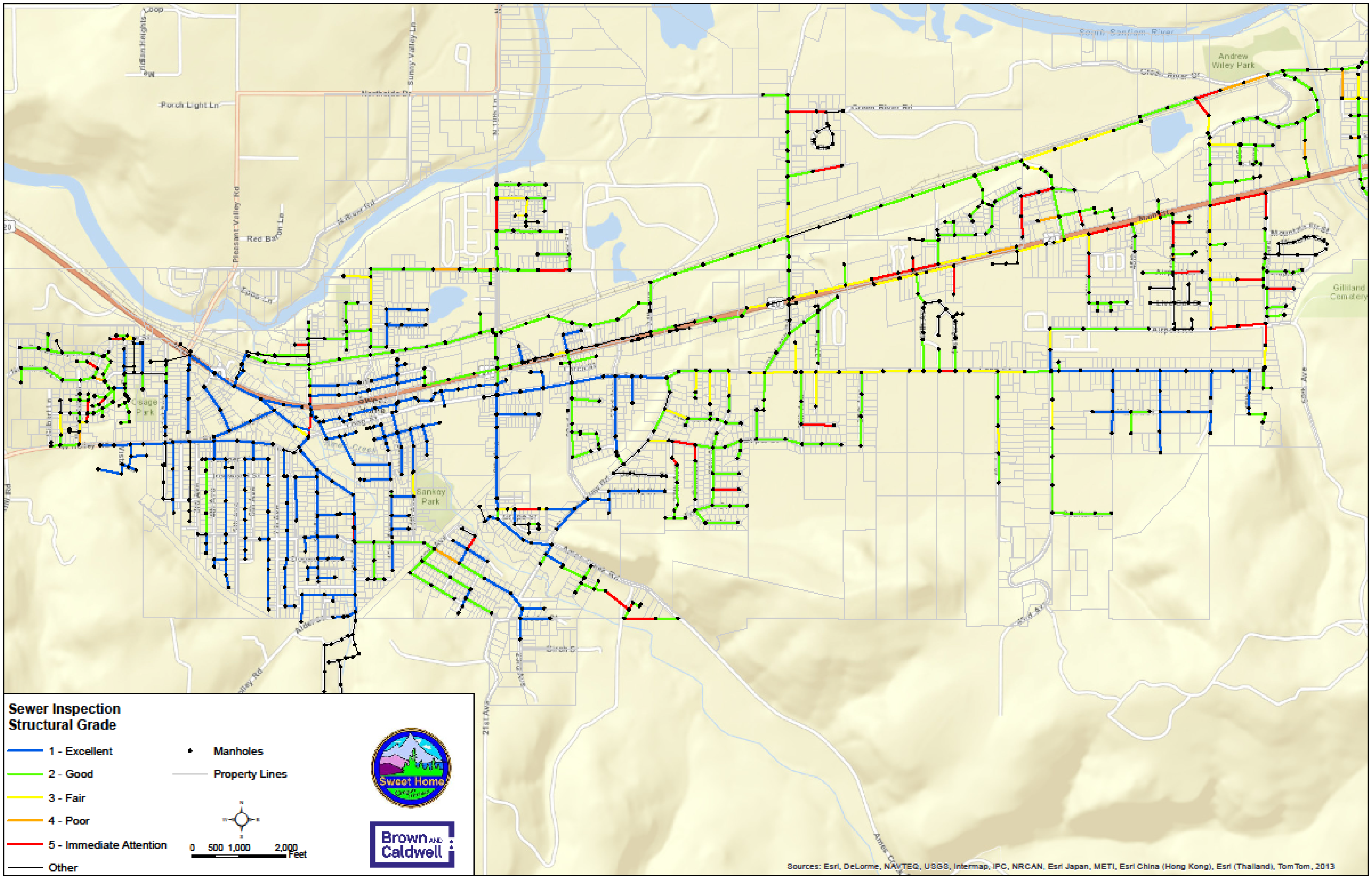


Figure 6-2. Post-Phase 4 Sewer Structural Condition Ratings

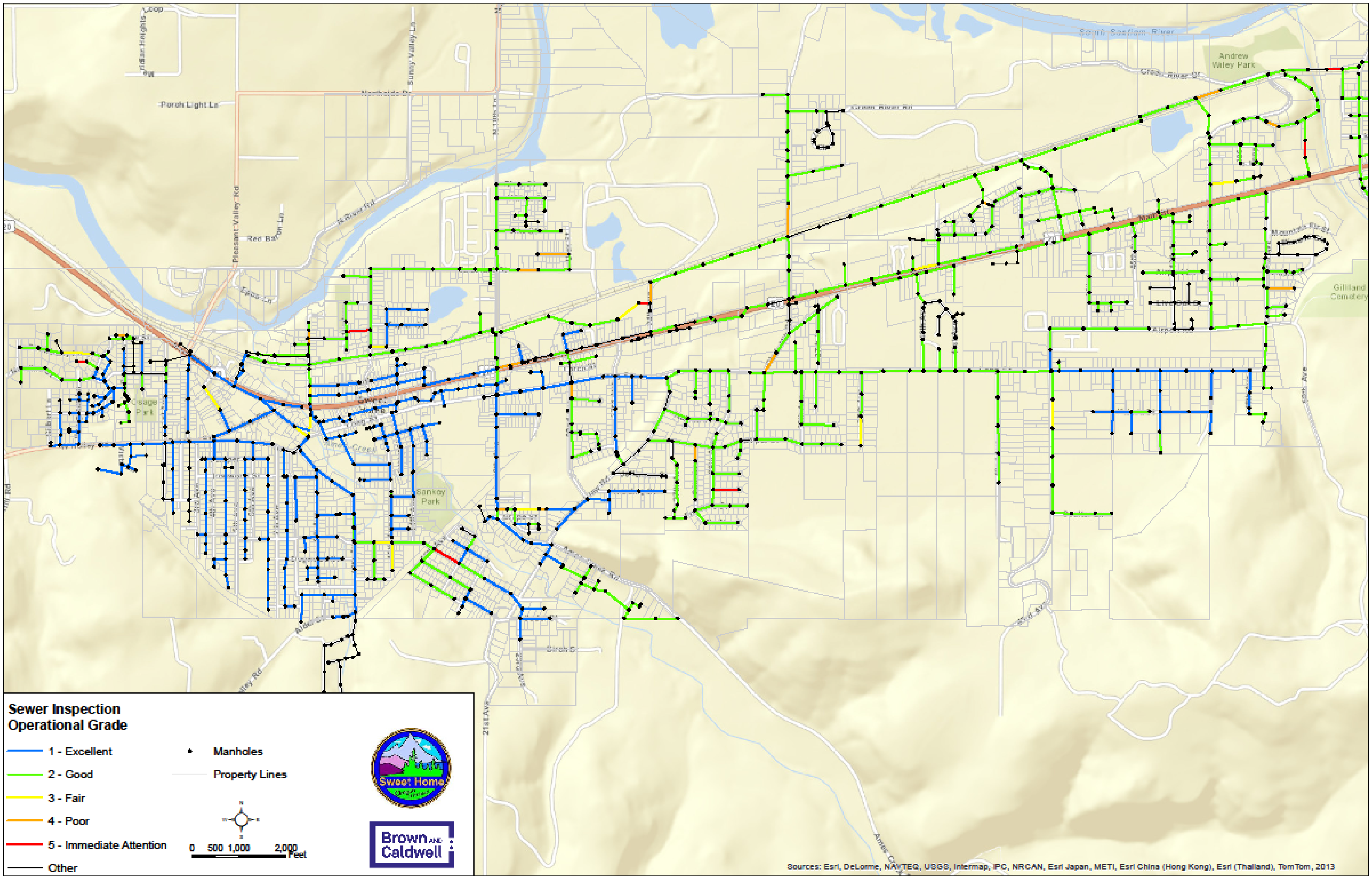


Figure 6-3. Post-Phase 4 Sewer Operational Condition Ratings

A summary of the condition grades following Phases 3 and 4 is listed in Table 6-3.

Table 6-3. Summary of Post-Phase 4 Condition Grades				
Condition grade	Structural		Operational	
	LF	Percent of total inspections	LF	Percent of total inspections
5 (Failed)	16,968	7.4	2,086	0.9
4 (Poor)	3,930	1.7	4,607	2.0
3 (Fair)	26,436	11.5	5,542	2.4
2 (Good)	109,184	47.3	137,059	59.3
1 (Excellent)	74,187	32.1[a]	81,806	35.4[1]

[a]*A structural or operational grade of 1 was reserved for brand new sewers only.*

The amount of Grade 5 pipe has increased since the 2005 CCTV inspections. This is due to a conservative estimate that approximately 15 percent of pipes rated as Grade 4 back in 2005 and not addressed as part the Phases 3 or 4 projects have worsened to Grade 5 in the past 8 years. In many cases, the structural rating is attributed to a point defect rather than the entire pipe segment.

The Grades 3, 4, and 5 pipes shown in Figures 6-1 and 6-2 are based on the 2005 inspections, since any pipe rehabilitated as part of Phases 3 and 4 would have a grade of 1 or 2 structurally or operationally. However, since the City currently inspects its collection system on a regular cycle, City staff should update the summary of the overall structural and operational ratings of individual pipes as annual inspections are completed to ensure the most recent information is on file. The City should consider cataloging and addressing any grade 5 point defects prior to the full segment failing as a result.

Section 7

Capacity, Management, Operation and Maintenance (CMOM)

In 2001, the U.S. Environmental Protection Agency (USEPA) proposed legislation to reduce the number and volume of sanitary sewer overflows (SSOs) significantly throughout the U.S. The USEPA determined that such actions were required to improve water quality. The proposed requirements would affect nearly all aspects of sanitary sewer management and operation. As proposed, each permit holder would be required to develop a CMOM program. The USEPA's promulgation of the CMOM requirements has stalled; however, elements of the proposed requirements have made their way into National Pollutant Discharge Elimination System (NPDES) permits and compliance is considered when evaluating permit violations and fees associated with SSOs. Increasingly, cities in Oregon are implementing CMOM elements in anticipation of it becoming a regulatory minimum. An overview of the elements of a CMOM program are discussed below.

7.1 CMOM Program

CMOM activities are primarily a best management practice approach to controlling SSOs. CMOM programs generally are comprised of the eight primary elements described in Table 7-1. When implemented, each permit holder's CMOM program improves the performance of the collection system, resulting in much reduced number and volume of SSOs, fewer customer complaints, improved efficiency of operation and maintenance (O&M) activities, and increased longevity of the collection system's infrastructure.

Table 7-1. CMOM Program Elements

Element	Purpose	Description
Goals	To provide direction on all aspects of managing the collection system.	Goals should be specific, realistic, achievable, and measureable. • Determine linear footage of sewers to be inspected annually. • Determine number of manholes to be upgraded annually. • Upgrade maintenance management system. • Develop fats, oils, and grease (FOG) program. • Set limits on number of SSOs per year.
Organization	To structure the organization for efficient operation and management of the collection system.	• Write organization and governing body description. • Prepare organization chart. • Write job descriptions. • Define lines of communication.
Legal authority	To establish the legal authority allowing the permit holder to direct all critical aspects of sanitary sewer management.	The permit holder has the legal authority to do the following: • Control rates. • Regulate the volume and strength of discharges. • Manage FOG. • Maintain and replace service laterals.

Table 7-1. CMOM Program Elements		
Element	**Purpose**	**Description**
O&M activities	To operate and maintain the sanitary sewer collection system in a way that achieves optimum sewer performance.	• Identify the O&M activities required to maintain, sewers, manholes, pump stations, force mains, and service laterals. • Establish frequencies for performing the required activities that optimize sewer performance.
Design and performance provisions	To establish minimum requirements for collection system design, construction, inspection, and final acceptance.	• Determine minimum requirements for design. • Determine minimum requirements for construction materials. • Clearly define inspection requirements and train inspectors.
Overflow Emergency Response Plan	To establish response capabilities for responding to sewer emergencies.	• Clearly define emergency procedures. • Provide equipment and personnel training. • Install operating alarm system. • Create public notification plan.
Capacity assurance	To identify where hydraulic deficiencies may occur in the sanitary sewer collection system.	• Map collection system completely and accurately. • Model the collection system including sewers and pump stations. • Identify potential hydraulic deficiencies and create a plan for addressing the deficiencies. • Identify potential operational problem areas and create a schedule for cleaning affected sewers. • Create action plan for addressing areas with excessive I/I.
Annual self auditing	To evaluate where improvements are required in managing the sanitary collection system through annual auditing.	• Compare collection system performance with goals established to identify where improvements may be required. • Conduct annual self-evaluation and practice continuous improvement.

7.2 Current CMOM Practices and Improvements

The City of Sweet Home (City) has implemented several of the elements listed in Table 8-1 as part of the last 12 years of I/I abatement and as required by its past and present NPDES permits. Of the eight elements, those listed in Table 7-2 have been implemented/or currently practiced.

Table 7-2. CMOM Implemented Elements	
Element	**Current practice**
Goals	Determined linear footage of sewers to be inspected annually
Legal authority	Past NPDES permit required City to establish legal authority to control inflow
O&M activities	• City has a self-run inspection program • City has a cleaning program/cycle
Design and performance provisions	City has adopted Oregon Department of Transportation standard provisions for construction
Capacity assurance	• City has mapped and modeled the sewer system in its geographic information system • City has rehabilitated priority mains and laterals in priority basins to address excessive I/I and is undertaking a WWTP facility plan update to determine how to handle I/I that is not cost-effective to remove • City has identified locations with hydraulic deficiencies as outlined in this report • City knows of areas with frequent cleaning needs and has implemented a cleaning program to maintain capacity of problem areas • City has implemented plan to address areas of system with high levels of I/I as outlined in this report.

The current practices listed in Table 7-2 are good steps toward achieving the goals of a CMOM program, but additional efforts should be taken to ensure that all efforts and results (i.e., sewer inspections footage and updated condition, documentation of sewer cleaning, etc.) are properly measured and documented. Much of the O&M activities are run in-house. Without proper documentation, the City runs the risk of the USEPA not recognizing its efforts.

7.3 CMOM Program Recommendation

Table 8-1 identifies the eight proposed components of a well-structured CMOM program. The City has taken progressive steps toward achieving the CMOM program goals by implementing five of the program elements. Brown and Caldwell (BC) recommends that the City expand on the five elements currently in practice by addressing all the requirements listed in the description column of Table 7-1. In addition, BC recommends that the City review its current collection system O&M and management practices and compare them with CMOM program requirements. Missing or partially completed elements listed in Table 7-1 should be addressed. Doing so would reflect an aggressive and proactive approach by the City to achieve the goals of a CMOM program. Documentation of the City's efforts could result in greater leniency from the Oregon Department of Environmental Quality in cases of non-compliance (e.g., overflows during events less than the 1-in-5 year storm).

Section 8

Conclusions and Recommendations

The City has invested over $15 million in planning and construction during the first four phases of rehabilitation and replacement (R&R) work in the collection system. The construction costs for each phase are listed in Table 8-1.

Table 8-1. Summary of R&R Costs by Phase	
Construction phase	Capital cost, million dollars
1	1.3
2	1.7
3	3.1
4	6.0

The City's I/I Abatement Program has addressed approximately 92,000 linear feet or 35 percent of the main line sewers. Over 30 percent of the laterals in Sweet Home have been rehabilitated using a variety of techniques. The extent of service lateral rehabilitation has been completed to varying degrees. Due to access constraints, funding requirements, and budget limitations, not all service laterals have been fully rehabilitated to the private building. This variable level of rehabilitation should be considered when evaluating the rehabilitation effectiveness numbers and when planning future R&R work within the City's collection system.

8.1 Future R&R

Future R&R work in the collection system should continue for the City, either to maintain the level of RDII entering the system or to further target RDII reductions while making structural improvements to the unaddressed sewers that are aging and deteriorating. However, the highest priority basins identified throughout the course of the I/I Abatement Program have been largely addressed and there is a diminishing rate of return on the dollars invested in the collection system. Table 8-2 lists the estimated rehabilitation costs for future R&R work, with the expected reduction in peak RDII.

Table 8-2. Future R&R Work Cost Effectiveness					
Sanitary Basin(s)[a]	Type of R&R	Cost of remaining R&R work, dollars	Peak RDII removed[b], mgd	Cost-effectiveness, dollars per mgd RDII removed	Rank
1	Full rehabilitation, complete uppers	1,620,000	0.18	9,000,000	12
2, 19	Complete uppers	310,000	0.17	1,800,000	1
3	R&R work complete	0	0	0	NA
4	Complete uppers	820,000	0.14	5,700,000	7
5, 6, 21	Complete uppers	970,000	0.39	2,500,000	2

Table 8-2. Future R&R Work Cost Effectiveness					
Sanitary Basin(s)[a]	Type of R&R	Cost of remaining R&R work, dollars	Peak RDII removed[b], mgd	Cost-effectiveness, dollars per mgd RDII removed	Rank
7,13,14,17	Full rehabilitation	7,350,000	1.55	4,700,000	6
8	Full rehabilitation, complete uppers	2,720,000	0.28	9,900,000	13
9	Full rehabilitation, complete uppers	910,000	0.29	3,100,000	4
10	Full rehabilitation, complete uppers	2,990,000	0.42	7,100,000	11
11,12	Full rehabilitation	3,770,000	0.53	7,100,000	10
15	Full rehabilitation	2,130,000	0.31	6,800,000	8
16	Full rehabilitation	2,520,000	0.58	4,400,000	5
18	Full rehabilitation	1,130,000	0.37	3,100,000	3
20	Complete uppers	630,000	0.09	7,000,000	9
	Total	27,900,000	5.30		

[a]*Basins grouped together due to flow monitoring locations and model calibration* methodology.

[b]*Assumes 65 percent reduction in RDII for full rehabilitation, 30 percent reduction for completing uppers.*

An estimated $28 million in construction costs would be required to remove an additional 5.3 mgd. Since $12 million was spent on the first four phases with over 10 mgd removed, the diminishing cost-effectiveness is apparent. However, future R&R work should focus on completing the upper laterals, particularly on Phase 4 sewers, with full rehabilitation efforts directed in Sanitary Basins 18, 9, and 16.

8.2 Findings and Conclusions

The following summarizes the conclusions BC has made based on the modeling results and hydraulic capacity evaluation.

- Post-rehabilitation and reconstruction flow monitoring and hydrologic modeling demonstrate that basin-wide work can remove approximately 65 percent of the projected 1-in-5 year event peak-hour RDII flow in that basin.
- Focusing efforts on rehabilitating sewer mains, manholes, and laterals to the private building has been found to be the most effective at removing peak-hour RDII. Focusing only on specific components such as mains or laterals offers some reduction but at a much lower cost-effectiveness.
- To date, over 50 percent of the peak-hour RDII has been removed from the system over four phases of R&R work.
- Approximately an additional 4.5 mgd of RDII will need to be removed or accommodated at the WWTP to pass the 1-in-5 peak-hour flow under existing conditions, and approximately 6.3 mgd will need to be removed to handle future conditions. These are conservative estimates based on the modeling work.

- Under existing conditions, a single manhole at Long and 18th streets is predicted to overflow in the 1-in-5 year event. The manhole and associated pipe segments were rehabilitated in Phase 4 but this manhole was not identified as a potential overflow location. It is possible that the slight reduction in inside diameter from the Phase 4 reconstruction work and refined flow data and model calibration since the 2009 modeling effort are contributing to the predicted overflows.
- The benefits of R&R work in select basins have not been realized fully due to partial lateral rehabilitation caused by funding agency constraints related to work on private property without a permanent easement and/or owner unwillingness to allow for the work to be completed. Completing the rehabilitation work on the uppers in these partially completed basins (see Table 8-2) is the most cost-effective way to remove additional RDII.
- Full replacement of sanitary basins 18 and 9 have the most cost-effective R&R remaining in the City, with an approximate cost of $2.04 million (2010 R&R costs) to remove approximately 0.66 mgd of peak-hour RDII. Sanitary Basin 8, conversely, has an approximately $2.7 million R&R cost to remove an estimated 0.28 mgd of peak-hour RDII.
- Upsizing and rerouting of flows from Sanitary Basins 5 and 6 toward Sanitary Basin 2 has significantly reduced the potential for overflows at the upstream of the siphon under Ames Creek, but may have resulted in the negative effect of allowing previously restricted I/I to now enter the system.
- A number of locations where overflows were identified as overflow points in the Post-Phase 3 modeling effort, particularly along the 18- to 24-inch main trunk that parallels the railroad, are now no longer projected to overflow based on the rehabilitation work conducted as part of Phase 4.
- Whereas the *Sanitary Sewer Master Plan* identified approximately $1.4 million in upsizing pipes to pass the 1-in-5 peak-hour flows (2012 dollars), the R&R work under the last four phases has essentially eliminated the need for upsizing of pipes. This assumes that the rate of RDII does not increase over time and that the City finds surcharging up to the manhole rim but not overflowing acceptable during the 1-in-5 year event. The City should continue to address RDII in the system on an annual basis. Under existing conditions, there is one manhole in Sweet Home that is predicted to overflow during the 1-in-5 year peak-hour flow event.
- Under future conditions, there are three additional manholes that are predicted to overflow during the 1-in-5 peak-hour flow. Several additional manholes on or immediately adjacent to the 24-inch main trunk line just upstream of the WWTP experience increased surcharging to within 3 feet of the manhole rim.

8.3 Recommendations

BC recommends that the City takes the following steps to continue to manage the I/I in the system with the goal of regulatory compliance:

- Closely monitor the single manhole at the downstream end of Sanitary Basin 10 on Long Street that is predicted to overflow during the 1-in-5 year peak-hour flow. Due to margin-of-error and compounding conservative assumptions within any modeling effort, it is possible the predicted overflow may be overly conservative. Therefore as a precaution, the City should clean and monitor this section of pipe annually and also prior to anticipated large wet-weather events. In addition, there is a significant portion of Sanitary Basin 10 that has not been addressed by the first four phases of the program. R&R work in Sanitary Basin 10 will likely greatly reduce the overflow potential, both in existing as well as future conditions. Additional flow monitoring at monitoring location 9.1 to validate the modeling predicted peak flows is also recommended.
- Evaluate sealing or raising the three manholes just east of 9th Avenue on the east-west 24-inch trunk paralleling the railroad tracks. These manholes are predicted to overflow under future conditions but

sealing or raising these manholes will prevent overflows while also not creating any adverse affect elsewhere in the City's collection system.

- Prepare an update to the City's Wastewater Facility Plan to determine the feasibility and cost of an upgrade to the Sweet Home WWTP to accommodate additional flows and determine the break-even point between WWTP upgrades and RDII reduction through future R&R work. As part of this update, re-evaluate the future growth projections and timing of expansion of the City's wastewater service areas.
- Prioritize completion of the rehabilitation work on upper laterals to complete the holistic basin approach, per Table 8-2. Further R&R work in the collection system aimed at reducing peak-hour RDII has diminishing returns. **However, at a minimum the City must continue with additional R&R work to maintain the current level of RDII in the system.** Sanitary Basins 18 and 9 are the next highest priority basins with the largest predicted RDII removal rates. Look for opportunities to remove I/I while also addressing the pipes with the worst structural ratings.
- Explore implementing a lateral rehabilitation program that can address the private laterals without the constraints of acquiring permanent easements.
- Update sewer condition maps that document the structural and operational condition of sewers. The last comprehensive update of sewer condition was completed in 2006.
- Evaluate the cost and feasibility for addressing Grade 5 sewers (as defined in Section 6 of the main report). Many Grade 5 sewers are likely rated so severely due to isolated point defects rather than full pipe issues. However, failure of point defects are as problematic as full length failures and the City should plan for the rehabilitation of these Grade 5 sewers.
- Begin preparing for and implementing a formal Capacity, Management, Operations, and Maintenance (CMOM) Program, in accordance with U.S. Environmental Protection Agency guidelines. The Oregon Department of Environmental Quality has guidance documents that indicate cities with compliant CMOM plans in place will receive greater leniency in cases of non-compliance (e.g., overflows during events less than the 1-in-5 year storm, see Appendix B).
- Install flow meters and increase the monitoring resolution in Sanitary Basins 7, 13, 14, and 17 to further delineate flows and determine if full basin rehabilitation would be effective. The City's post-Phase 4 flow monitoring was extremely successful, and the City can utilize their flow monitoring equipment and experience to identify and prioritize areas of additional RDII reduction.

By continuing to monitor flows and completing rehabilitation projects, the City can expect to further quantify I/I problems, focus the I/I reduction program on priority areas, and quantify the impact of specific projects, all while focusing funds on the most cost-effective solutions. This further the goal of reducing peak wet weather flows and meeting regulatory compliance. By addressing I/I with a methodical and long-term approach, the City can expect to maximize effectiveness and minimize the financial burden of I/I reduction projects.

Section 9

Limitations

This document was prepared solely for City of Sweet Home, Oregon in accordance with professional standards at the time the services were performed and in accordance with the contract between City of Sweet Home, Oregon and Brown and Caldwell dated January 21, 2010. This document is governed by the specific scope of work authorized by City of Sweet Home, Oregon; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on information or instructions provided by City of Sweet Home, Oregon and other parties and, unless otherwise expressly indicated, have made no independent investigation as to the validity, completeness, or accuracy of such information.

Section 10

References

Water Environment Research Foundation project report "Sanitary Sewer Overflow (SSO) Flow Prediction Technologies," Project 97-CTS-8, April 1999.

Earth Tech Team, *Regional Needs Assessment Report*, Regional Infiltration and Inflow Control Program, Appendix A5 Assumptions for Regional I/I Control Program, King County, 2005 (pg. 7)

City of Sweet Home Sanitary Sewer Master Plan, Brown and Caldwell, 2002.

City of Sweet Home Wastewater Facility Plan, Brown and Caldwell, 2002.

City of Sweet Home Sewer Inspection Project, Brown and Caldwell, 2006.

Appendix A: SFE Flow Monitoring Report, 2012-2013

Final Report for

City of Sweet Home
Oregon, USA

Attn: Ms. Corianne Hart P.E.
Brown and Caldwell

2012/2013 Sanitary Sewer Flow Monitoring

5 Flow Sites and 1 Rain Gauge

Prepared and submitted by:

SFE Global Inc.
1313 East Maple Street
Bellingham, Washington 98225
Toll Free: 1-866-332-9876

May 22, 2013

Ms. Corianne Hart P.E.
Brown and Caldwell Engineering
6500 South Macadam Ave
Portland, Oregon
97239

FINAL REPORT: SWEET HOME, OREGON
Sanitary Sewer Flow Monitoring
November 2012 to March 2013
(5 Flow Sites plus 1 Rain Gauge)

Dear Ms. Hart,

Please find enclosed SFE's Final Report for the above mentioned project. If you have any questions, comments or concerns, please do not hesitate to contact us at your earliest convenience.

Thank you for having SFE conduct this work on your behalf. We are appreciative of the opportunity to work with you and your team on this project. We look forward to working together again in the near future.

Sincerely,
SFE Global
SFE File #U12-118

Paul Loving
Operations Manager
(604) 992-6792
Paul.loving@sfeglobal.com
www.sfeonline.com

TABLE OF CONTENTS

1. Introduction

This report provides details of the sanitary sewer flow monitoring project conducted in the City of The Sweet Home, Oregon. SFE Global was retained by The Brown and Caldwell, under the direction of Ms. Corianne Hart. Mr. Paul Loving represented SFE Global as Project Manager during this project.

As requested, SFE installed (5) sanitary sewer flow monitors and (1) Tipping Bucket Rain Gauges to collect data for a Five (5) month period. The stations were installed by November, 2012 and removed April 1, 2013. The monitoring stations are as follows:

Site #	Location	Meter Utilized
U12-118-1A	4th Ave at Main Street	ISCO 2150 AV Flow Meter C/W SFE CCW Weir
U12-118-2	490 Main Street	ISCO 2150 AV Flow Meter C/W SFE CCW Weir
U12-118-3	8th Ave West of 9th Ave Intersection	ISCO 2150 AV Flow Meter C/W SFE CCW Weir
U12-118-8	Off 15th Ave in greasy area	ISCO 2150 AV Flow Meter C/W SFE CCW Weir
U12-118-8A	18th Ave at RR Tracks	ISCO 2150 AV Flow Meter C/W SFE CCW Weir
Rain Gauge	Public Works Yard	Isco 2105 Data logger with RG

2. Flow Monitoring Stations

Prior to installing flow monitoring stations, SFE performed detailed site assessments of each potential site to determine the appropriate monitoring method. Factors such as pipe size, channel condition, site location, site access, and flow hydraulics were all considered and documented while performing site assessments. See Appendix #2 of this report for site assessment details.

SFE installed the flow monitoring stations in accordance with the approved site assessment documentation. The meters were calibrated and set to log data at 5 minute intervals as per spec and standard SFE procedure . To ensure proper operation of the stations, a regular maintenance schedule was adhered to for the duration of the project. During each site maintenance inspection conducted by SFE, corresponding meter and field readings were obtained and recorded on the field maintenance sheet. These readings provided an indication of the accuracy and operation of the meter. See Appendix #3 of this report for the field report sheets detailing site inspection information, calibrations, and depth verifications.

U12-118 – Site 1 : 4th Avenue at Main Street

SFE installed an ISCO Area Velocity Meter to monitor level readings through the installed SFE Custom Compound Weir within the manhole to monitor flow from the 8 inch diameter pipe. Flow was calculated using the Head/Flow table entered into the flow meter's internal computer. Monitoring duration was from November 1 2012 to April 1, 2013. All equipment was removed from the site. There was a meter failure at this site January 15th 2013 that resulted in complete replacement of installed equipment. Data was edited from this point on due to level readings being recorded too high as per maintenance visits.

U12-118 – Site 2 : 490 Main Street

SFE installed an ISCO Area Velocity Meter to monitor level readings through the installed SFE Custom Compound Weir within the manhole to monitor flow from the 18 inch diameter pipe. Flow was calculated using the Head/Flow table entered into the flow meter's internal computer. Monitoring duration was from November 1 2012 to April 1, 2013. All equipment was removed from the site and no data issues were observed.

U12-118 – Site 3 : 8th Avenue West of 9th Avenue intersection

SFE installed an ISCO Area Velocity Meter to monitor level readings through the installed SFE Custom Compound Weir within the manhole to monitor flow from the 24 inch diameter pipe. Flow was calculated using the Head/Flow table entered into the flow meter's internal computer. Monitoring duration was from November 1 2012 to April 1, 2013. All equipment was removed from the site and no data issues were observed.

U12-118– Site 8 : 15th Avenue in Grassy Area

SFE installed an ISCO Area Velocity Meter to monitor level readings through the installed SFE Custom Compound Weir within the manhole to monitor flow from the 24 inch diameter pipe. Flow was calculated using the Head/Flow table entered into the flow meter's internal computer. Monitoring duration was from November 1 2012 to April 1, 2013. All equipment was removed from the site and no data issues were observed.

U12-118 – Site 8A : 18th Avenue at Railroad Tracks

SFE installed an ISCO Area Velocity Meter to monitor level readings through the installed SFE Custom Compound Weir within the manhole to monitor flow from the 10 inch diameter pipe. Flow was calculated using the Head/Flow table entered into the flow meter's internal computer. Monitoring duration was from November 1 2012 to April 1, 2013. All equipment was removed from the site and no data issues were observed.

U12-118 – Rain Gage 1 - Public Works Yard

A tipping bucket Rain Gage with an Isco 2105 Datalogger was utilized to collect Rainfall Data for the last 2 months of this project after City RG malfunctioned.

3. SFE Custom Compound Weir and Area Velocity Meter Sensors

See Appendix 1 of this report for technical information that provides details on the SFE Custom Compound Weir and Area Velocity Meter Sensors.

4. Site Maintenance

SFE conducted thorough site maintenance and field data verifications throughout the monitoring period. All field maintenance sheets are included as Appendix #3 of this report.

5. QA/QC and Safety Statement

SFE confirms that all flow monitoring stations were installed according to SFE's QA/QC methodology and protocol, and standard industry practice. All flow monitoring equipment has been removed from the site locations.

SFE has a comprehensive Company Safety Manual and can be reviewed upon request.

Confined space entry procedures and general site/traffic safety was adhered to during site installation and site maintenance. SFE utilizes an approved rescue system, a 2800 CFM air induction device and four-gas air quality monitors. All of our staff members are thoroughly trained and certified in confined space entry procedures. Certificates are available upon request.

A thorough traffic control plan was established and used by SFE Global crews where required.

6. Data

Data collected during this project has previously been submitted to Brown and Caldwell Eng, via web access. All data submitted is in RAW format and has not been altered.

Appendices

Appendix #1	Includes technical information on the SFE Weir and Area Velocity Meter Sensors
Appendix #2	Includes all site assessment sheets, site photos, field set-up reports and site maintenance sheets
Appendix #3	Data Summaries and Graphs

Report End
May 2013

SFE Global
File #U12-118

Appendix 1

Technical Information on the SFE Weir and Area Velocity Meter Sensors

SFE Custom Compound Weir - A Technical Discussion

SFE's Custom Compound Weir (CCW) Technology was first developed in 1983. This system consists of the following two components:

- *A customized primary device (Custom Compound Weir or CCW), which provides a predictable relationship of "head" versus "flow"*
- *A water level sensor and data logger*

Testing & Awards

The relationship between "head" and "flow" for the primary device was initially established in a hydraulics lab in conjunction with the Canadian Center for Inland Waterways (CCIW) and published in a report prepared for a local utility. In subsequent years the monitoring techniques were further refined and additional laboratory work was carried out for the primary device. The work was recognized in 1988 by the Association of Consulting Engineers with an Award of Merit at their annual national engineering awards program.

Any level sensing device may be used to reliably measure flows including ultrasonic level indicators, pressure transducers and floats. The system was designed to make it economically feasible for even small utilities to be able to operate a network of stations for a long duration - the low operating costs & high accuracy/reliability prevailing over other measurement systems.

Self-Cleaning

The primary device has a rectangular notch, which then flares out into a "V" section and then a rectangular upper portion. The notch and "V" section have chamfered 1 ½ inch thick "lips" which make them self cleaning and result in a very high weir flow coefficient.

The self-cleaning properties of these weirs have been amply field proven over the past 20 years at approximately 2200 such stations. Each of our Custom Compound Weirs is custom designed by an open channel hydraulics specialist, for the manhole, chamber or channel configuration it is to be used in.

Low Flow Accuracy

For sewers up to 21 inches in diameter the notch is typically 4 inches wide and 5 ½ inches deep. This results in a flow rate of roughly 0.25 GPM for a head of 1 inch. Since a 2.5 psi pressure transducer or narrow beam ultrasonic indicator is usually capable of measuring water levels within +/- ¼", flow rates down to 0.25 GPM can readily be measured (a special unit has previously been designed to measure pre-treated wastewater flow rates down to 0.025 GPM).

No Sewer Backups

The lower notch magnifies the variation of the water level with small changes in flow rate (e.g. for the base flow regime). The overall primary device or "weir" normally has an opening greater than the pipe cross sectional area and capacities greater than that of the sewer in which they are placed.

Any Size, Any Shape

SFE has installed custom compound weirs in sewers from 6 inch to 12 foot as well as in varying sizes of pond outlets, creeks, WWTP's, etc. Custom designing the primary device for the manhole or channel in which it will be placed means that you have considerable control over the final flow regime. This has allowed many difficult hydraulic situations to be handled including bends, junctions, slopes over 10%, drop connections, and drops in the main pipe invert.

Velocity Measurements Not Required

One of the major advantages of SFE's Custom Compound Weir is that it only requires a depth sensor and logger; a velocity sensor is not used. Many of the problems associated with sewer flow monitoring are related to the velocity sensor and the need to measure average velocity. Velocity sensors are prone to fouling with subsequent "drifting" of the signal whereas pressure sensors will still accurately register variations in water level even if they have debris on them.

No "In the flow" Probes

The use of SFE's Custom Compound Weir further improves the performance of pressure sensors since they no longer represent an effective obstruction in the flow (they are installed behind the weir). They will always have a reasonable "head" on them as the weir lip elevation maintains a minimum depth of 4 inches behind the weir. As pressure transducers are much less accurate when depths approach zero; this situation becomes a problem for Area-Velocity A-V) type meters in small pipes where base flow rates are low.

Less Expensive

"Level only" monitors such as those used with our Custom Compound Weir are less expensive than A-V meters and need less power to operate. Flow profiling is needed for conventional A-V meters to ensure that the velocity sensed at a point or across a band of flow is properly transformed into average velocity across the pipe section. Since the Custom Compound Weir does not use velocity, profiling becomes redundant.

High Accuracy

Dye dilution and full-scale lab comparisons have been conducted and the results have been excellent. In most cases +/- 5% over the full range of flows is readily achievable.

Temporary or Permanent

The Custom Compound Weir's (CCW's) are normally located in the manhole chamber about 12 inches from the downstream end.

Material	Life Expectancy	Uses
Lumber/Lexan	1 week to several years	Short Term (E.g. I/I Study)
Plywood	Up to 2 years	Temporary
Pressure Treated Lumber	5 years	Semi-Permanent
Lexan and 316 Stainless	50 Years	Permanent

No Surcharges

Is there a possibility of sewer surcharges causing basement flooding because of the use of such primary devices or weirs? The question has been raised many times and was addressed on a project when the Custom Compound Weir was first designed. The purpose of that first project was to determine the cause of persistent sewer related basement flooding. The client was very concerned that the study procedures did not create more flooding since two Custom Compound Weir stations were just downstream of the area receiving the flooding. The design and placement of the Custom Compound Weirs addressed this as follows:

> *Each CCW was located in a manhole, and not in the pipe, approximately 12 inches from the downstream end so that if the weir were to ever get blocked it could simply overflow safely. (This event has never occurred).*
>
> *For manholes with a chamber larger than the pipe (i.e. 18 inch pipe in standard 42 inch manhole), the weir opening is greater than the pipe area. The flow over the weir is also at critical depth and therefore at a higher velocity than normally occurs in the pipe itself. As a result, the weir capacity is much greater than the pipe capacity in most installations.*
>
> *A rating curve was provided for a demo weir that has the standard opening used in pipes up to 18 inches. The table below shows the flow capacity of this weir configuration at selected heads versus the full flow capacity of selected pipe sizes up to 18 inches at a 0.25 % grade. The comparison illustrates that the CCW capacity can be much greater than the pipe capacity.*

Flow Capacity of Standard Small Pipe Configuration at Selected Heads (Custom Compound Weir range)		Full Flow Capacity of Selected Pipes @ 0.25 % Grade (Pipe range)	
Head (in.)	Flow (US GPM)	Pipe Diameter (in.)	Capacity (US GPM)
1	15.85	8	254
5.5	190	10	471
8	349	12	761
12.5	999	15	1388
20	2298	18	2267
24	3638		

Laboratory Tested

Hydraulic model testing conducted at the Canada Center for Inland Waters (CCIW), provided the opportunity of observing the pipe / weir / manhole performance as the flow rates in the system were increased to the point that it surcharged. As the system started to surcharge, the "control" shifted from the weir to the downstream pipe and there was essentially no drop in the water surface across the weir (under surcharge, the weir was not influencing the water levels upstream).

Custom Designs

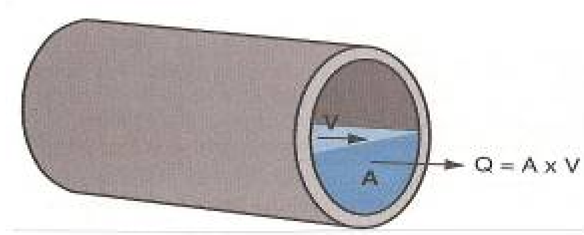
Every Custom Compound Weir is custom designed with a rectangular low flow notch and chamfered lips to give it a high weir flow coefficient. This means that it passes a greater flow for a given head than normal sharp crested weirs. Custom designed means specific concerns are addressed at specific sites.

Area Velocity Meter - Calibration & Verification of Monitor Sensors

Pipe Conduit Measurements

The measurement and condition of all sites were recorded during meter installation.

General Site Installation



Meter velocity was field calibrated according to the manufacturer's methodology and data was verified utilizing SFE Standard Protocol as outlined below.

Depth Verification

Depth verification was conducted at site and all data included on the field report. Five depth measurements from the meter and corresponding water depth are obtained simultaneously at sequential time intervals and recorded on the field worksheet. The lowest and highest measurements are discarded. The remaining three (3) measurements must be within 2.0 cm of each other. The averaged monitor reading must be within 5 % of the averaged field measurement to be acceptable.

Velocity Verification

Depth and velocity profiles were performed utilizing a Marsh McBirney Flow Mate point velocity meter. This instrument uses the Faraday principle to measure water velocity flowing over three electrodes. This allows an accurate velocity to be measured in a small area of the total flow.

SFE standard procedure is to use the 2-D method to determine average velocity. Numerous measurements are taken form the invert to water surface at the left, center and right thirds of the pipe. These measurements are averaged with the inclusion of readings taken from the upper left and right corner of flow.

SFE's alternate procedure when the pipe diameter is small or flow is sufficient is to use the .9-Vmax method. Point velocity readings are taken throughout the cross section of flow. The highest repeatable Velocity obtained is multiplied by 0.9 to determine average velocity. This average velocity is then correlated to the average velocity reading from the meter and must be within 10 %.

Velocity profiles were conducted and obtained for all sites.

Flow Monitoring Programs – SFE Technology Selection Approach

*SFE **does not** manufacture equipment - we select equipment and technology that in our experience will meet the project objectives in a cost effective and accurate manner.*

Our selection of a flow monitoring technology and the type of meter we use is based on these factors:

- ***A level of accuracy that is conducive to a high level of confidence in the project goals.***
- ***A high rate of recoverability and a focus on collecting as much "usable", un-modified, raw data as possible (greater than 95%).***
- ***The delivery of exceptional information in a timely manner.***

SFE focuses not only on the accuracy of the equipment; we also focus on the **best-suited** equipment and technology (i.e. Area Velocity versus Custom Compound Weirs) for each site. SFE views flow monitoring as matching the best technology to the prevalent "flow regime" at each site as opposed to selecting a specific flow meter. We may for example reject certain Area-Velocity (AV) flow meters as they are unable to provide acceptable combinations of redundant sensors; a combination we believe is imperative for flow monitoring programs in order to reduce the quantity and quality of poor data anticipated, particularly that due to low flow in small pipe (less than 18 inch). Other reasons for SFE to reject certain flow meters could be poor local service support, beta testing problems and QA/QC issue's, supply issues, etc. Conversely, we may accept and draw from any Area-Velocity (AV) meter deemed capable provided they are currently accurate to specification and suited for the project.

The approach described above was recently used at a regional sanitary sewer district; whereby the equipment and technology were evaluated versus an emphasis on evaluation of just the flow meter brand. The flow meters being considered did not have as much influence on accuracy as the type of technology used did. I.e. Several flow meter manufacturers installed various Area-Velocity (AV) meters while SFE also installed a Custom Compound Weir (CCW). The meters were all installed at the same manhole - all but one of the AV meters preformed to specifications, however, they were still not able to provide as much usable and reliable data at "this particular site" as the CCW did due to their inability to collect flow data during low flow, high velocity or turbulent conditions. The CCW collected reliable flow data over the full range of flows and was transmitted and monitored using CDPD wireless technology.

We found that in most cases, AV devices (meters) such as Isco, Sigma ADS, Geotivity, Marsh-McBirney, etc., have acceptable accuracies in terms of reading and reporting, however, it is the flow conditions or flow "regime" that exposes limitations.

For example, the scatter graph in ***Figure One*** below illustrates a flow-monitoring site that is exhibiting good flow characteristics relative to the use of an AV meter.

Figure One

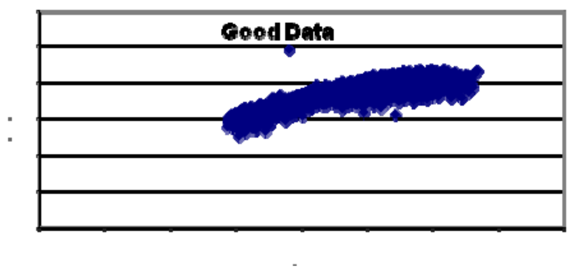


Figure Two

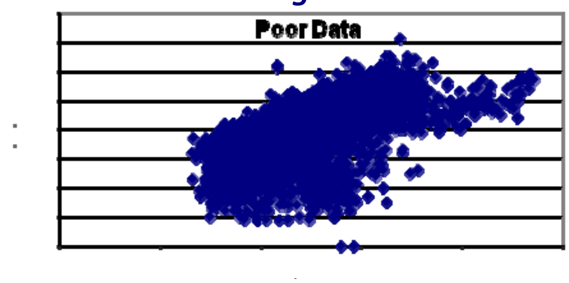
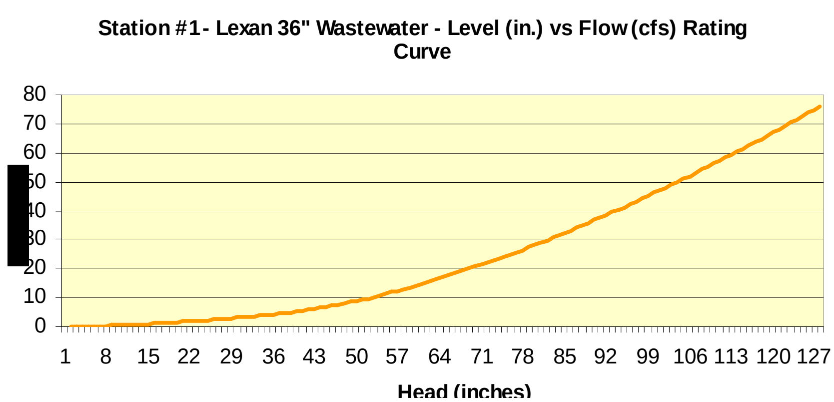


Figure Two illustrates a scatter graph from an AV flow-monitoring site that is not conducive to AV technology due to low flow and/or turbulence. In this case, while the AV meter is recording accurately, the flow characteristics (flow regime) of the site render less than 50% of the data as usable. Data modification and sub-analysis must be conducted in order to extract usable data.

Figure Three below by comparison, is a rating curve used with Custom Compound Weir Technology. Data scatter is eliminated, as there is a known relationship between velocity and depth, which eliminates the need to monitor velocity. The result is greater than 95% recoverability of usable data over the full range of flows at an accuracy of +/- 5% of full scale.

Figure Three



The case point is that while most of the Area-Velocity flow meters used preformed to specification, it is the addition of a primary flow-monitoring device (the Custom Compound Weir) that provided the basis for the collection of accurate data. The flow meters themselves become secondary devices. This does not mean that AV meters are not to be used – they have many suitable uses.

SFE has used CCW Technology at several thousand sites throughout North America and has received the "Order of Merit" by the Association of Professional Engineers.

CCW technology has the following benefits

- Reduces sub analysis and modified data resulting in increased "R-squared" confidence factors for producing I/I summaries
- Highly accurate over the full range of flows
- Highly accurate at low flows
- Highly accurate at high velocity
- Highly accurate at turbulence
- Eliminates data scatter and velocity reading requirements
- Self scouring

Our approach, therefore, is to assess each flow monitoring station and apply the <u>best suited</u> technology to that station. Sites could be AV or CCW, but will be dependant on the prevailing conditions at each location.

Appendix 2

Site Assessment Sheets, Site Photos, Field Set-up Reports and Site Maintenance Sheets

FIELD MAINTENANCE RECORD

NAME: Sweet Home
SFE SITE #: 1A
ADDRESS: 401 Main
GPS:
SENSOR TYPE: Av
PRIMARY DEVICE: 350 Weir

CONSTANTS		
D1 (in):	0.000	D1-lip to x-bar
TOM (in):	71.000	Raw Weir L - x-bar to water
METER #		DATE:
METER #		DATE:
METER #		DATE:

LEGEND	
DL - DOWNLOAD	**PC** - PROGRAM COMPLETE
CB - CHG BATTERY	**PM** - PROG. METER
V - VERIFY	**VIS** - VISUAL
LA - LEVEL ADJUST	**VP** - VELOCITY PROFILE
DO - DEPTH ONLY	**CD** - CHG DESICCANT

DATE	TIME	METER TIME	METER DEPTH	FIELD DEPTH	METER VEL	FIELD VEL-VIS	FLOW	BATT	SILT	Raw Weir L	Calc Weir L	MTC BY	COMMENTS
M/D/YY	HH:MM	HH:MM	in	in	fps	fps	cfs	V	in	in	in	(INIT.)	
10/17/12	10:56	9:51	2.132	2.5	*	*	0	12.32	0	na		JS	install
10/18/12	13:01	12:02	2.1	2.5	*	*	0	12.3	0	na		AM	
11/02/12	12:59	11:54	2.9	3	*	*	0	11	0	na		AM	
11/16/12	10:45	10:40	2.515	2.5	*	*	0	10.7	0	na		DC	confirm rating chart
11/29/12	15:44	15:40	2.646	3	*	*	0	10.4/12.4	0	na		DC	CB, FP
12/12/12	10:04	9:57	2.44	2.125	*	*	0	10.8	0	NA		AM	DL
01/03/12	9:12	9:05	1.24	1.75	*	*	0	12.2	0	NA		AM	DL CB
01/15/13	11:14	11:06	-0.814	2	*	*	0	11.5/12.2	0	NA		AM	DL CB install new sensor meter running neg
01/15/13	11:15	11:16	2.34	2.5	*	*	0	11.3	0	NA		AM	calibrate new sensor
02/05/13	10:58	10:51	5.2	5	*	*	0	10.7	0	NA		AM	month end down load
02/20/13	14:20	14:25	3.8	3.75	*	*	0	10.5	0	NA		AM	DL Clean site
03/06/13	14:36	14:36	2.39	2.75	*	*	0	12.2	0	N/a		AM	DL clean site CB
03/21/13	13:40	12:35	3.55	3.875	*	*	0	11.7	0	NA		AM	DL clean
04/04/13	7:46	6:43	2.12	2	*	*	1.63	10.2	0	NA		AM	DL and remove site

DATE:	NOTES:

Site Assessment

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**1A**
Date / Time:	Oct 17 2012		

Project Specific Information

Client Name:	Brown and Caldwell	
End User Name:	Same	
Project Name:	U12-118	
Client Contact:	Rob Lee	503-977-6625
Field Contact:	Adrian Marshall	509-312-0612
SFE PM Contact:	Paul Loving	604-992-6792

Site Equipment

Install / Remove Date:	Oct 17 2012
Meter Make & Model:	Isco 2150 AV
Level Type:	Pressure
Velocity Type:	NA
Primary Device:	SFE CCW
Wireless:	Yes
Redundancy:	Yes, additional AV in Pipe
Logging Rate:	5min

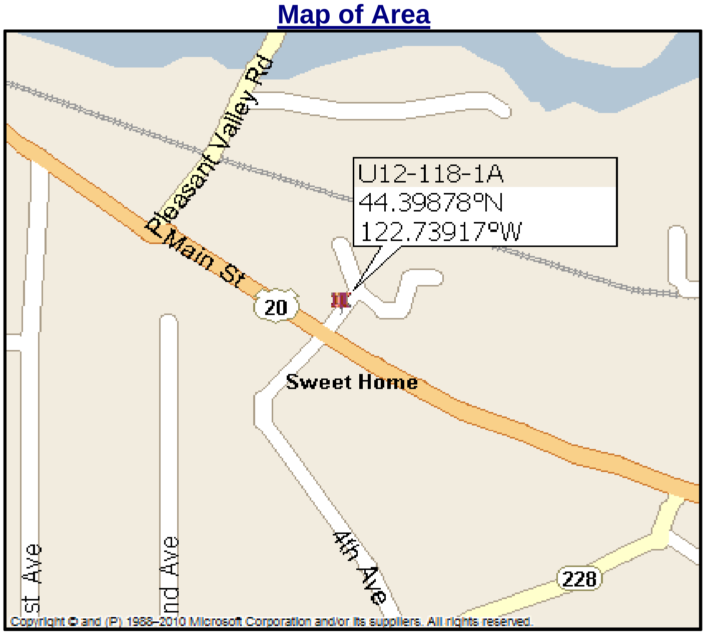
Site Location Information

Client Manhole #:	1A	
Address (Location):	4th Avenue at Main Street	
City, State:	Sweethome, Oregon	
GPS (North - West):	44.39878	-122.73917
Landmarks:		
Additional Information:	Drop pipe into MH above Weir lip	
	need additional meter for redun.	

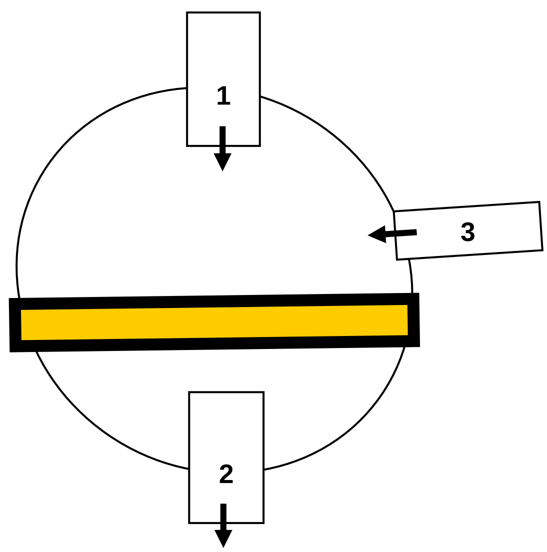
Site Profile

Pipe #1 Size:	8	Inches
Pipe #2 Size:	8	Inches
Pipe #3 Size:	6	Inches
Pipe #4 Size:	NA	Inches
Manhole Depth:	79	Inches
Laterals / Rungs:	Yes	
Additional Information:		

Map of Area



Manhole Layout



Traffic Control Requirements

Provider:	Third Party
Condition	Moderate traffic
Frequency:	Install/Removal
Speed Limit:	35
# of Lanes Effected:	1
Lane Configuration:	Center of 2 lane road
Additional Information:	

Notes

1
2

Site Hydraulics

Date & Time:	Oct 17 2012	13:30
Depth:	1.25	Inches
Velocity:	1	FPS
Turbulent:	No	
Surcharge:	Possible	
Silting:	No	
Solids:	Yes	

Notes

3
4

Site Pictures

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**1A**
Date / Time:	Oct 17 2012		

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Notes

1
2
3

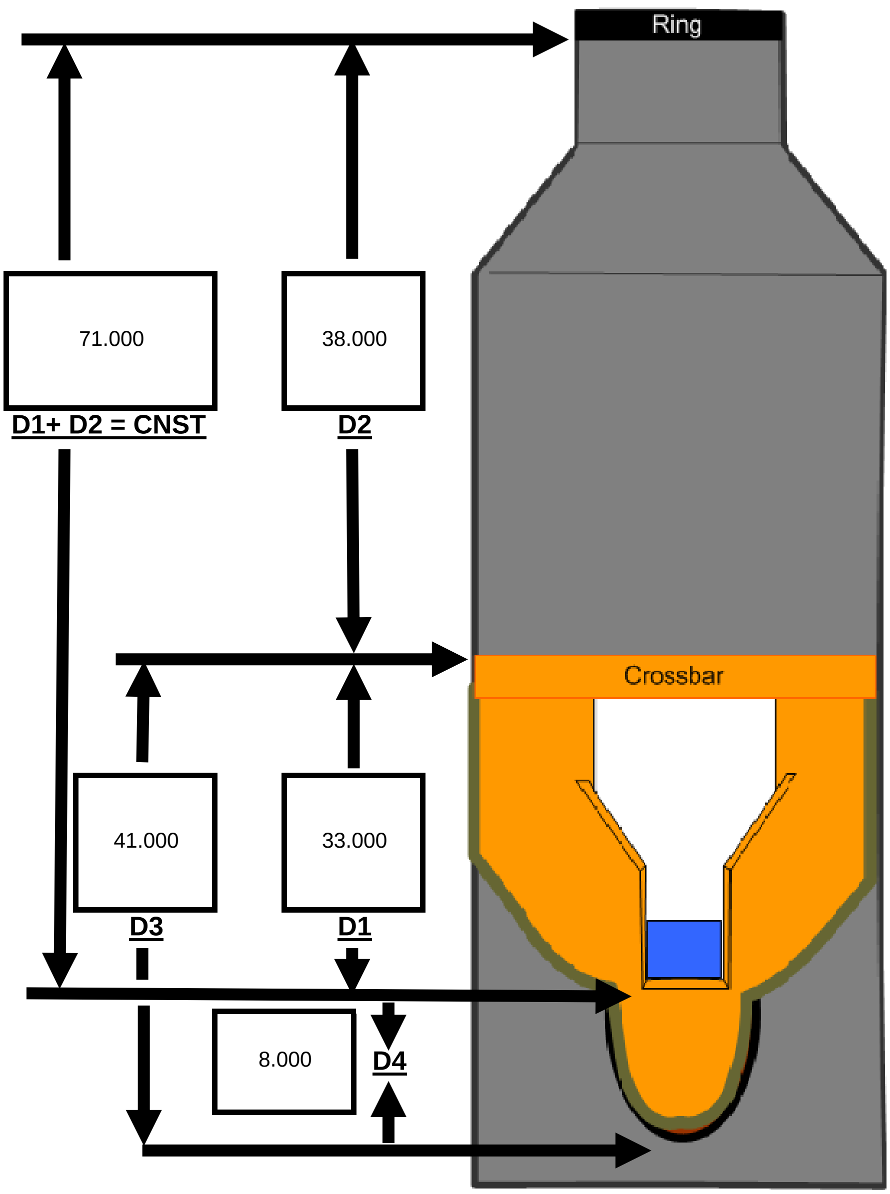
CCW Installation Form

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 17 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **1A**
Technician 1: A Marshall 509-312-0612
Technician 2: Dylan Carvin

Meter Depth vs.. Field Depth Calibration / Verification

Reading Number	Date	Time	Field Meas (in.)	Meter Depth (in)	Comments (Zero Meter Level before Installation)
Initial	Oct 17 2012	14:40	2.500	2.45	Install
1	Oct 17 2012	14:42	2.500	2.53	
2	Oct 17 2012	14:45	2.500	2.59	
3	Oct 17 2012	14:50	2.500	2.51	
Average				.	



Constant Measurement (in)
Rim to Weir Lip
71.000

Pipe Diameters (in)

Pipe 1	8
Pipe 2	8
Pipe 3	6
Pipe 4	NA

D4=Invert to Weir Lip (D3-D1)
8.000

Obvert to Weir Lip
NA

Final Check-off Sheet

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**1A**
Date / Time:	Oct 17 2012		

Flow Meter Information

Meter Make:	Isco	Logging Rate:	5 Minute
Meter Model:	2150	Flow Units:	CFS
Sensor Type	AV	Velocity Units:	FPS
Meter Serial Number:	NA	Depth Units	Inches
Battery Volts:	12.2	Surcharge Meter (Y/N):	Yes

Site Physical Information

Silt Level:	0	Weather:	Sunny
Slope:	NA	Weir Size:	350
Uniform Flow (Y/N)	Y	Depth Only(DO)	NA
Debris in Flow (Y/N):	Y	or Look up Table(LT)	LT
Pipe Material:	Concrete	Comments	Drop pipe into manhole

Check Off List

	Yes	**No**
Time Set:	x	
Depth Calibrated:	x	
Velocity Profile:		x
Download Data:	x	
Meter Running:	x	
Pipe Size Verified:	x	
Photograph Taken:	x	
Site Cleaned:	x	
Site Secured:	x	

FIELD MAINTENANCE RECORD

NAME: Sweet Home
SFE SITE #: U12-118-2
ADDRESS: 490 Main
GPS:
SENSOR TYPE: Av
PRIMARY DEVICE: 600 Weir

CONSTANTS		
D1 (in):	0.000	D1-lip to x-bar
TOM (in):	75.750	Raw Weir L - x-bar to water
METER #		DATE:
METER #		DATE:
METER #		DATE:

LEGEND	
DL - DOWNLOAD	**PC** - PROGRAM COMPLETE
CB - CHG BATTERY	**PM** - PROG. METER
V - VERIFY	**VIS** - VISUAL
LA - LEVEL ADJUST	**VP** - VELOCITY PROFILE
DO - DEPTH ONLY	**CD** - CHG DESICCANT

DATE M/D/YY	TIME HH:MM	METER TIME HH:MM	METER DEPTH in	FIELD DEPTH in	METER VEL fps	FIELD VEL-VIS fps	FLOW cfs	BATT V	SILT in	Raw Weir L in	Calc Weir L in	MTC BY (INIT.)	COMMENTS
10/17/12	14:51	14:02	2.81	2.75	*	*	0.315	12.37	0			JS	install
10/18/12	12:46	11:50	3.1	2.75	*	*	0.428	12	0			AM	
11/02/12	11:49	11:00	3.75	3.772	*	*	0.485	11.7	0			JS	
11/02/12	11:53	11:07	3.5	3.588	*	*	0.457	11.7	0			JS	
11/16/12	8:13	8:24	3.686	3.75	*	*	0.467	10/12/04	0			DC	confirm pipe size, weir measurements, rating chart
11/29/12	16:11	16:21	4.767	4.75	*	*	0.561	10/12/04	0			DC	CB, FP
12/12/12	9:47	9:57	4.656	4.5	*	*	0.669	10.1/12.3	0			DC	CB
01/03/13	8:52	8:02	3.96	3.75	*	*	0.527	8.73/11.93	0			AM	DL, CB
01/15/13	10:20	10:24	4	3.75	*	*	0.551	10.07/12.1	0			AM	DL CB clean weir
02/04/13	16:32	16:41	3.9	3.75	*	*	0.52	12.2	0			AM	DL CB clean weir
02/20/13	15:44	15:44	3	3.25	*	*	0.35	12.2	0			AM	DL CB clean weir
03/06/13	16:10	16:12	4.1	4.25	*	*	0.573	9.8/12	0			AM	DL CB clean weir
03/21/13	13:27	12:36	4.8	5	*	8	4.88	9.9/12.2	0			Am	DL CB clean weir
04/04/13	11:50	11:01	3.37	3.25	*	*	0.391	10.2	0			AM	DL , check and remove

DATE:	NOTES:

Site Assessment

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 17 2012

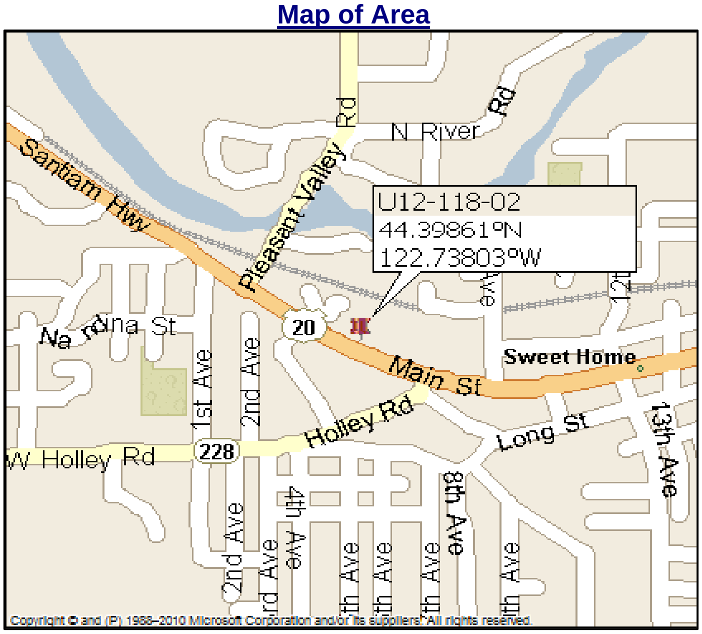
Project Specific Information

Client Name:	Brown and Caldwell	
End User Name:	Same	
Project Name:	U12-118	
Client Contact:	Rob Lee	503-977-6625
Field Contact:	Adrian Marshall	509-312-0612
SFE PM Contact:	Paul Loving	604-992-6792

Site Location Information

Client Manhole #:	2	
Address (Location):	490 Main Street	
City, State:	Sweethome, Oregon	
GPS (North - West):	44.39861	-122.73803
Landmarks:		
Additional Information:		

Map of Area



Traffic Control Requirements

Provider:	NA
Condition	NA
Frequency:	NA
Speed Limit:	NA
# of Lanes Effected:	NA
Lane Configuration:	
Additional Information:	

Notes

1
2

SFE PROJECT #: **U12-118**
SFE SITE #: **2**

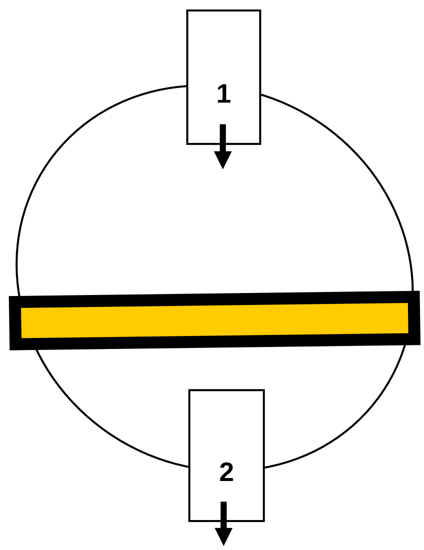
Site Equipment

Install / Remove Date:	Oct 17 2012
Meter Make & Model:	Isco 2150 AV
Level Type:	Pressure
Velocity Type:	NA
Primary Device:	SFE CCW
Wireless:	Yes
Redundancy:	Yes
Logging Rate:	5min

Site Profile

Pipe #1 Size:	18	Inches
Pipe #2 Size:	18	Inches
Pipe #3 Size:	NA	Inches
Pipe #4 Size:	NA	Inches
Manhole Depth:	83	Inches
Laterals / Rungs:	Yes	
Additional Information:		

Manhole Layout



Site Hydraulics

Date & Time:	Oct 17 2012	8:00
Depth:	2	Inches
Velocity:	1	FPS
Turbulent:	No	
Surcharge:	Possible	
Silting:	No	
Solids:	Yes	

Notes

3
4

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**2**
Date / Time:	Oct 17 2012		

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Notes

1
2
3

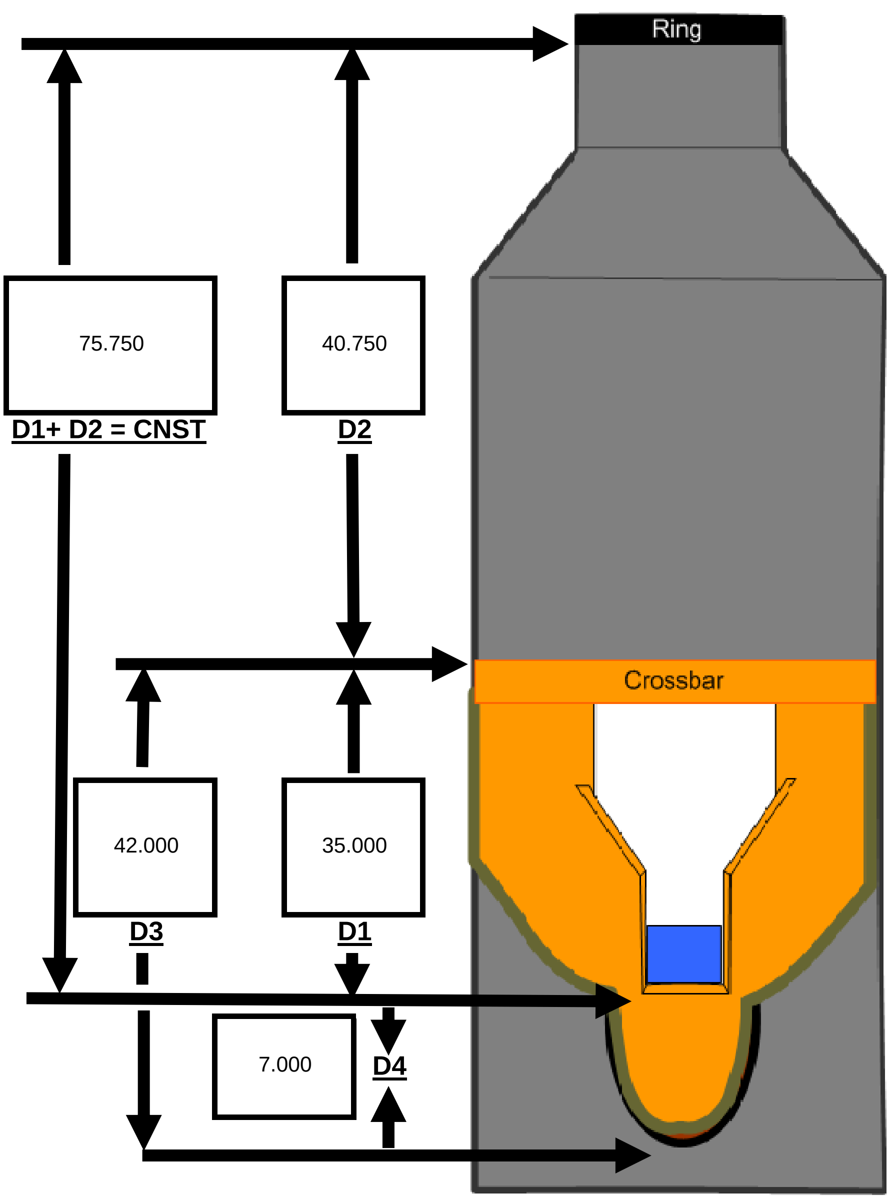
CCW Installation Form

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 17 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **2**
Technician 1: A Marshall 509-312-0612
Technician 2: Dylan Carvin

Meter Depth vs.. Field Depth Calibration / Verification

Reading Number	Date	Time	Field Meas (in.)	Meter Depth (in)	Comments (Zero Meter Level before Installation)
Initial	Oct 17 2012	9:00	3.000	2.98	Install
1	Oct 17 2012	9:05	3.000	3.06	
2	Oct 17 2012	9:07	3.000	3.09	
3	Oct 17 2012	9:10	3.000	3.03	
Average				.	



Constant Measurement (in)
Rim to Weir Lip
75.750

Pipe Diameters (in)

Pipe 1	18
Pipe 2	18
Pipe 3	NA
Pipe 4	NA

D4=Invert to Weir Lip (D3-D1)
7.000

Obvert to Weir Lip
NA

Final Check-off Sheet

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**2**
Date / Time:	Oct 17 2012		

Flow Meter Information

Meter Make:	Isco	Logging Rate:	5 Minute
Meter Model:	2150	Flow Units:	CFS
Sensor Type	AV	Velocity Units:	FPS
Meter Serial Number:	NA	Depth Units	Inches
Battery Volts:	12.2	Surcharge Meter (Y/N):	Yes

Site Physical Information

Silt Level:	0	Weather:	Sunny
Slope:	NA	Weir Size:	600
Uniform Flow (Y/N)	Y	Depth Only(DO)	NA
Debris in Flow (Y/N):	Y	or Look up Table(LT)	LT
Pipe Material:	PVC	Comments	

Check Off List

	Yes	No
Time Set:	x	
Depth Calibrated:	x	
Velocity Profile:		x
Download Data:	x	
Meter Running:	x	
Pipe Size Verified:	x	
Photograph Taken:	x	
Site Cleaned:	x	
Site Secured:	x	

FIELD MAINTENANCE RECORD

NAME:	Sweet Home
SFE SITE #:	U12-118-3
ADDRESS:	110
GPS:	
SENSOR TYPE:	Av
PRIMARY DEVICE:	900 Weir

CONSTANTS		
D1 (in):	0.000	D1-lip to x-bar
TOM (in):	138.000	Raw Weir L - x-bar to water
METER #		DATE:
METER #		DATE:
METER #		DATE:

LEGEND	
DL - DOWNLOAD	**PC** - PROGRAM COMPLETE
CB - CHG BATTERY	**PM** - PROG. METER
V - VERIFY	**VIS** - VISUAL
LA - LEVEL ADJUST	**VP** - VELOCITY PROFILE
DO - DEPTH ONLY	**CD** - CHG DESICCANT

DATE	TIME	METER TIME	METER DEPTH	FIELD DEPTH	METER VEL	FIELD VEL-VIS	FLOW	BATT	SILT	Raw Weir L	Calc Weir L	MTC BY	COMMENTS
M/D/YY	HH:MM	HH:MM	in	in	fps	fps	cfs	V	in	in	in	(INIT.)	
10/18/12	10:29	9:28	5.257	5.25	*	*	1.612	12.276	0			JS	install
11/02/12	11:20	10:18	6.05	6.25	*	*	2.013	11.8	0			AM	
11/16/12	9:16	9:14	5.164	5.5	*	*	1.561	11.3	0			DC	confirm pipe size, weir measurements, rating chart
11/29/12	16:28	16:26	8.218	8	*	*	3.122	10.7/12.3	0			DC	CB, FP, attempt to regain wireless signal
12/12/12	9:11	9:09	7.77	7.25	*	*	2.84	11.5	0			AM	DC, check site, change ant.
01/03/13	8:30	8:35	6.04	5.5	*	*	1.97	10.8	0			AM	DL
01/15/13	10:03	10:00	6.01	5.75	*	*	2.06	11.1	0			AM	DLClean weir
02/04/13	16:20	16:17	6.67	6.7	*	*	2.29	11.1	0			AM	DL clean weir
02/20/13	15:23	15:23	5.2	5	*	*	1.56	11	0			AM	DL clean weir Data go
03/06/13	16:45	16:50	6.7	6.5	*	*	2.33	10.8	0			AM	DL clean weir CB
03/21/13	13:17	12:16	7.6	7.25	*	*	2.8	10.8	0			AM	DL clean weir
04/04/13	10:50	9:51	5.32	5	*	*	1.63	9.3	0			AM	DL Remove

DATE:	**NOTES:**

Site Assessment

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**3**
Date / Time:	Oct 17 2012		

Project Specific Information

Client Name:	Brown and Caldwell	
End User Name:	Same	
Project Name:	U12-118	
Client Contact:	Rob Lee	503-977-6625
Field Contact:	Adrian Marshall	509-312-0612
SFE PM Contact:	Paul Loving	604-992-6792

Site Equipment

Install / Remove Date:	Oct 17 2012
Meter Make & Model:	Isco 2150 AV
Level Type:	Pressure
Velocity Type:	NA
Primary Device:	SFE CCW
Wireless:	Yes
Redundancy:	Yes
Logging Rate:	5min

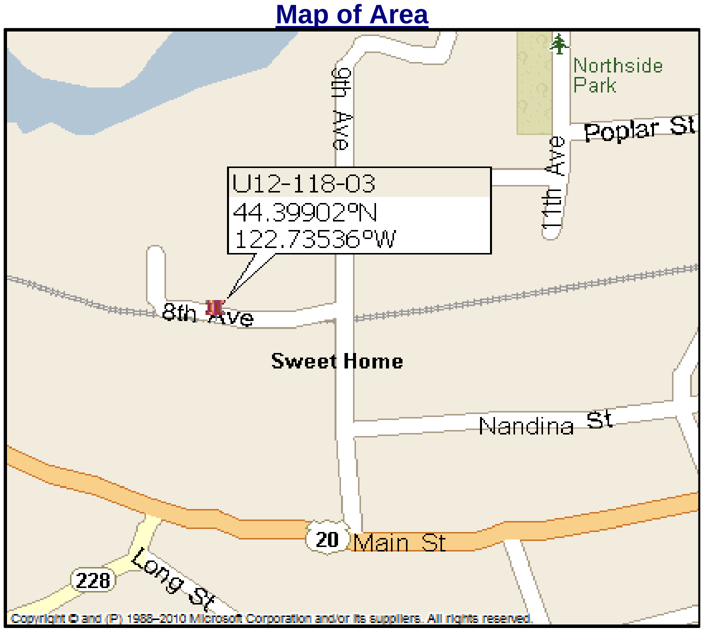
Site Location Information

Client Manhole #:	3	
Address (Location):	8th Avenue	
City, State:	Sweethome, Oregon	
GPS (North - West):	44.39902	-122.73536
Landmarks:		
Additional Information:		

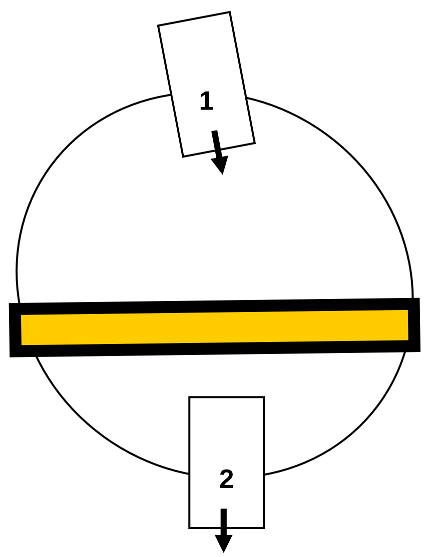
Site Profile

Pipe #1 Size:	24	Inches
Pipe #2 Size:	24	Inches
Pipe #3 Size:	NA	Inches
Pipe #4 Size:	NA	Inches
Manhole Depth:	159	Inches
Laterals / Rungs:	Yes	
Additional Information:		

Map of Area



Manhole Layout



Traffic Control Requirements

Provider:	SFE
Condition	Gravel Lane
Frequency:	NA
Speed Limit:	NA
# of Lanes Effected:	1
Lane Configuration:	
Additional Information:	

Notes

1
2

Site Hydraulics

Date & Time:	Oct 17 2012	11:30
Depth:	1.25	Inches
Velocity:	1	FPS
Turbulent:	No	
Surcharge:	Possible	
Silting:	No	
Solids:	Yes	

Notes

3
4

Site Pictures

CLIENT FLOW MONITORING #: U12-118
NAME: Sweethome, Oregon
Date / Time: Oct 17 2012

SFE PROJECT #: U12-118
SFE SITE #: 3

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Notes

1
2
3

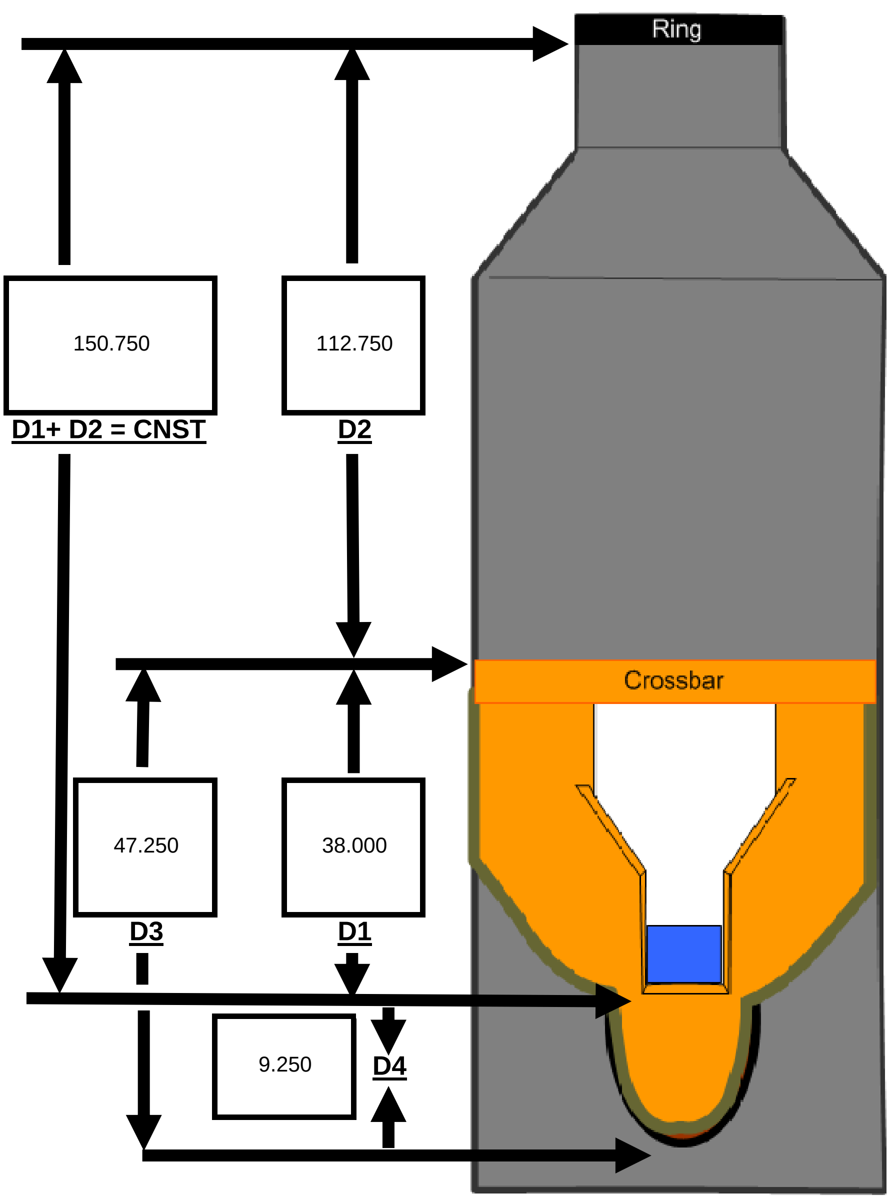
CCW Installation Form

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 17 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **3**
Technician 1: A Marshall 509-312-0612
Technician 2: Dylan Carvin

Meter Depth vs.. Field Depth Calibration / Verification

Reading Number	Date	Time	Field Meas (in.)	Meter Depth (in)	Comments (Zero Meter Level before Installation)
Initial	Oct 17 2012	12:05	1.500	1.43	Install
1	Oct 17 2012	12:08	1.250	1.28	
2	Oct 17 2012	12:11	1.250	1.22	
3	Oct 17 2012	12:13	1.500	1.56	
Average				.	



Constant Measurement (in)
Rim to Weir Lip
150.750

Pipe Diameters (in)

Pipe 1	24
Pipe 2	24
Pipe 3	NA
Pipe 4	NA

D4=Invert to Weir Lip (D3-D1)
9.250

Obvert to Weir Lip
NA

Final Check-off Sheet

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**3**
Date / Time:	Oct 17 2012		

Flow Meter Information

Meter Make:	Isco	Logging Rate:	5 Minute
Meter Model:	2150	Flow Units:	CFS
Sensor Type	AV	Velocity Units:	FPS
Meter Serial Number:	NA	Depth Units	Inches
Battery Volts:	12.2	Surcharge Meter (Y/N):	Yes

Site Physical Information

Silt Level:	0	Weather:	Sunny
Slope:	NA	Weir Size:	900
Uniform Flow (Y/N)	Y	Depth Only(DO)	NA
Debris in Flow (Y/N):	Y	or Look up Table(LT)	LT
Pipe Material:	Concrete	Comments	

Check Off List

	Yes	**No**
Time Set:	x	
Depth Calibrated:	x	
Velocity Profile:		x
Download Data:	x	
Meter Running:	x	
Pipe Size Verified:	x	
Photograph Taken:	x	
Site Cleaned:	x	
Site Secured:	x	

FIELD MAINTENANCE RECORD

NAME:	Sweet Home
SFE SITE #:	U12-118-8
ADDRESS:	1400.5 NADD
GPS:	
SENSOR TYPE:	Av
PRIMARY DEVICE:	900 Weir

CONSTANTS			
D1 (in):	0.000	D1-lip to x-bar	
TOM (in):	83.500	Raw Weir L - x-bar to water	
METER #		DATE:	
METER #		DATE:	
METER #		DATE:	

LEGEND	
DL - DOWNLOAD	**PC** - PROGRAM COMPLETE
CB - CHG BATTERY	**PM** - PROG. METER
V - VERIFY	**VIS** - VISUAL
LA - LEVEL ADJUST	**VP** - VELOCITY PROFILE
DO - DEPTH ONLY	**CD** - CHG DESICCANT

DATE	TIME	METER TIME	METER DEPTH	FIELD DEPTH	METER VEL	FIELD VEL-VIS	FLOW	BATT	SILT	Raw Weir L	Calc Weir L	MTC BY	COMMENTS
M/D/YY	HH:MM	HH:MM	in	in	fps	fps	cfs	V	in	in	in	(INIT.)	
10/16/12	18:25	17:19	4.153	4.625	*	*		13.4	0			JS	install
10/17/12	11:00	9:59	4.41	4.5	*	*		12.4	0			AM	
11/02/12	10:45	9:50	4.761	4.75	*	*	1.392	11.5	0			JS	add cell unit
11/16/12	11:13	11:13	5.037	5	*	*	1.505	9.7/12.3	0			DC	CB, confirm pipe size, weir measurements and
X												X	rating chart
11/29/12	20:57	20:57	6.425	6	*	*	2.18	10.1/12.4	0			DC	CB, FP, attempt to regain wireless signal
12/12/12	8:48	8:46	5.935	6.25	*	*	1.919	10.2/12.2	0			DC	DL
01/03/13	8:22	8:22	4.63	4.75	*	*	1.32	8.7/12.2	0			AM	DL
01/15/13	9:39	9:37	4.74	4.5	*	*	1.378	10.03/12.02	0			AM	DL CB check weir & clean
02/05/13	16:09	16:04	5.26	5.5	*	*	1.607	9.4/12.02	0			AM	DL cb check level
02/20/13	15:07	15:05	3.8	4	*	*	0.988	9.6/118	0			DC	DL cean lite rag less than 1/8 in
03/06/13	15:33	15:32	4.83	4.75	*	*	1.46	7.6/12	0			AM	DL clean weir CB
03/21/13	12:23	11:22	5.8	6	*	*	1.9	9.8/12	0			AM	DL clean weir CB
04/04/13	10:21	9;20	3.9	4	*	*	1.06	9.6	0			AM	DL Remove and check

DATE:	NOTES:

Site Assessment

CLIENT FLOW MONITORING #: U12-118
NAME: Sweethome, Oregon
Date / Time: Oct 16 2012

SFE PROJECT #: U12-118
SFE SITE #: 8

Project Specific Information

Client Name:	Brown and Caldwell	
End User Name:	Same	
Project Name:	U12-118	
Client Contact:	Rob Lee	503-977-6625
Field Contact:	Adrian Marshall	509-312-0612
SFE PM Contact:	Paul Loving	604-992-6792

Site Equipment

Install / Remove Date:	Oct 16 2012
Meter Make & Model:	Isco 2150 AV
Level Type:	Pressure
Velocity Type:	NA
Primary Device:	SFE CCW
Wireless:	Yes
Redundancy:	Yes
Logging Rate:	5min

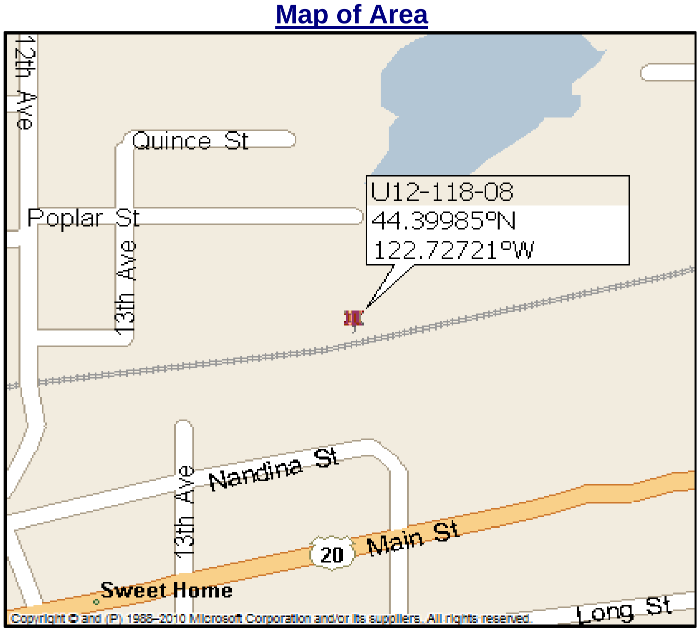
Site Location Information

Client Manhole #:	8	
Address (Location):	15th Avenue	
City, State:	Sweethome, Oregon	
GPS (North - West):	44.39985	-122.72721
Landmarks:		
Additional Information:		

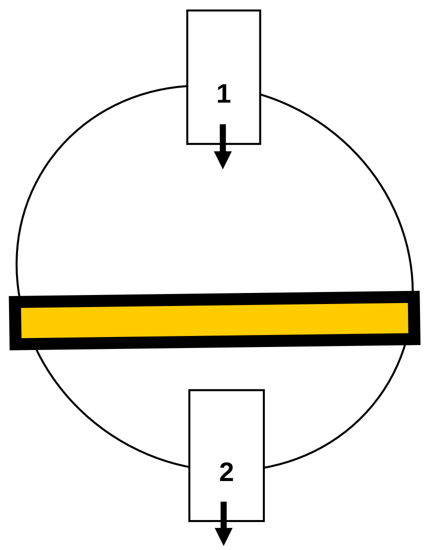
Site Profile

Pipe #1 Size:	24	Inches
Pipe #2 Size:	24	Inches
Pipe #3 Size:	NA	Inches
Pipe #4 Size:	NA	Inches
Manhole Depth:	91	Inches
Laterals / Rungs:	Yes	
Additional Information:		

Map of Area



Manhole Layout



Traffic Control Requirements

Provider:	NA
Condition	NA
Frequency:	NA
Speed Limit:	NA
# of Lanes Effected:	NA
Lane Configuration:	
Additional Information:	

Site Hydraulics

Date & Time:	Oct 16 2012	10:55
Depth:	2	Inches
Velocity:	1	FPS
Turbulent:	No	
Surcharge:	Possible	
Silting:	No	
Solids:	Yes	

Notes
1
2

Notes
3
4

Site Pictures

CLIENT FLOW MONITORING #:	U12-118	SFE PROJECT #:	U12-118
NAME:	Sweethome, Oregon	SFE SITE #:	8
Date / Time:	Oct 16 2012		

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Notes

1
2
3

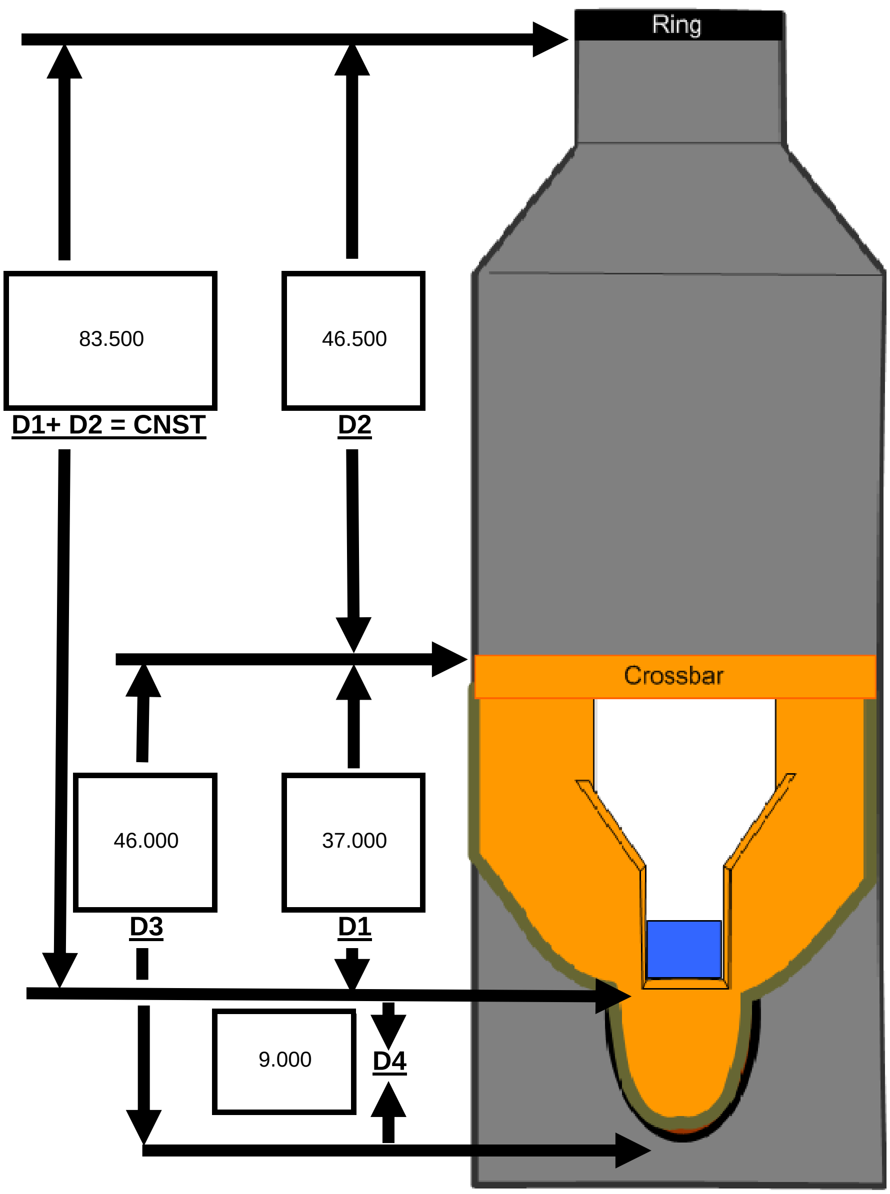
CCW Installation Form

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 16 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **8**
Technician 1: A Marshall 509-312-0612
Technician 2: Dylan Carvin

Meter Depth vs.. Field Depth Calibration / Verification

Reading Number	Date	Time	Field Meas (in.)	Meter Depth (in)	Comments (Zero Meter Level before Installation)
Initial	Oct 16 2012	11:23	2.250	2.19	Install
1	Oct 16 2012	11:26	2.500	2.43	
2	Oct 16 2012	11:33	2.500	2.53	
3	Oct 16 2012	11:37	2.500	2.31	
Average				.	



Constant Measurement (in)
Rim to Weir Lip
83.500

Pipe Diameters (in)

Pipe 1	24
Pipe 2	24
Pipe 3	NA
Pipe 4	NA

D4=Invert to Weir Lip (D3-D1)
9.000

Obvert to Weir Lip
NA

Final Check-off Sheet

CLIENT FLOW MONITORING #:	**U12-118**	**SFE PROJECT #:**	**U12-118**
NAME:	**Sweethome, Oregon**	**SFE SITE #:**	**8**
Date / Time:	Oct 16 2012		

Flow Meter Information

Meter Make:	Isco	Logging Rate:	5 Minute
Meter Model:	2150	Flow Units:	CFS
Sensor Type	AV	Velocity Units:	FPS
Meter Serial Number:	NA	Depth Units	Inches
Battery Volts:	12.2	Surcharge Meter (Y/N):	Yes

Site Physical Information

Silt Level:	0	Weather:	Sunny
Slope:	NA	Weir Size:	900
Uniform Flow (Y/N)	Y	Depth Only(DO)	NA
Debris in Flow (Y/N):	Y	or Look up Table(LT)	LT
Pipe Material:	Concrete	Comments	

Check Off List

	Yes	**No**
Time Set:	x	
Depth Calibrated:	x	
Velocity Profile:		x
Download Data:	x	
Meter Running:	x	
Pipe Size Verified:	x	
Photograph Taken:	x	
Site Cleaned:	x	
Site Secured:	x	

FIELD MAINTENANCE RECORD

NAME: Sweet Home
SFE SITE #: U12-118-8A
ADDRESS: 18th Ave & Train Tracks
GPS:
SENSOR TYPE: Av
PRIMARY DEVICE: 600 Weir

CONSTANTS			
D1 (in):	0.000	D1-lip to x-bar	
TOM (in):	93.250	Raw Weir L - x-bar to water	
METER #		DATE:	
METER #		DATE:	
METER #		DATE:	

LEGEND	
DL - DOWNLOAD	**PC** - PROGRAM COMPLETE
CB - CHG BATTERY	**PM** - PROG. METER
V - VERIFY	**VIS** - VISUAL
LA - LEVEL ADJUST	**VP** - VELOCITY PROFILE
DO - DEPTH ONLY	**CD** - CHG DESICCANT

DATE M/D/YY	TIME HH:MM	METER TIME HH:MM	METER DEPTH in	FIELD DEPTH in	METER VEL fps	FIELD VEL-VIS fps	FLOW cfs	BATT V	SILT in	Raw Weir L in	Calc Weir L in	MTC BY (INIT.)	COMMENTS
10/16/12	14:21	13:10	2.88	2.75	*	*	0.31	12.1	0			JS	install
10/18/12	11:16	10:11	2.977	2.75	*	*	0.337	11.8	0			AM	
11/01/12	10:21	9:15	4.128	4.25	*	*	0.445	11	0			AM	ragged
11/01/12	10:22	9:16	3.34	3.5	*	*	0.402	11	0			AM	Drop .75 in
11/16/12	11:47	11:42	3.946	4	*	*	0.526	10.7	0			DC	ragged, confirm pipe sizes and weir measurements
11/16/12	12:01	11:56	3.503	3.75	*	*	0.433	10.7	0			DC	Drop .25 in, confirm rating chart
11/29/12	17:15	17:19	4.88	4.75	*	*	0.712	10.6/12.4	0			DC	CB, FP
12/12/12	8:32	8:26	4.691	4.75	*	*	0.674	11.5	0			DC	DL
01/03/13	8:07	8:01	4.27	4.25	*	*	0.892	10.9	0			AM	DL
01/15/13	9:09	9:03	4.85	5	*	*	0.908	10.6	0			AM	DL clean weir all good
02/04/13	15:48	15:56	3.88	3.75	*	*	0.478	10.8	0			AM	Clean weir DL
02/20/13	14:55	14:56	3:25	3.25	*	*	0.397	10.5	0			DC	Lite Ragging .25 high
03/06/13	15:20	15:15	3.98	4.25	*	*	0.5	9.9/12.1	0			AM	DL clean weir CB
03/21/13	12:11	11:06	5.21	5	*	*	0.856	11.3	0			AM	DL CB
04/04/13	9:49	8:43	3.12	3.29	*	*	0.349	11.1	0			AM	DL CB Remove

DATE:	NOTES:

Site Assessment

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 16 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **8A**

Project Specific Information

Client Name:	Brown and Caldwell	
End User Name:	Same	
Project Name:	U12-118	
Client Contact:	Rob Lee	503-977-6625
Field Contact:	Adrian Marshall	509-312-0612
SFE PM Contact:	Paul Loving	604-992-6792

Site Equipment

Install / Remove Date:	Oct 16 2012
Meter Make & Model:	Isco 2150 AV
Level Type:	Pressure
Velocity Type:	NA
Primary Device:	SFE CCW
Wireless:	Yes
Redundancy:	Yes
Logging Rate:	5min

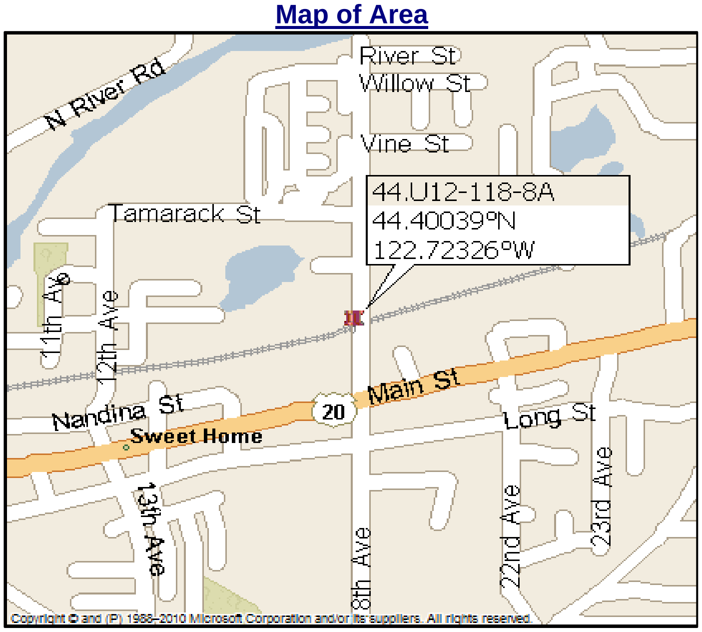
Site Location Information

Client Manhole #:	8A	
Address (Location):	18th Avenue	
City, State:	Sweethome, Oregon	
GPS (North - West):	44.40039	-122.72326
Landmarks:		
Additional Information:		

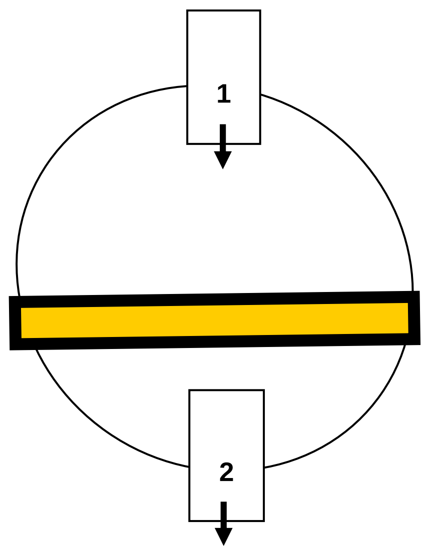
Site Profile

Pipe #1 Size:	10	Inches
Pipe #2 Size:	10	Inches
Pipe #3 Size:	NA	Inches
Pipe #4 Size:	NA	Inches
Manhole Depth:	101	Inches
Laterals / Rungs:	Yes	
Additional Information:		

Map of Area



Manhole Layout



Traffic Control Requirements

Provider:	SFE
Condition	Local only
Frequency:	Never
Speed Limit:	NA
# of Lanes Effected:	NA
Lane Configuration:	
Additional Information:	

Site Hydraulics

Date & Time:	Oct 16 2012	13:10
Depth:	3	Inches
Velocity:	1	FPS
Turbulent:	No	
Surcharge:	Possible	
Silting:	No	
Solids:	Yes	

Notes

1
2

Notes

3
4

Site Pictures

CLIENT FLOW MONITORING #: U12-118
NAME: Sweethome, Oregon
Date / Time: Oct 16 2012

SFE PROJECT #: U12-118
SFE SITE #: 8A

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Notes

1
2
3

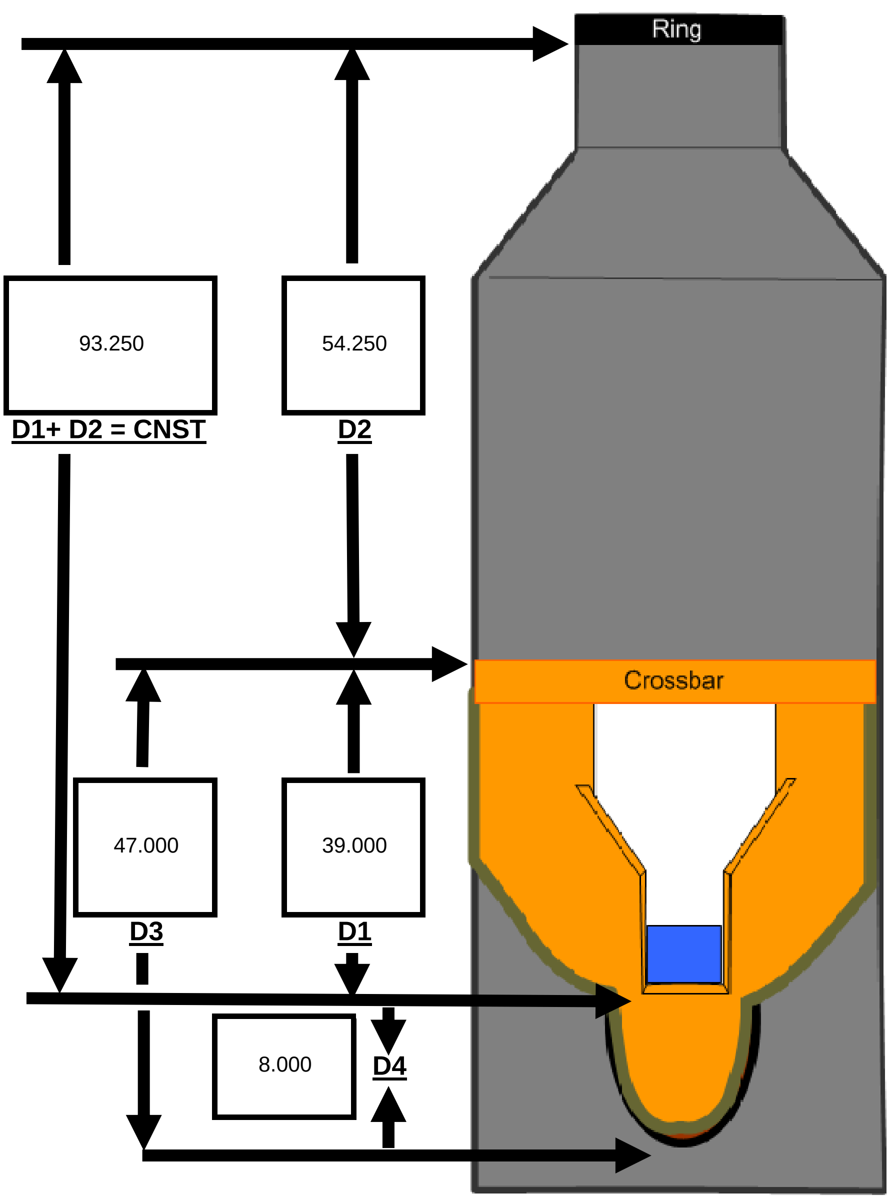
CCW Installation Form

CLIENT FLOW MONITORING #: **U12-118**
NAME: **Sweethome, Oregon**
Date / Time: Oct 16 2012

SFE PROJECT #: **U12-118**
SFE SITE #: **8A**
Technician 1: A Marshall 509-312-0612
Technician 2: Dylan Carvin

Meter Depth vs.. Field Depth Calibration / Verification

Reading Number	Date	Time	Field Meas (in.)	Meter Depth (in)	Comments (Zero Meter Level before Installation)
Initial	Oct 16 2012	14:10	3.250	3.28	Install
1	Oct 16 2012	14:13	3.000	3.04	
2	Oct 16 2012	14:16	3.000	3.10	
3	Oct 16 2012	14:20	3.000	2.96	
Average				.	



Constant Measurement (in)
Rim to Weir Lip
93.250

Pipe Diameters (in)

Pipe 1	10
Pipe 2	10
Pipe 3	NA
Pipe 4	NA

D4=Invert to Weir Lip (D3-D1)
8.000

Obvert to Weir Lip
NA

Final Check-off Sheet

CLIENT FLOW MONITORING #: U12-118
NAME: Sweethome, Oregon
Date / Time: Oct 16 2012

SFE PROJECT #: U12-118
SFE SITE #: 8A

Flow Meter Information

Meter Make:	Isco	Logging Rate:	5 Minute
Meter Model:	2150	Flow Units:	CFS
Sensor Type	AV	Velocity Units:	FPS
Meter Serial Number:	NA	Depth Units	Inches
Battery Volts:	12.2	Surcharge Meter (Y/N):	Yes

Site Physical Information

Silt Level:	0	Weather:	Sunny
Slope:	NA	Weir Size:	600
Uniform Flow (Y/N)	Y	Depth Only(DO)	NA
Debris in Flow (Y/N):	Y	or Look up Table(LT)	LT
Pipe Material:	Concrete	Comments	

Check Off List

	Yes	No
Time Set:	x	
Depth Calibrated:	x	
Velocity Profile:		x
Download Data:	x	
Meter Running:	x	
Pipe Size Verified:	x	
Photograph Taken:	x	
Site Cleaned:	x	
Site Secured:	x	

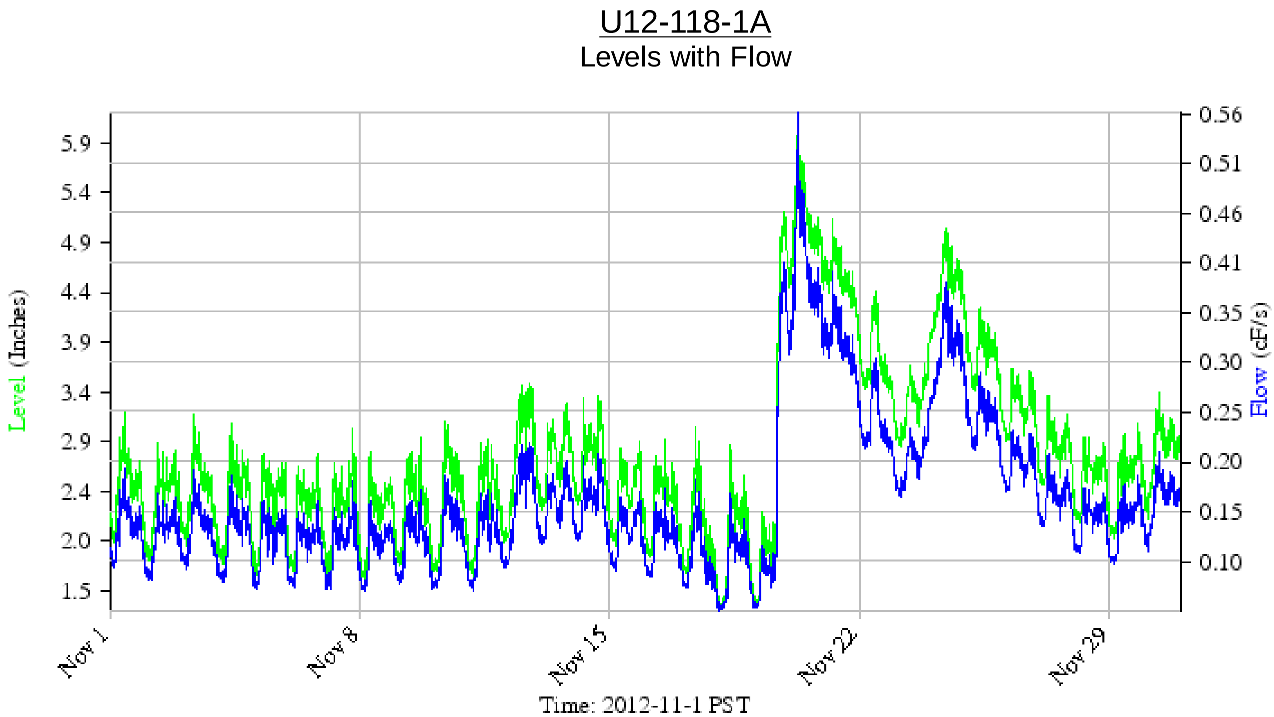
Appendix 3

Data Summaries and Graphs

Site U12-118-1A

Summary Report - November, 2012

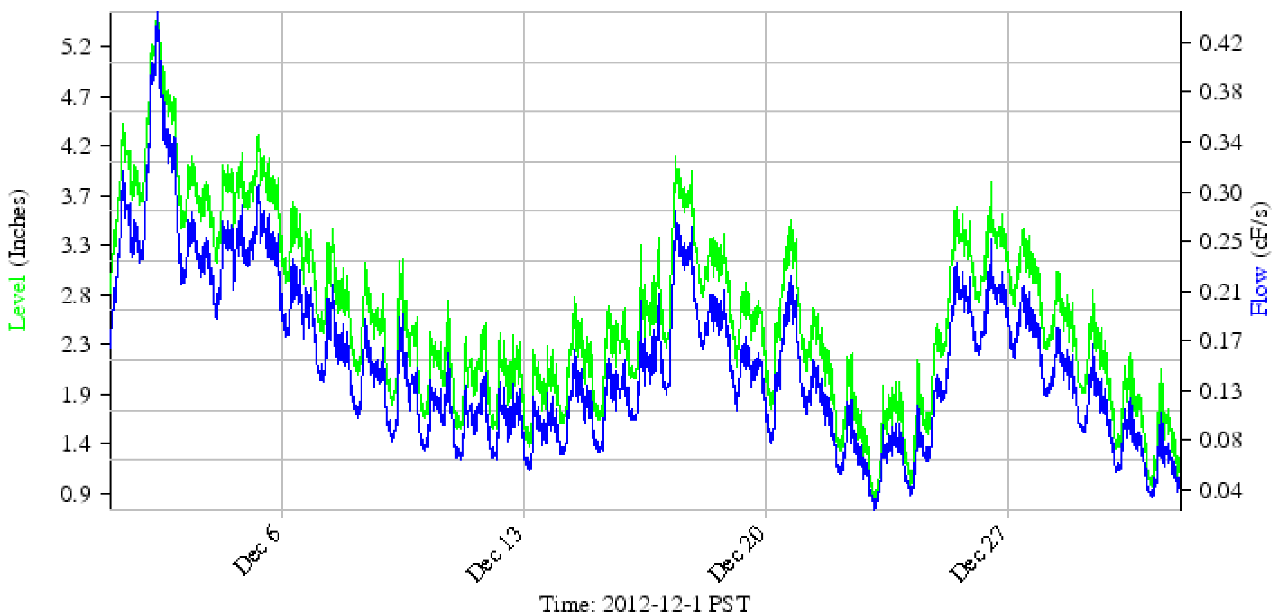
U12-118-1A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Nov	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.085	0.206	0.135	0.087
2	0.081	0.189	0.127	0.082
3	0.091	0.217	0.134	0.086
4	0.078	0.201	0.128	0.083
5	0.071	0.181	0.122	0.079
6	0.071	0.169	0.117	0.075
7	0.069	0.211	0.120	0.077
8	0.069	0.179	0.115	0.074
9	0.076	0.184	0.121	0.078
10	0.071	0.221	0.127	0.082
11	0.069	0.184	0.122	0.079
12	0.090	0.247	0.161	0.104
13	0.116	0.220	0.159	0.103
14	0.106	0.227	0.160	0.103
15	0.090	0.188	0.131	0.085
16	0.082	0.241	0.122	0.079
17	0.067	0.224	0.107	0.069
18	0.050	0.193	0.099	0.064
19	0.051	0.420	0.164	0.106
20	0.295	0.575	0.395	0.255
21	0.241	0.428	0.323	0.209
22	0.180	0.333	0.240	0.155
23	0.161	0.274	0.206	0.133
24	0.236	0.425	0.315	0.204
25	0.188	0.304	0.245	0.158
26	0.158	0.255	0.200	0.129
27	0.123	0.221	0.165	0.107
28	0.104	0.199	0.143	0.092
29	0.096	0.194	0.141	0.091
30	0.111	0.231	0.163	0.106
Mean	**0.113**	**0.251**	**0.167**	**0.108**
Maximum	**0.295**	**0.575**	**0.395**	**0.255**
Minimum	**0.050**	**0.169**	**0.099**	**0.064**
Total Flow (mg)		**3.234**		


U12-118-1A
Levels with Flow
Level (Inches)
5.9
5.4
4.9
4.4
3.9
3.4
2.9
2.4
2.0
1.5
Flow (cF/s)
0.56
0.51
0.46
0.41
0.35
0.30
0.25
0.20
0.15
0.10
Nov 1
Nov 8
Nov 15
Nov 22
Nov 29
Time: 2012-11-1 PST

Summary Report - December, 2012

U12-118-1A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Dec	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.153	0.348	0.247	0.160
2	0.241	0.470	0.350	0.227
3	0.197	0.293	0.241	0.156
4	0.174	0.312	0.241	0.156
5	0.191	0.329	0.252	0.163
6	0.148	0.268	0.197	0.127
7	0.121	0.237	0.162	0.105
8	0.100	0.208	0.138	0.089
9	0.079	0.213	0.125	0.081
10	0.073	0.162	0.106	0.069
11	0.065	0.146	0.101	0.065
12	0.063	0.150	0.099	0.064
13	0.056	0.150	0.091	0.059
14	0.069	0.177	0.112	0.073
15	0.069	0.179	0.114	0.074
16	0.097	0.230	0.143	0.092
17	0.120	0.294	0.212	0.137
18	0.139	0.236	0.183	0.118
19	0.101	0.192	0.142	0.092
20	0.079	0.236	0.153	0.099
21	0.072	0.176	0.120	0.077
22	0.036	0.136	0.074	0.048
23	0.023	0.120	0.066	0.042
24	0.033	0.156	0.077	0.050
25	0.106	0.254	0.182	0.118
26	0.145	0.262	0.196	0.127
27	0.128	0.242	0.180	0.116
28	0.109	0.194	0.142	0.092
29	0.077	0.167	0.114	0.074
30	0.048	0.137	0.082	0.053
31	0.034	0.130	0.059	0.038
Mean	**0.101**	**0.219**	**0.152**	**0.098**
Maximum	**0.241**	**0.470**	**0.350**	**0.227**
Minimum	**0.023**	**0.120**	**0.059**	**0.038**
Total Flow (mg)		**3.039**		

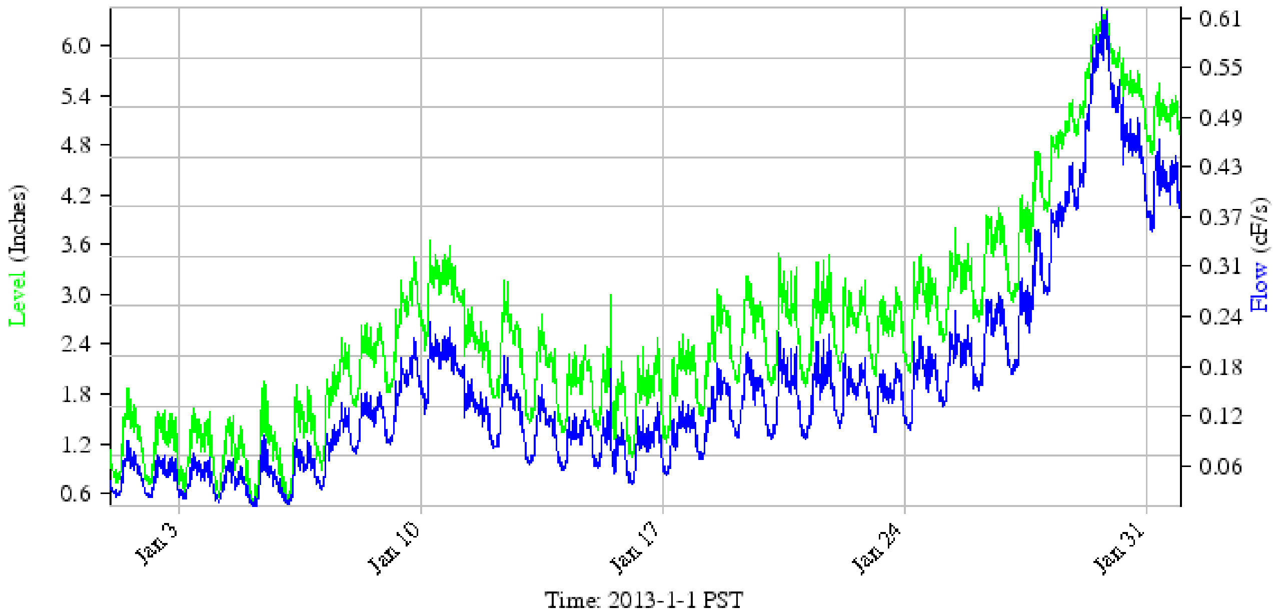
U12-118-1A
Levels with Flow



Summary Report - January, 2013

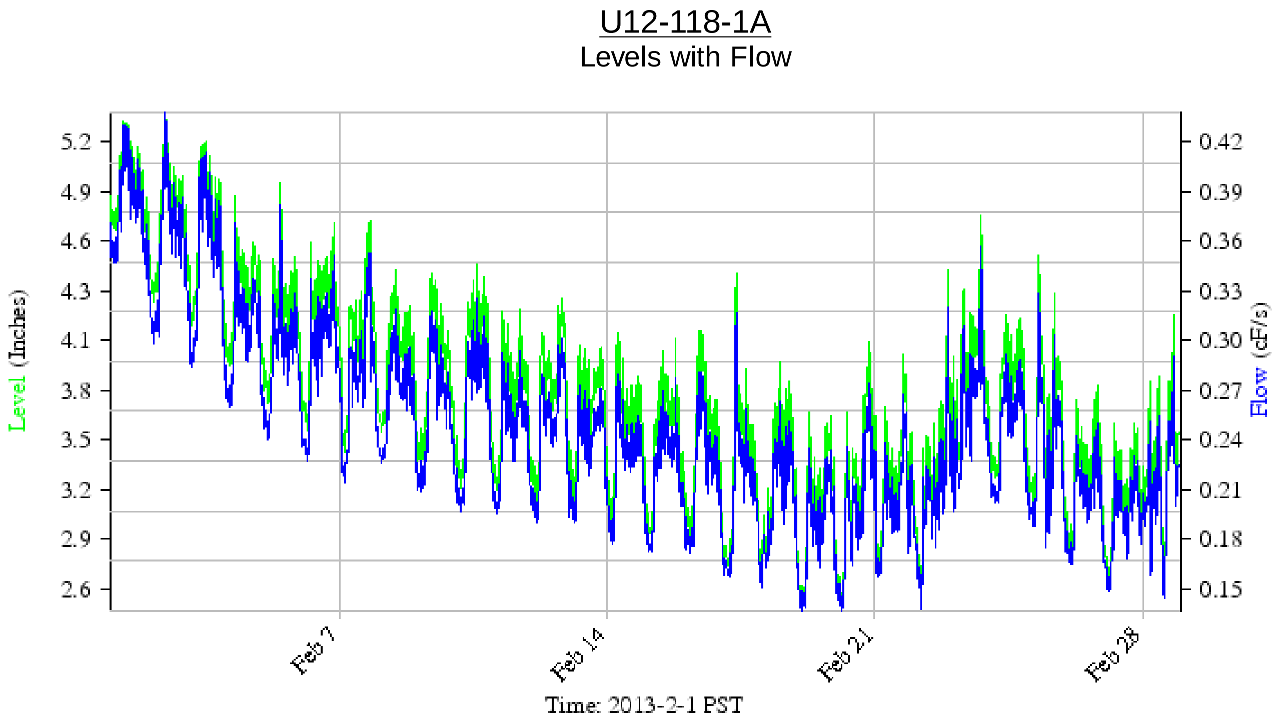
U12-118-1A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Jan	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.021	0.118	0.050	0.032
2	0.020	0.090	0.047	0.031
3	0.018	0.085	0.046	0.030
4	0.015	0.076	0.041	0.026
5	0.009	0.125	0.041	0.027
6	0.013	0.108	0.052	0.033
7	0.029	0.150	0.086	0.056
8	0.075	0.172	0.118	0.076
9	0.085	0.244	0.153	0.099
10	0.123	0.253	0.188	0.122
11	0.101	0.203	0.146	0.095
12	0.081	0.219	0.127	0.082
13	0.061	0.185	0.101	0.066
14	0.053	0.149	0.093	0.060
15	0.045	0.197	0.085	0.055
16	0.036	0.148	0.085	0.055
17	0.047	0.184	0.096	0.062
18	0.065	0.227	0.127	0.082
19	0.093	0.222	0.146	0.094
20	0.091	0.230	0.149	0.096
21	0.091	0.243	0.149	0.097
22	0.105	0.213	0.153	0.099
23	0.101	0.195	0.147	0.095
24	0.104	0.237	0.165	0.107
25	0.130	0.277	0.188	0.121
26	0.149	0.285	0.218	0.141
27	0.170	0.369	0.251	0.162
28	0.261	0.455	0.357	0.231
29	0.368	0.642	0.518	0.335
30	0.377	0.560	0.469	0.303
31	0.327	0.490	0.403	0.260
Mean	**0.105**	**0.237**	**0.161**	**0.104**
Maximum	**0.377**	**0.642**	**0.518**	**0.335**
Minimum	**0.009**	**0.076**	**0.041**	**0.026**
Total Flow (mg)		**3.229**		

U12-118-1A
Levels with Flow



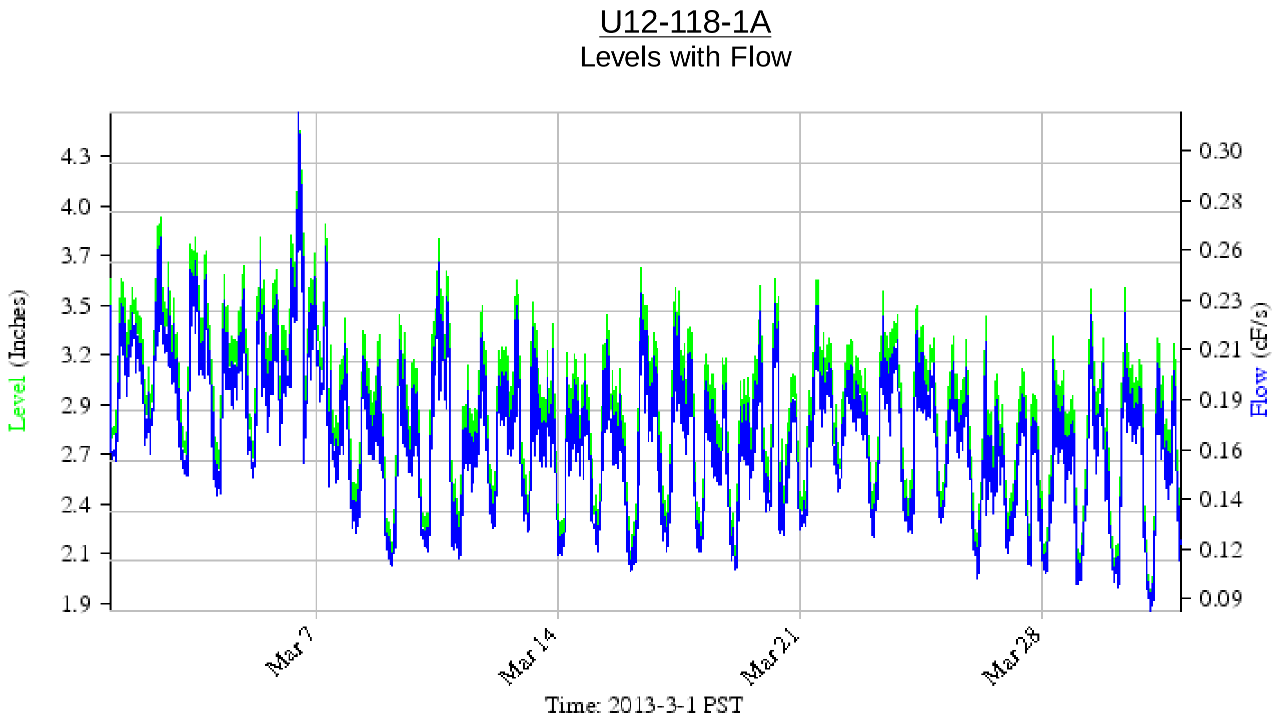
Summary Report - February, 2013

U12-118-1A	Flow (cF/s)			
	Min	Max	Avg	Total
Feb	cF/s	cF/s	cF/s	mg/d
1	0.324	0.459	0.378	0.245
2	0.292	0.454	0.355	0.229
3	0.274	0.445	0.343	0.222
4	0.254	0.381	0.301	0.194
5	0.236	0.411	0.288	0.186
6	0.217	0.374	0.287	0.186
7	0.211	0.370	0.274	0.177
8	0.220	0.331	0.265	0.171
9	0.203	0.339	0.256	0.165
10	0.188	0.359	0.256	0.165
11	0.190	0.317	0.241	0.156
12	0.187	0.312	0.245	0.158
13	0.181	0.284	0.234	0.151
14	0.174	0.289	0.224	0.145
15	0.169	0.280	0.220	0.142
16	0.165	0.297	0.219	0.141
17	0.151	0.329	0.206	0.133
18	0.146	0.275	0.203	0.131
19	0.128	0.246	0.184	0.119
20	0.134	0.282	0.195	0.126
21	0.152	0.289	0.200	0.129
22	0.131	0.329	0.202	0.130
23	0.189	0.379	0.257	0.166
24	0.198	0.312	0.243	0.157
25	0.156	0.332	0.229	0.148
26	0.158	0.261	0.202	0.131
27	0.144	0.241	0.191	0.123
28	0.141	0.314	0.211	0.136
Mean	**0.190**	**0.332**	**0.247**	**0.159**
Maximum	**0.324**	**0.459**	**0.378**	**0.245**
Minimum	**0.128**	**0.241**	**0.184**	**0.119**
Total Flow (mg)		**4.464**		


U12-118-1A
Levels with Flow
Level (Inches)
Flow (cF/s)
5.2
4.9
4.6
4.3
4.1
3.8
3.5
3.2
2.9
2.6
0.42
0.39
0.36
0.33
0.30
0.27
0.24
0.21
0.18
0.15
Feb 7
Feb 14
Feb 21
Feb 28
Time: 2013-2-1 PST

Summary Report - March, 2013

U12-118-1A	Flow (cF/s)			
	Min	Max	Avg	Total
Mar	cF/s	cF/s	cF/s	mg/d
1	0.150	0.244	0.200	0.129
2	0.157	0.286	0.205	0.132
3	0.140	0.257	0.200	0.129
4	0.140	0.244	0.190	0.123
5	0.148	0.263	0.195	0.126
6	0.149	0.329	0.221	0.143
7	0.131	0.273	0.181	0.117
8	0.117	0.225	0.162	0.105
9	0.108	0.228	0.157	0.102
10	0.111	0.259	0.174	0.112
11	0.100	0.228	0.160	0.104
12	0.124	0.267	0.172	0.111
13	0.117	0.227	0.164	0.106
14	0.111	0.218	0.159	0.103
15	0.109	0.228	0.159	0.103
16	0.103	0.253	0.166	0.107
17	0.111	0.237	0.168	0.109
18	0.113	0.210	0.158	0.102
19	0.106	0.231	0.160	0.103
20	0.121	0.245	0.162	0.104
21	0.123	0.241	0.177	0.114
22	0.138	0.218	0.175	0.113
23	0.120	0.231	0.177	0.114
24	0.118	0.230	0.172	0.111
25	0.120	0.211	0.166	0.107
26	0.101	0.219	0.147	0.095
27	0.096	0.220	0.147	0.095
28	0.098	0.278	0.151	0.098
29	0.096	0.261	0.155	0.100
30	0.093	0.237	0.158	0.102
31	0.085	0.216	0.146	0.094
Mean	**0.118**	**0.242**	**0.170**	**0.110**
Maximum	**0.157**	**0.329**	**0.221**	**0.143**
Minimum	**0.085**	**0.210**	**0.146**	**0.094**
Total Flow (mg)		**3.414**		

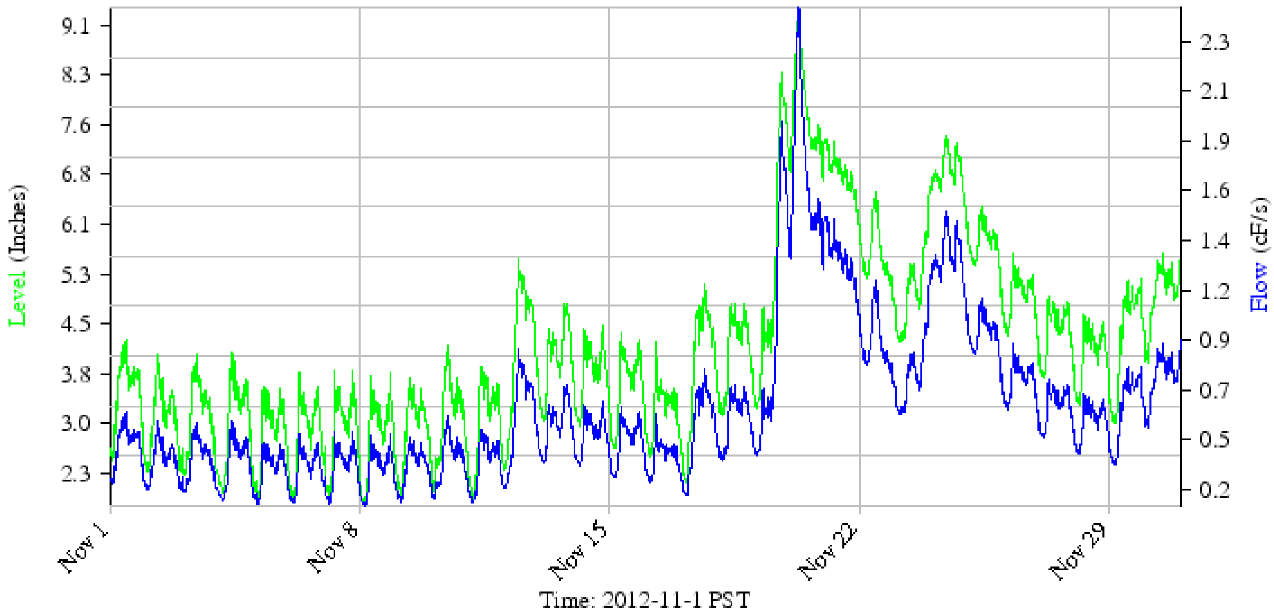

U12-118-1A
Levels with Flow
Level (Inches)
Flow (cF/s)
4.3
4.0
3.7
3.5
3.2
2.9
2.7
2.4
2.1
1.9
0.30
0.28
0.26
0.23
0.21
0.19
0.16
0.14
0.12
0.09
Mar 7
Mar 14
Mar 21
Mar 28
Time: 2013-3-1 PST

Site U12-118-2

Summary Report - November, 2012

U12-118-2	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Nov	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.242	0.609	0.427	0.276
2	0.229	0.562	0.367	0.237
3	0.218	0.566	0.368	0.238
4	0.177	0.568	0.362	0.234
5	0.159	0.471	0.323	0.209
6	0.169	0.511	0.326	0.211
7	0.163	0.533	0.341	0.220
8	0.151	0.519	0.327	0.212
9	0.173	0.499	0.333	0.215
10	0.178	0.614	0.368	0.238
11	0.168	0.554	0.353	0.228
12	0.227	0.928	0.577	0.373
13	0.352	0.749	0.538	0.348
14	0.340	0.652	0.494	0.320
15	0.282	0.676	0.457	0.295
16	0.259	0.609	0.387	0.250
17	0.200	0.805	0.518	0.335
18	0.362	0.743	0.560	0.362
19	0.383	2.062	0.918	0.594
20	1.251	2.511	1.709	1.104
21	1.040	1.547	1.308	0.845
22	0.696	1.235	0.912	0.590
23	0.577	1.266	0.785	0.508
24	1.032	1.566	1.324	0.855
25	0.715	1.162	0.962	0.622
26	0.591	0.940	0.742	0.479
27	0.470	0.753	0.620	0.401
28	0.398	0.706	0.553	0.357
29	0.345	0.829	0.616	0.398
30	0.508	0.955	0.758	0.490
Mean	**0.402**	**0.890**	**0.621**	**0.401**
Maximum	**1.251**	**2.511**	**1.709**	**1.104**
Minimum	**0.151**	**0.471**	**0.323**	**0.209**
Total Flow (mg)		**12.044**		

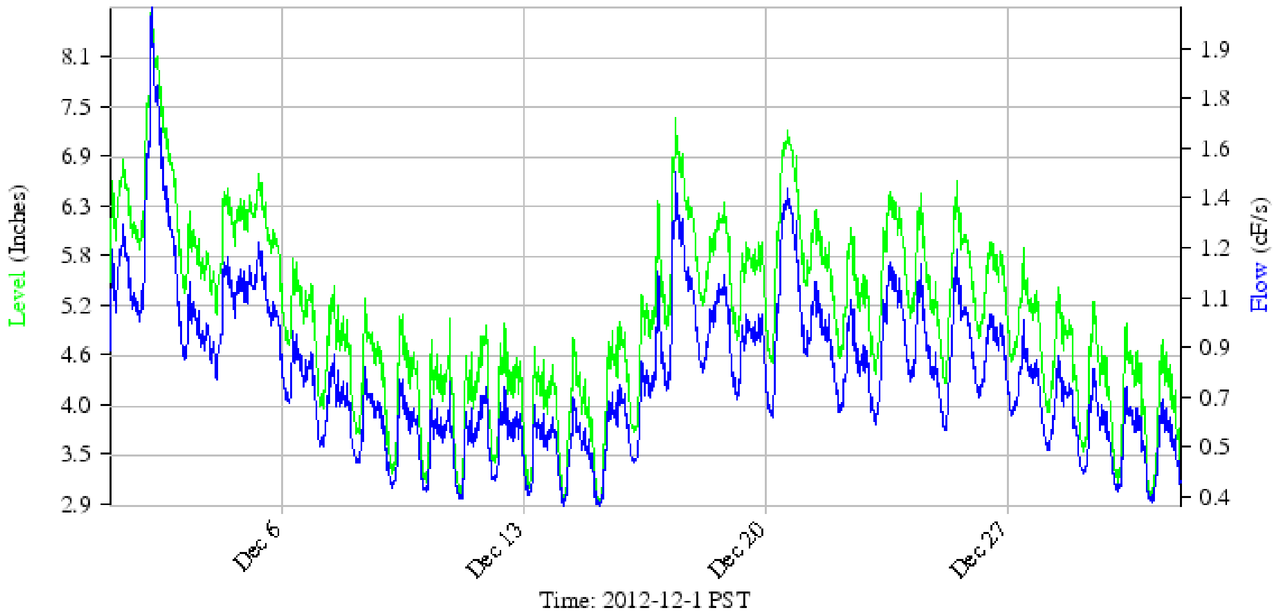
U12-118-2
Levels with Flow



Summary Report - December, 2012

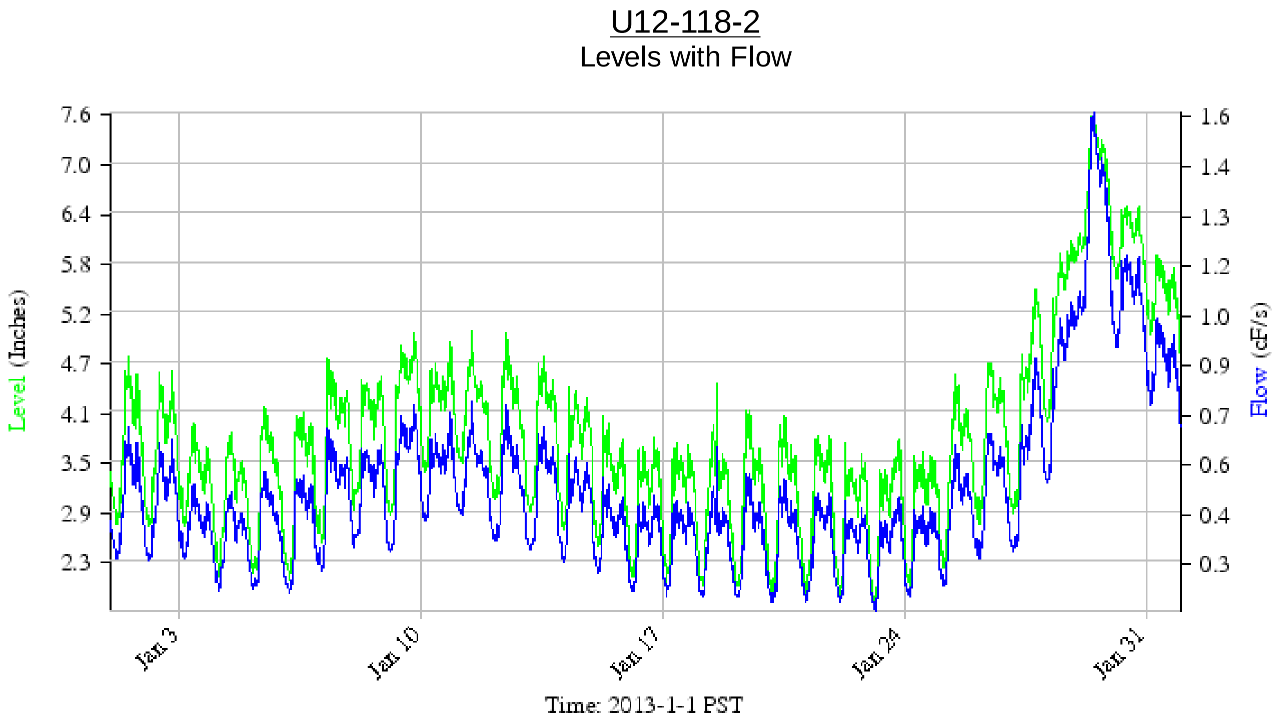
U12-118-2	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Dec	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.851	1.354	1.111	0.718
2	0.996	2.137	1.509	0.975
3	0.818	1.152	0.934	0.604
4	0.767	1.219	1.043	0.674
5	0.780	1.296	1.067	0.690
6	0.640	0.966	0.790	0.511
7	0.522	0.881	0.681	0.440
8	0.466	0.838	0.620	0.400
9	0.385	0.791	0.573	0.370
10	0.356	0.810	0.543	0.351
11	0.333	0.774	0.552	0.357
12	0.403	0.780	0.563	0.364
13	0.350	0.700	0.526	0.340
14	0.319	0.715	0.507	0.328
15	0.316	0.766	0.556	0.359
16	0.471	1.197	0.722	0.467
17	0.718	1.541	1.093	0.707
18	0.791	1.166	0.967	0.625
19	0.695	1.049	0.881	0.569
20	0.628	1.495	1.099	0.710
21	0.766	1.168	0.947	0.612
22	0.641	1.095	0.825	0.533
23	0.600	1.225	0.935	0.604
24	0.732	1.217	0.887	0.573
25	0.579	1.235	0.894	0.578
26	0.685	1.050	0.860	0.556
27	0.620	1.018	0.768	0.496
28	0.514	0.870	0.666	0.430
29	0.432	0.851	0.593	0.383
30	0.362	0.768	0.546	0.353
31	0.328	0.738	0.503	0.325
Mean	**0.576**	**1.060**	**0.799**	**0.516**
Maximum	**0.996**	**2.137**	**1.509**	**0.975**
Minimum	**0.316**	**0.700**	**0.503**	**0.325**
Total Flow (mg)		**16.003**		

U12-118-2
Levels with Flow



Summary Report - January, 2013

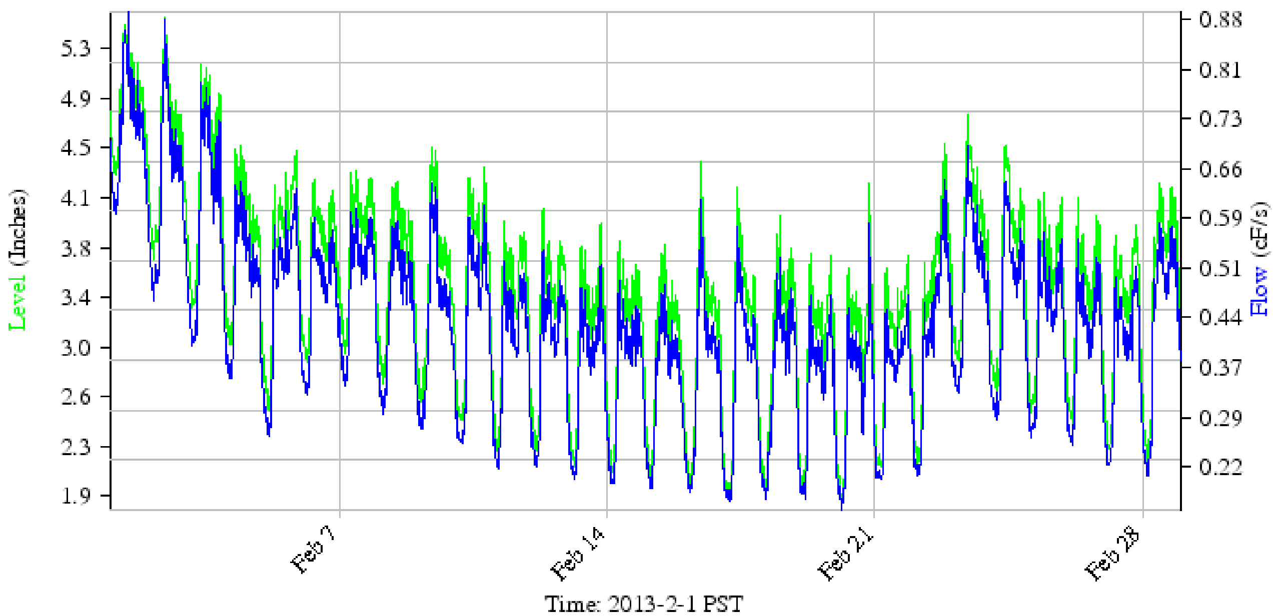
U12-118-2	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Jan	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.305	0.730	0.483	0.312
2	0.299	0.677	0.473	0.306
3	0.297	0.581	0.406	0.262
4	0.209	0.517	0.366	0.237
5	0.214	0.603	0.387	0.250
6	0.205	0.573	0.396	0.256
7	0.271	0.727	0.501	0.324
8	0.329	0.670	0.515	0.333
9	0.319	0.783	0.570	0.368
10	0.410	0.753	0.572	0.370
11	0.432	0.791	0.565	0.365
12	0.355	0.810	0.531	0.343
13	0.328	0.717	0.510	0.330
14	0.297	0.672	0.470	0.304
15	0.267	0.581	0.398	0.257
16	0.207	0.517	0.366	0.237
17	0.185	0.533	0.361	0.233
18	0.193	0.834	0.362	0.234
19	0.192	0.597	0.370	0.239
20	0.176	0.568	0.358	0.232
21	0.175	0.523	0.356	0.230
22	0.170	0.488	0.338	0.219
23	0.155	0.512	0.339	0.219
24	0.194	0.484	0.357	0.231
25	0.222	0.666	0.444	0.287
26	0.297	0.699	0.510	0.330
27	0.315	0.910	0.596	0.385
28	0.520	1.086	0.831	0.537
29	0.996	1.633	1.304	0.843
30	0.900	1.214	1.062	0.687
31	0.674	1.031	0.872	0.564
Mean	**0.326**	**0.725**	**0.515**	**0.333**
Maximum	**0.996**	**1.633**	**1.304**	**0.843**
Minimum	**0.155**	**0.484**	**0.338**	**0.219**
Total Flow (mg)	**10.322**			


U12-118-2
Levels with Flow
7.6
7.0
6.4
5.8
5.2
4.7
4.1
3.5
2.9
2.3
Level (Inches)
1.6
1.4
1.3
1.2
1.0
0.9
0.7
0.6
0.4
0.3
Flow (cF/s)
Jan 3
Jan 10
Jan 17
Jan 24
Jan 31
Time: 2013-1-1 PST

Summary Report - February, 2013

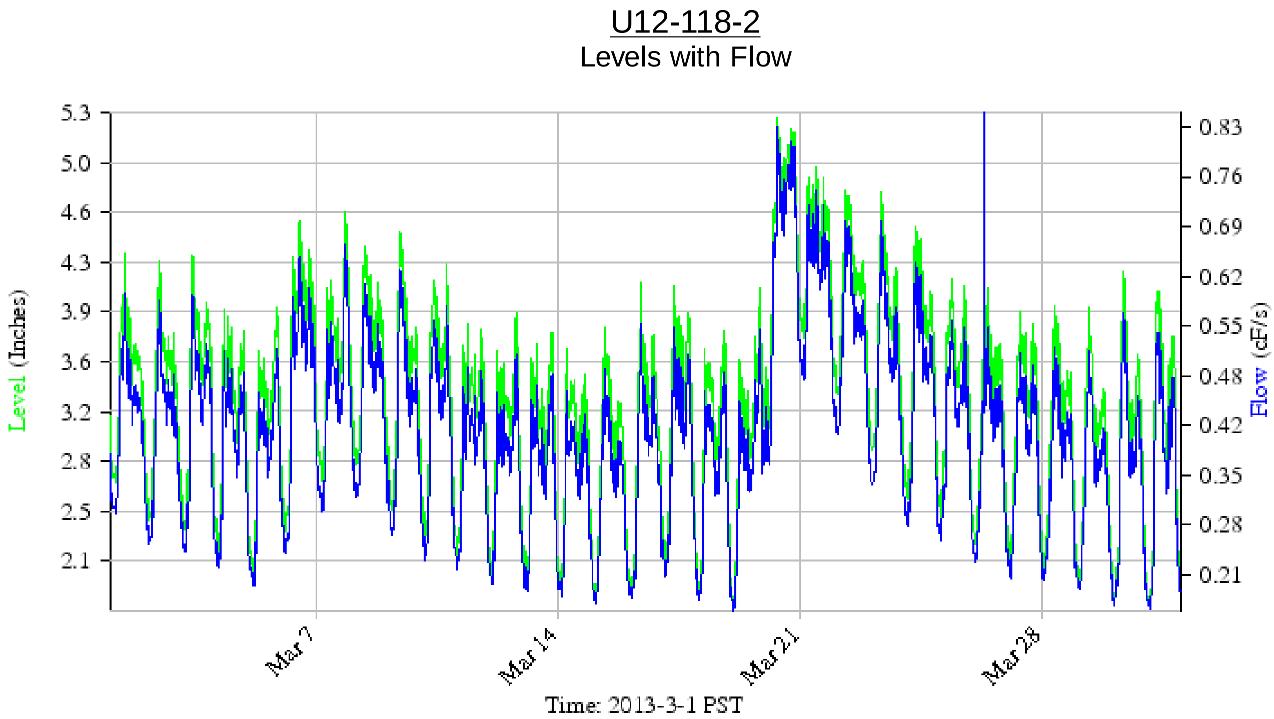
U12-118-2	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Feb	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.583	0.927	0.719	0.465
2	0.464	0.922	0.638	0.412
3	0.398	0.804	0.592	0.382
4	0.344	0.669	0.495	0.320
5	0.261	0.651	0.471	0.304
6	0.322	0.610	0.470	0.304
7	0.334	0.622	0.493	0.319
8	0.293	0.618	0.459	0.297
9	0.274	0.658	0.441	0.285
10	0.241	0.620	0.433	0.280
11	0.214	0.555	0.392	0.253
12	0.218	0.570	0.384	0.248
13	0.194	0.551	0.367	0.237
14	0.191	0.520	0.360	0.233
15	0.185	0.508	0.347	0.225
16	0.177	0.624	0.358	0.231
17	0.169	0.592	0.349	0.226
18	0.165	0.569	0.354	0.229
19	0.169	0.493	0.335	0.217
20	0.153	0.588	0.334	0.216
21	0.198	0.491	0.345	0.223
22	0.204	0.692	0.411	0.266
23	0.323	0.725	0.493	0.318
24	0.288	0.661	0.456	0.295
25	0.262	0.587	0.427	0.276
26	0.247	0.577	0.402	0.260
27	0.219	0.537	0.382	0.247
28	0.199	0.605	0.436	0.282
Mean	**0.260**	**0.627**	**0.434**	**0.280**
Maximum	**0.583**	**0.927**	**0.719**	**0.465**
Minimum	**0.153**	**0.491**	**0.334**	**0.216**
Total Flow (mg)	**7.848**			

U12-118-2
Levels with Flow



Summary Report - March, 2013

U12-118-2	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Mar	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.290	0.629	0.426	0.276
2	0.247	0.628	0.406	0.262
3	0.233	0.616	0.413	0.267
4	0.213	0.545	0.380	0.245
5	0.186	0.543	0.360	0.233
6	0.228	0.665	0.473	0.306
7	0.292	0.673	0.457	0.295
8	0.311	0.657	0.460	0.297
9	0.260	0.655	0.427	0.276
10	0.219	0.586	0.403	0.261
11	0.206	0.513	0.365	0.236
12	0.180	0.541	0.351	0.227
13	0.183	0.513	0.342	0.221
14	0.175	0.491	0.331	0.214
15	0.164	0.516	0.329	0.212
16	0.170	0.587	0.353	0.228
17	0.196	0.560	0.371	0.240
18	0.167	0.504	0.342	0.221
19	0.155	0.561	0.360	0.233
20	0.354	0.844	0.656	0.424
21	0.424	0.762	0.606	0.392
22	0.357	0.736	0.539	0.349
23	0.310	0.716	0.482	0.312
24	0.262	0.694	0.450	0.291
25	0.233	0.598	0.412	0.266
26	0.222	0.982	0.387	0.250
27	0.202	0.526	0.372	0.240
28	0.195	0.532	0.358	0.231
29	0.184	0.528	0.340	0.220
30	0.162	0.593	0.347	0.224
31	0.157	0.571	0.349	0.225
Mean	**0.230**	**0.615**	**0.408**	**0.264**
Maximum	**0.424**	**0.982**	**0.656**	**0.424**
Minimum	**0.155**	**0.491**	**0.329**	**0.212**
Total Flow (mg)		**8.174**		

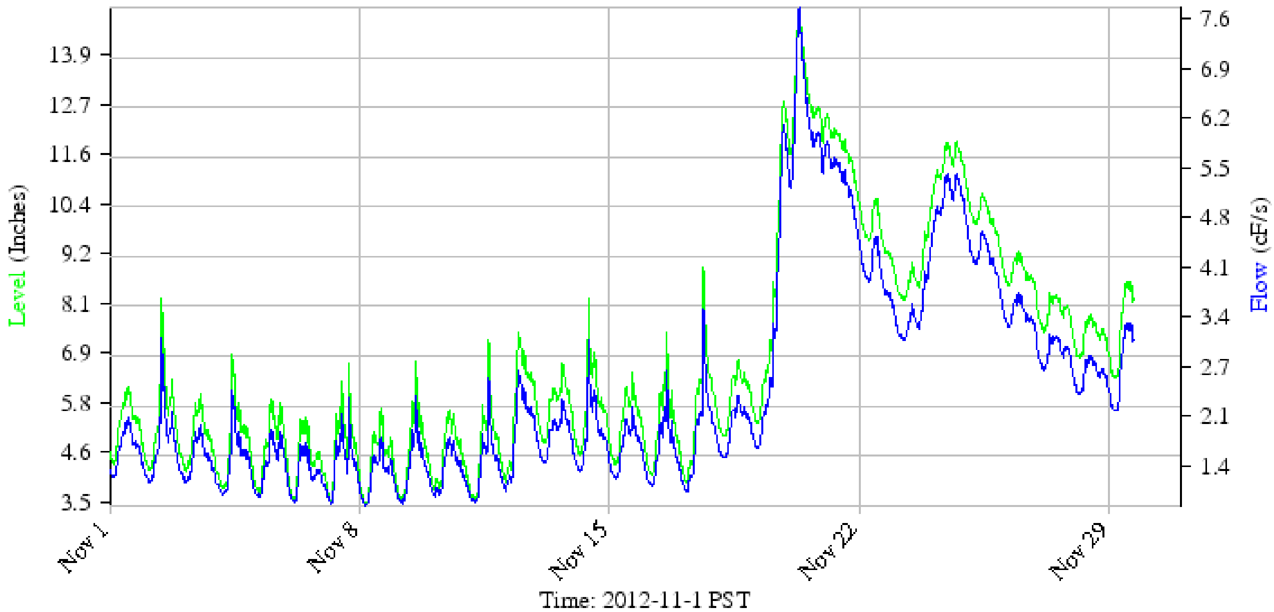

U12-118-2
Levels with Flow
Level (Inches)
Flow (cF/s)
5.3
5.0
4.6
4.3
3.9
3.6
3.2
2.8
2.5
2.1
0.83
0.76
0.69
0.62
0.55
0.48
0.42
0.35
0.28
0.21
Mar 7
Mar 14
Mar 21
Mar 28
Time: 2013-3-1 PST

Site U12-118-3

Summary Report - November, 2012

U12-118-3	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Nov	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	1.207	2.051	1.601	1.035
2	1.141	3.233	1.670	1.079
3	1.102	1.978	1.460	0.944
4	0.975	2.455	1.476	0.954
5	0.932	1.890	1.396	0.902
6	0.866	1.752	1.260	0.814
7	0.859	2.357	1.395	0.902
8	0.827	1.813	1.261	0.815
9	0.849	2.399	1.339	0.866
10	0.945	1.781	1.322	0.854
11	0.875	2.648	1.386	0.896
12	1.030	2.724	1.853	1.198
13	1.408	2.320	1.813	1.171
14	1.294	3.199	1.808	1.168
15	1.195	2.224	1.628	1.052
16	1.097	2.716	1.591	1.028
17	1.022	3.556	1.810	1.170
18	1.485	2.352	1.906	1.232
19	1.625	6.116	3.162	2.043
20	5.193	7.740	6.203	4.009
21	4.473	5.872	5.337	3.449
22	3.469	4.615	4.022	2.600
23	3.099	4.353	3.474	2.245
24	4.382	5.435	5.056	3.268
25	3.598	4.782	4.272	2.761
26	2.965	3.793	3.463	2.238
27	2.611	3.227	2.945	1.903
28	2.367	2.919	2.636	1.704
29	2.127	3.471	2.732	1.766
30	na	na	na	na
Mean	**1.897**	**3.302**	**2.458**	**1.589**
Maximum	**5.193**	**7.740**	**6.203**	**4.009**
Minimum	**0.827**	**1.752**	**1.260**	**0.814**
Total Flow (mg)	**46.067**			

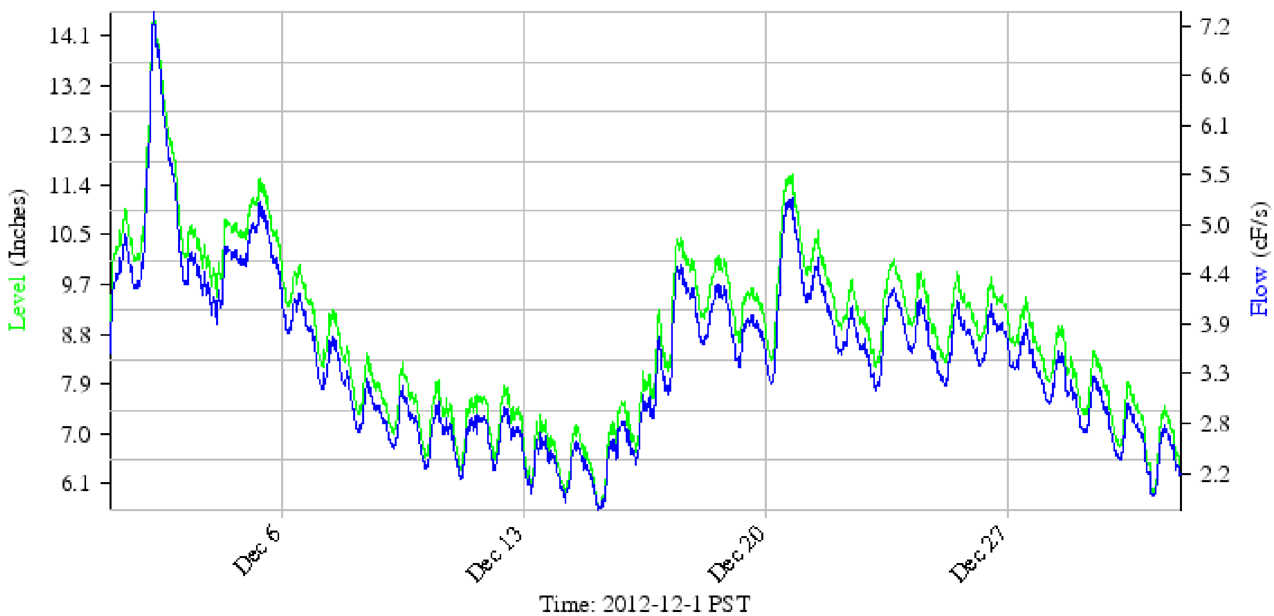
U12-118-3
Levels with Flow



Summary Report - December, 2012

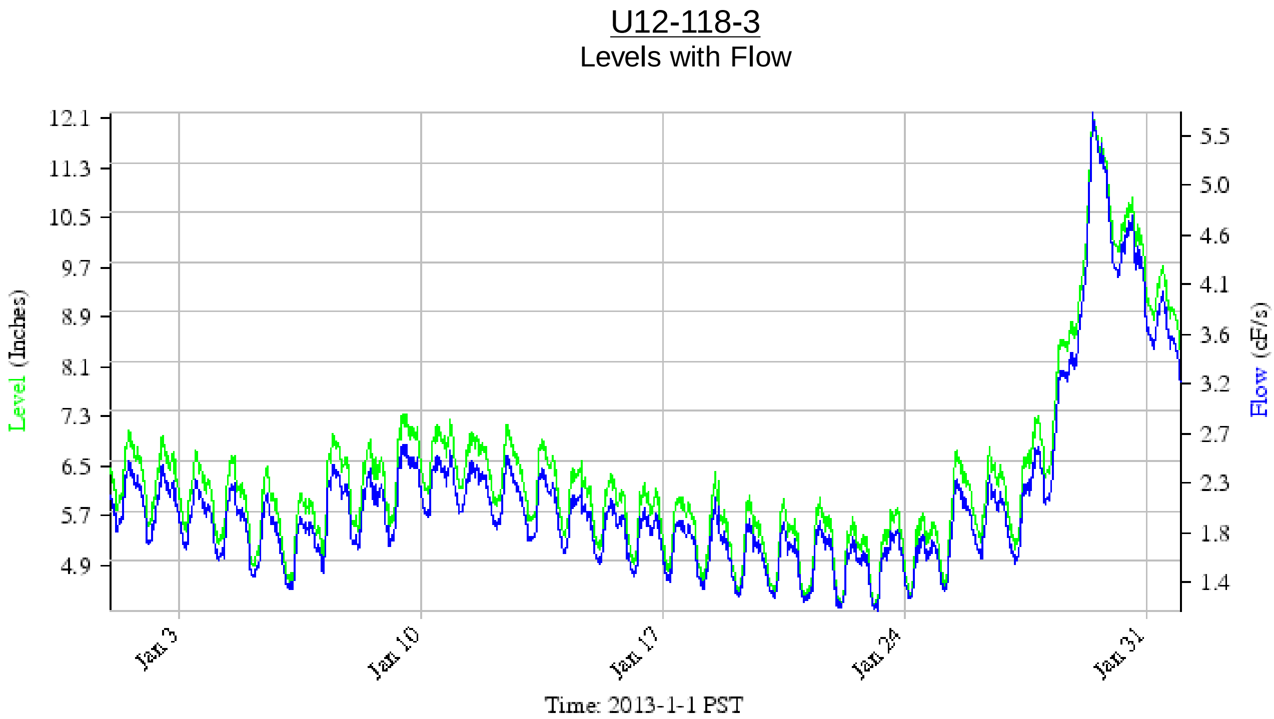
U12-118-3	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Dec	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	3.517	4.881	4.408	2.849
2	4.734	7.372	6.047	3.908
3	3.946	4.783	4.391	2.838
4	3.839	4.759	4.462	2.884
5	4.081	5.294	4.797	3.100
6	3.389	4.253	3.901	2.521
7	3.003	3.769	3.357	2.170
8	2.655	3.294	2.900	1.874
9	2.469	3.215	2.774	1.793
10	2.257	3.024	2.624	1.696
11	2.124	2.873	2.593	1.676
12	2.226	2.964	2.612	1.688
13	1.979	2.679	2.344	1.515
14	1.869	2.591	2.242	1.449
15	1.794	2.794	2.314	1.495
16	2.285	3.790	2.870	1.855
17	3.079	4.549	3.956	2.557
18	3.626	4.340	3.988	2.577
19	3.358	3.978	3.734	2.414
20	3.197	5.291	4.370	2.825
21	3.789	4.610	4.214	2.723
22	3.381	4.110	3.686	2.382
23	3.103	4.286	3.759	2.430
24	3.283	4.167	3.717	2.402
25	3.167	4.142	3.648	2.358
26	3.389	4.096	3.720	2.404
27	3.185	3.869	3.492	2.257
28	2.962	3.569	3.200	2.068
29	2.654	3.324	2.925	1.891
30	2.231	3.014	2.636	1.704
31	1.949	2.752	2.348	1.518
Mean	**2.985**	**3.949**	**3.485**	**2.252**
Maximum	**4.734**	**7.372**	**6.047**	**3.908**
Minimum	**1.794**	**2.591**	**2.242**	**1.449**
Total Flow (mg)		**69.821**		

U12-118-3
Levels with Flow



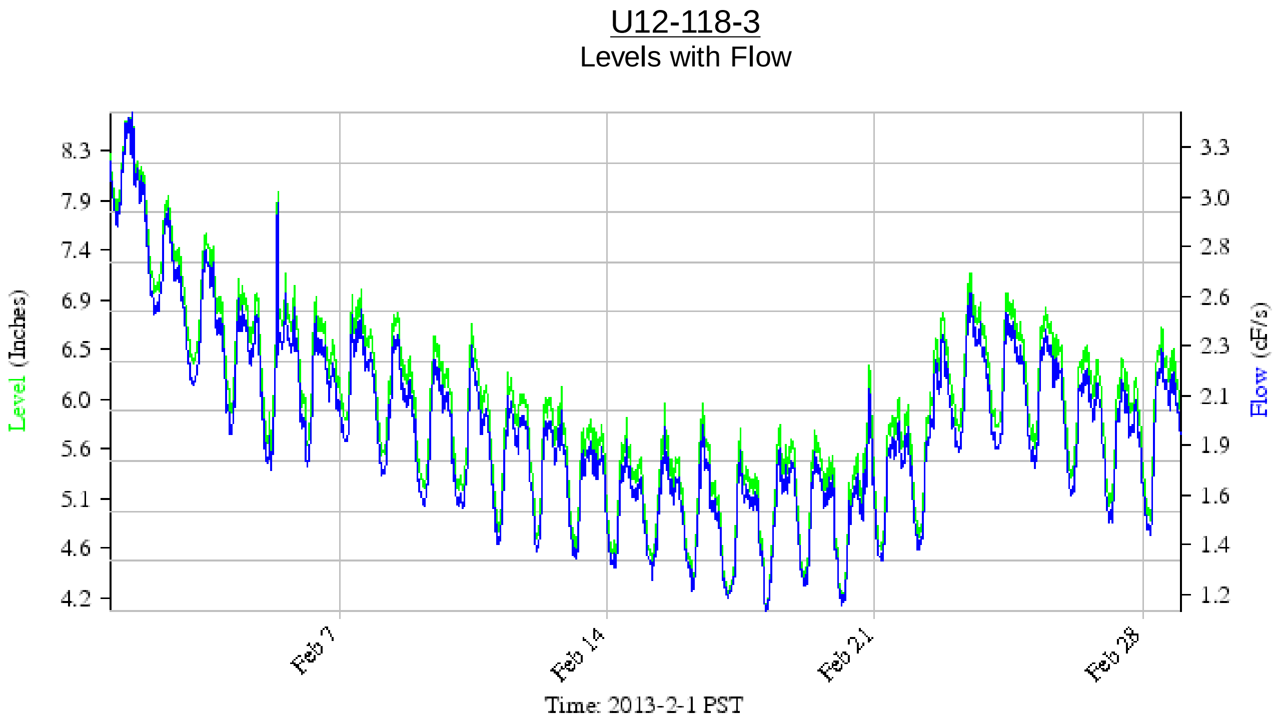
Summary Report - January, 2013

U12-118-3	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Jan	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	1.833	2.489	2.175	1.406
2	1.708	2.484	2.099	1.357
3	1.671	2.341	1.999	1.292
4	1.576	2.283	1.915	1.238
5	1.412	2.199	1.771	1.145
6	1.301	1.990	1.676	1.083
7	1.456	2.475	2.070	1.338
8	1.668	2.448	2.065	1.335
9	1.705	2.656	2.268	1.466
10	1.930	2.590	2.305	1.490
11	1.993	2.542	2.265	1.464
12	1.851	2.554	2.175	1.406
13	1.750	2.422	2.080	1.345
14	1.635	2.240	1.954	1.263
15	1.523	2.160	1.855	1.199
16	1.427	2.069	1.783	1.152
17	1.379	1.972	1.715	1.108
18	1.260	2.156	1.671	1.080
19	1.231	2.013	1.578	1.020
20	1.215	1.944	1.556	1.006
21	1.196	1.956	1.552	1.003
22	1.123	1.794	1.490	0.963
23	1.044	1.871	1.544	0.998
24	1.213	1.819	1.566	1.012
25	1.279	2.341	1.863	1.204
26	1.545	2.400	1.973	1.275
27	1.530	2.639	2.089	1.350
28	2.077	3.498	2.901	1.875
29	3.424	5.694	4.790	3.096
30	3.806	4.767	4.367	2.822
31	3.198	4.064	3.669	2.371
Mean	**1.708**	**2.544**	**2.154**	**1.392**
Maximum	**3.806**	**5.694**	**4.790**	**3.096**
Minimum	**1.044**	**1.794**	**1.490**	**0.963**
Total Flow (mg)		**43.161**		


U12-118-3
Levels with Flow
Level (Inches)
Flow (cF/s)
12.1
11.3
10.5
9.7
8.9
8.1
7.3
6.5
5.7
4.9
5.5
5.0
4.6
4.1
3.6
3.2
2.7
2.3
1.8
1.4
Jan 3
Jan 10
Jan 17
Jan 24
Jan 31
Time: 2013-1-1 PST

Summary Report - February, 2013

U12-118-3	Flow (cF/s)			
	Min	Max	Avg	Total
Feb	cF/s	cF/s	cF/s	mg/d
1	2.773	3.518	3.098	2.002
2	2.364	2.979	2.654	1.715
3	2.128	2.782	2.429	1.570
4	1.904	2.556	2.253	1.456
5	1.729	3.018	2.187	1.414
6	1.759	2.503	2.123	1.372
7	1.872	2.501	2.183	1.411
8	1.713	2.398	2.041	1.319
9	1.565	2.284	1.924	1.243
10	1.560	2.333	1.890	1.221
11	1.377	2.099	1.800	1.163
12	1.358	2.042	1.747	1.129
13	1.324	1.882	1.660	1.073
14	1.253	1.922	1.579	1.021
15	1.219	1.975	1.572	1.016
16	1.175	1.958	1.524	0.985
17	1.138	1.865	1.471	0.951
18	1.087	1.850	1.519	0.981
19	1.198	1.850	1.536	0.993
20	1.112	2.135	1.539	0.995
21	1.313	1.984	1.699	1.098
22	1.366	2.404	1.900	1.228
23	1.824	2.604	2.190	1.415
24	1.819	2.491	2.159	1.396
25	1.743	2.408	2.103	1.359
26	1.625	2.233	1.966	1.270
27	1.488	2.180	1.868	1.207
28	1.433	2.331	1.946	1.258
Mean	**1.579**	**2.325**	**1.949**	**1.259**
Maximum	**2.773**	**3.518**	**3.098**	**2.002**
Minimum	**1.087**	**1.850**	**1.471**	**0.951**
Total Flow (mg)		**35.263**		

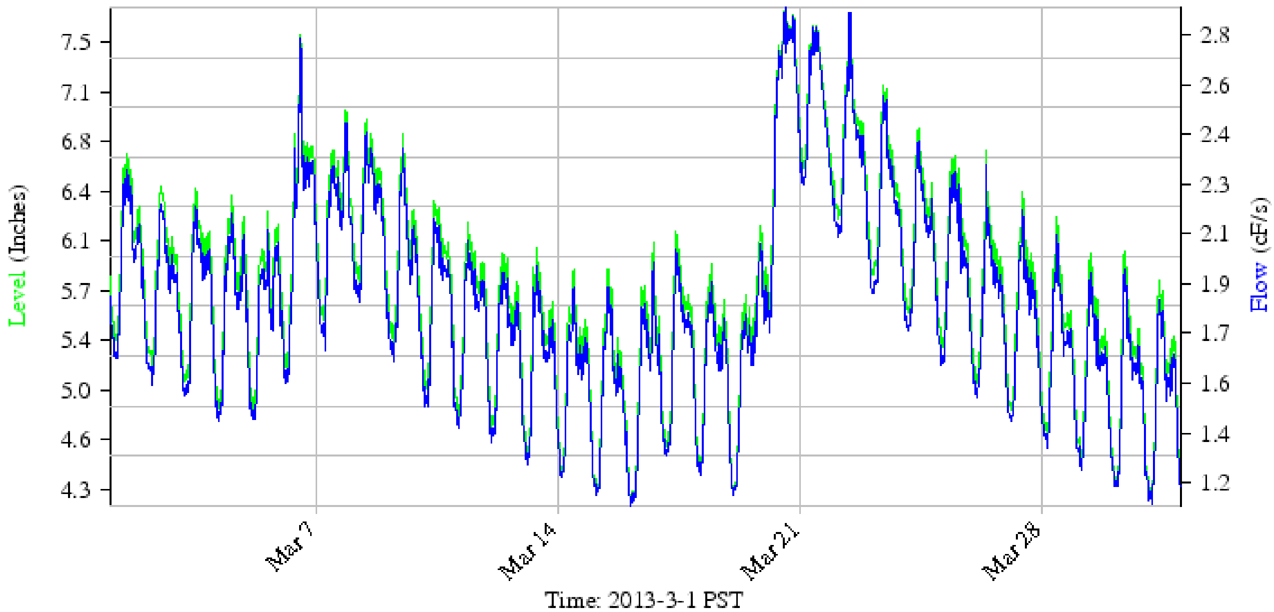


U12-118-3
Levels with Flow
Level (Inches)
Flow (cF/s)
8.3
7.9
7.4
6.9
6.5
6.0
5.6
5.1
4.6
4.2
3.3
3.0
2.8
2.6
2.3
2.1
1.9
1.6
1.4
1.2
Feb 7
Feb 14
Feb 21
Feb 28
Time: 2013-2-1 PST

Summary Report - March, 2013

U12-118-3	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Mar	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	1.640	2.342	1.979	1.279
2	1.543	2.200	1.872	1.210
3	1.508	2.182	1.849	1.195
4	1.417	2.165	1.809	1.169
5	1.420	2.097	1.787	1.155
6	1.542	2.781	2.108	1.363
7	1.663	2.496	2.101	1.358
8	1.833	2.458	2.132	1.378
9	1.621	2.395	1.947	1.259
10	1.455	2.144	1.816	1.174
11	1.397	2.061	1.762	1.139
12	1.343	1.949	1.685	1.089
13	1.273	1.995	1.629	1.053
14	1.227	1.896	1.562	1.009
15	1.166	1.912	1.510	0.976
16	1.129	2.020	1.543	0.997
17	1.240	2.032	1.645	1.063
18	1.229	1.925	1.586	1.025
19	1.159	2.051	1.644	1.062
20	1.720	2.903	2.482	1.604
21	2.133	2.820	2.495	1.613
22	1.986	2.929	2.348	1.518
23	1.809	2.567	2.161	1.397
24	1.709	2.419	2.033	1.314
25	1.592	2.311	1.946	1.258
26	1.489	2.332	1.829	1.182
27	1.419	2.218	1.756	1.135
28	1.322	2.135	1.658	1.071
29	1.241	1.960	1.582	1.022
30	1.187	1.958	1.534	0.991
31	1.137	1.866	1.479	0.956
Mean	**1.469**	**2.243**	**1.847**	**1.194**
Maximum	**2.133**	**2.929**	**2.495**	**1.613**
Minimum	**1.129**	**1.866**	**1.479**	**0.956**
Total Flow (mg)		**37.014**		

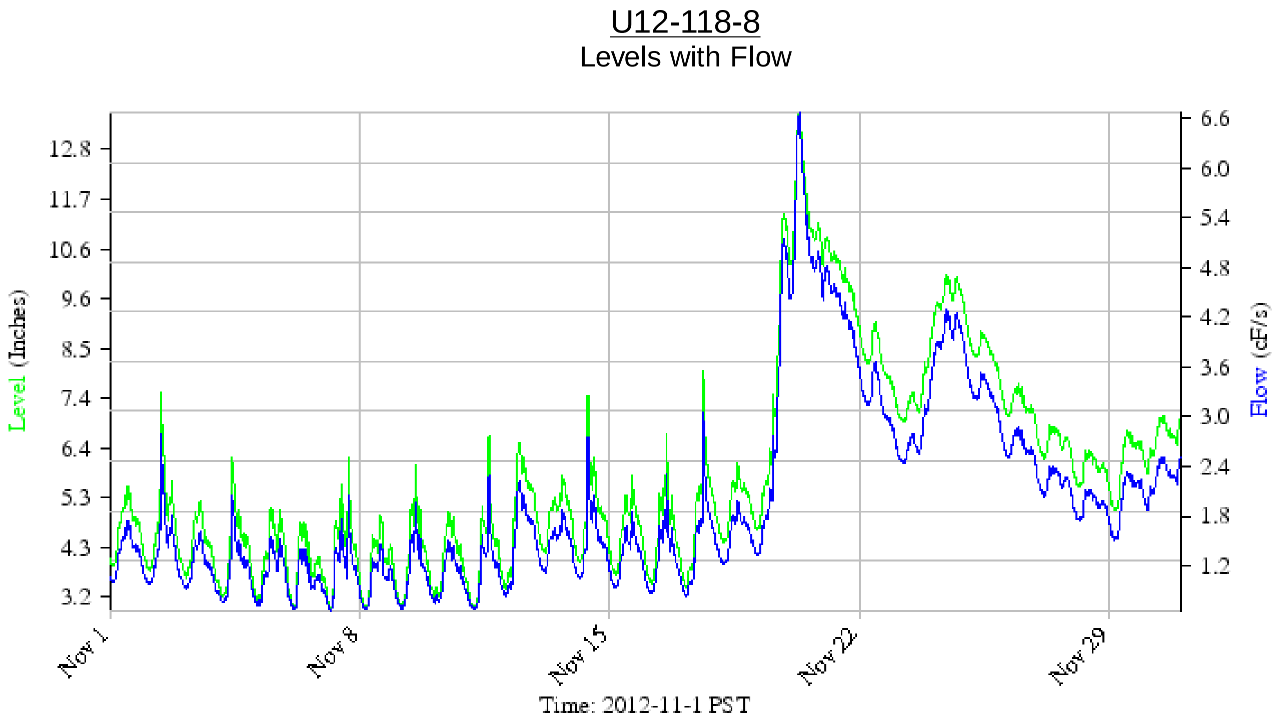
U12-118-3
Levels with Flow



Site U12-118-8

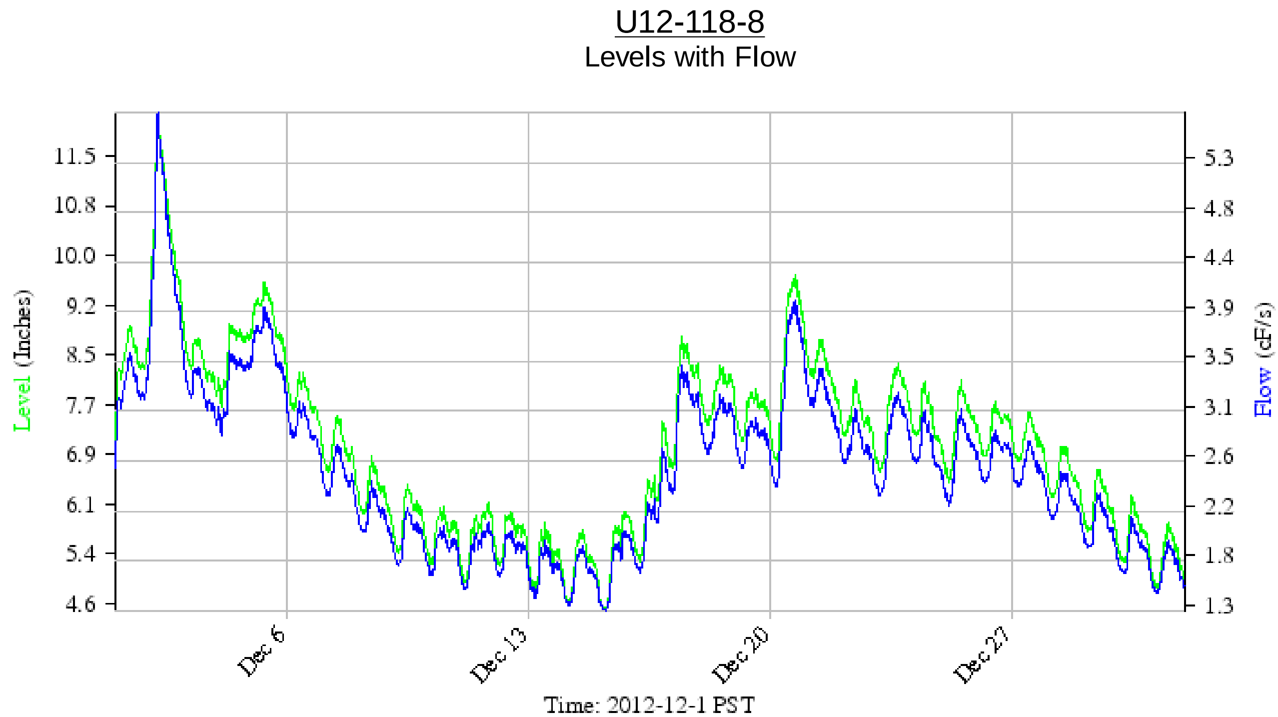
Summary Report - November, 2012

U12-118-8	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Nov	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.990	1.770	1.340	0.866
2	0.962	2.887	1.403	0.907
3	0.868	1.616	1.182	0.764
4	0.758	2.153	1.192	0.770
5	0.699	1.565	1.112	0.719
6	0.671	1.538	1.006	0.650
7	0.658	2.071	1.145	0.740
8	0.665	1.555	1.029	0.665
9	0.670	2.150	1.096	0.708
10	0.741	1.509	1.071	0.692
11	0.673	2.359	1.137	0.735
12	0.814	2.222	1.516	0.980
13	1.101	1.870	1.462	0.945
14	1.030	2.815	1.468	0.949
15	0.933	1.861	1.304	0.843
16	0.862	2.397	1.305	0.843
17	0.817	3.039	1.495	0.966
18	1.200	1.995	1.559	1.008
19	1.323	5.130	2.598	1.679
20	4.341	6.676	5.182	3.349
21	3.574	4.828	4.275	2.763
22	2.730	3.650	3.185	2.058
23	2.408	3.406	2.704	1.748
24	3.386	4.282	3.943	2.549
25	2.698	3.638	3.264	2.110
26	2.223	2.890	2.615	1.690
27	1.913	2.418	2.201	1.422
28	1.709	2.140	1.920	1.241
29	1.498	2.361	1.980	1.279
30	1.859	2.564	2.251	1.455
Mean	**1.492**	**2.712**	**1.965**	**1.270**
Maximum	**4.341**	**6.676**	**5.182**	**3.349**
Minimum	**0.658**	**1.509**	**1.006**	**0.650**
Total Flow (mg)	**38.095**			


U12-118-8
Levels with Flow
Level (Inches)
Flow (cF/s)
12.8
11.7
10.6
9.6
8.5
7.4
6.4
5.3
4.3
3.2
6.6
6.0
5.4
4.8
4.2
3.6
3.0
2.4
1.8
1.2
Nov 1
Nov 8
Nov 15
Nov 22
Nov 29
Time: 2012-11-1 PST

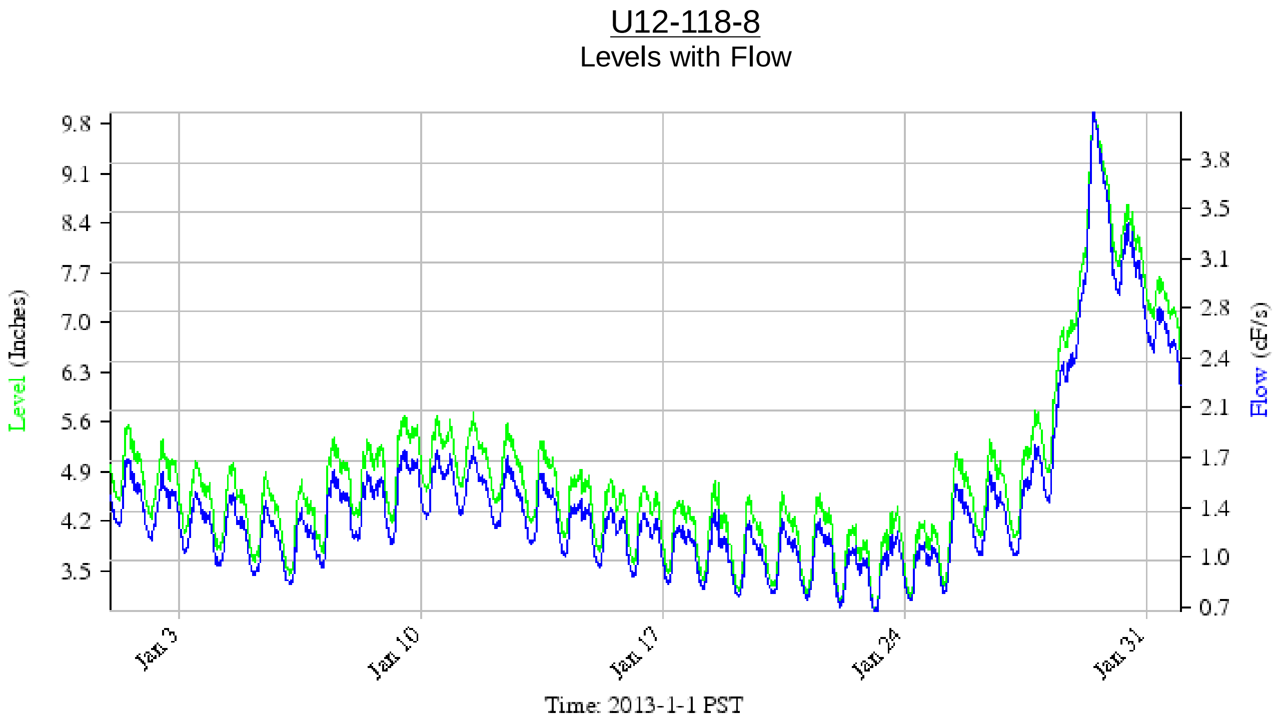
Summary Report - December, 2012

U12-118-8	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Dec	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	2.515	3.554	3.227	2.085
2	3.436	5.672	4.504	2.911
3	2.898	3.492	3.203	2.070
4	2.770	3.595	3.317	2.144
5	3.072	3.967	3.623	2.341
6	2.500	3.153	2.877	1.860
7	2.186	2.803	2.459	1.589
8	1.931	2.453	2.121	1.371
9	1.661	2.195	1.923	1.243
10	1.568	2.029	1.805	1.167
11	1.454	2.062	1.784	1.153
12	1.564	2.009	1.787	1.155
13	1.371	1.902	1.628	1.052
14	1.299	1.845	1.579	1.021
15	1.263	1.977	1.652	1.068
16	1.598	2.723	2.054	1.327
17	2.285	3.450	2.954	1.909
18	2.641	3.208	2.924	1.890
19	2.516	2.993	2.772	1.792
20	2.350	4.024	3.292	2.128
21	2.784	3.467	3.158	2.041
22	2.449	3.058	2.730	1.765
23	2.267	3.217	2.773	1.793
24	2.401	3.061	2.703	1.747
25	2.194	3.081	2.638	1.705
26	2.388	2.878	2.634	1.702
27	2.278	2.787	2.522	1.630
28	2.022	2.528	2.255	1.458
29	1.805	2.335	2.021	1.306
30	1.589	2.116	1.805	1.166
31	1.416	1.902	1.626	1.051
Mean	**2.144**	**2.888**	**2.527**	**1.634**
Maximum	**3.436**	**5.672**	**4.504**	**2.911**
Minimum	**1.263**	**1.845**	**1.579**	**1.021**
Total Flow (mg)		**50.639**		


U12-118-8
Levels with Flow
Level (Inches)
11.5
10.8
10.0
9.2
8.5
7.7
6.9
6.1
5.4
4.6
Flow (cF/s)
5.3
4.8
4.4
3.9
3.5
3.1
2.6
2.2
1.8
1.3
Dec 6
Dec 13
Dec 20
Dec 27
Time: 2012-12-1 PST

Summary Report - January, 2013

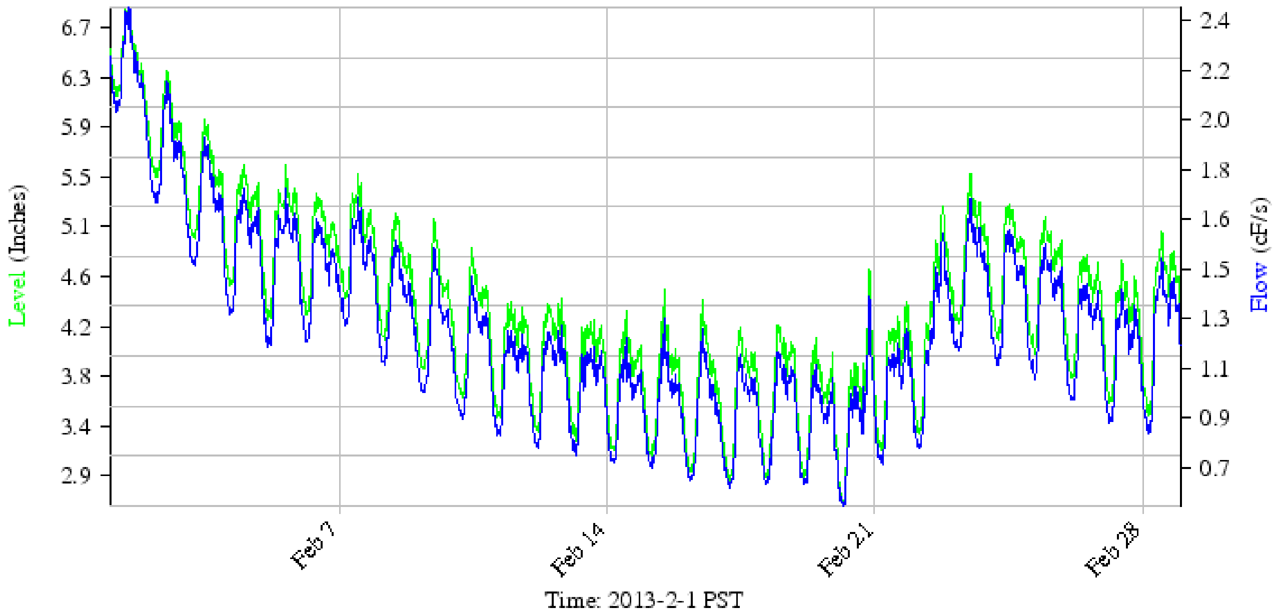
U12-118-8	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Jan	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	1.258	1.745	1.481	0.957
2	1.158	1.704	1.394	0.901
3	1.059	1.542	1.290	0.834
4	0.974	1.512	1.228	0.794
5	0.910	1.472	1.156	0.747
6	0.848	1.390	1.114	0.720
7	0.955	1.684	1.359	0.878
8	1.157	1.653	1.428	0.923
9	1.123	1.812	1.535	0.992
10	1.312	1.803	1.577	1.019
11	1.331	1.845	1.554	1.005
12	1.239	1.764	1.465	0.947
13	1.131	1.654	1.380	0.892
14	1.043	1.469	1.278	0.826
15	0.970	1.404	1.204	0.778
16	0.899	1.362	1.162	0.751
17	0.850	1.290	1.100	0.711
18	0.810	1.418	1.078	0.697
19	0.765	1.305	1.024	0.662
20	0.785	1.327	1.033	0.668
21	0.742	1.315	1.034	0.668
22	0.693	1.133	0.936	0.605
23	0.662	1.232	0.964	0.623
24	0.732	1.162	0.983	0.635
25	0.791	1.574	1.222	0.790
26	1.046	1.661	1.334	0.862
27	1.049	1.850	1.439	0.930
28	1.421	2.539	2.052	1.326
29	2.482	4.208	3.479	2.248
30	2.681	3.409	3.075	1.988
31	2.244	2.816	2.584	1.670
Mean	**1.133**	**1.744**	**1.450**	**0.937**
Maximum	**2.681**	**4.208**	**3.479**	**2.248**
Minimum	**0.662**	**1.133**	**0.936**	**0.605**
Total Flow (mg)		**29.047**		


U12-118-8
Levels with Flow
Level (Inches)
Flow (cF/s)
9.8
9.1
8.4
7.7
7.0
6.3
5.6
4.9
4.2
3.5
3.8
3.5
3.1
2.8
2.4
2.1
1.7
1.4
1.0
0.7
Jan 3
Jan 10
Jan 17
Jan 24
Jan 31
Time: 2013-1-1 PST

Summary Report - February, 2013

U12-118-8	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Feb	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	1.911	2.452	2.182	1.411
2	1.692	2.160	1.881	1.216
3	1.460	1.949	1.685	1.089
4	1.291	1.777	1.545	0.999
5	1.166	1.769	1.486	0.960
6	1.182	1.661	1.440	0.930
7	1.246	1.733	1.479	0.956
8	1.103	1.579	1.331	0.860
9	0.998	1.558	1.232	0.796
10	0.904	1.449	1.162	0.751
11	0.846	1.260	1.074	0.694
12	0.798	1.254	1.063	0.687
13	0.775	1.198	1.020	0.659
14	0.744	1.226	0.972	0.628
15	0.725	1.380	0.965	0.624
16	0.684	1.252	0.929	0.601
17	0.655	1.154	0.916	0.592
18	0.666	1.183	0.940	0.607
19	0.669	1.124	0.894	0.578
20	0.591	1.377	0.906	0.585
21	0.740	1.243	1.020	0.659
22	0.795	1.597	1.188	0.768
23	1.151	1.724	1.430	0.924
24	1.093	1.634	1.370	0.886
25	1.050	1.564	1.325	0.856
26	0.971	1.421	1.225	0.792
27	0.881	1.391	1.157	0.747
28	0.846	1.522	1.223	0.791
Mean	**0.987**	**1.521**	**1.251**	**0.809**
Maximum	**1.911**	**2.452**	**2.182**	**1.411**
Minimum	**0.591**	**1.124**	**0.894**	**0.578**
Total Flow (mg)	**22.646**			

U12-118-8
Levels with Flow

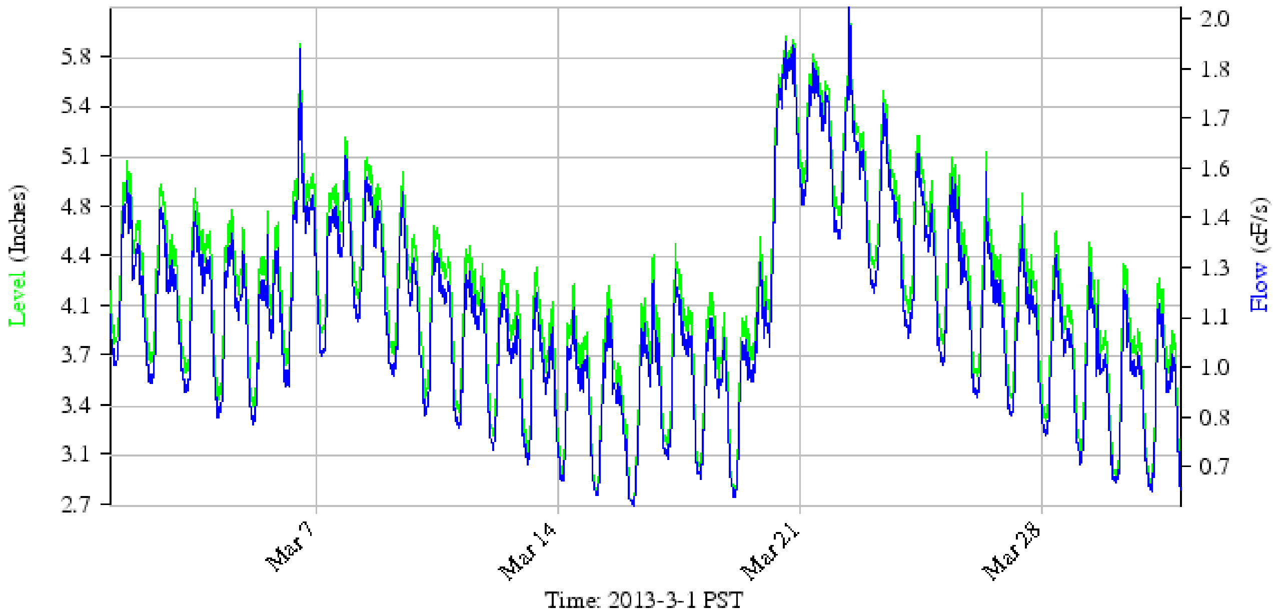


Summary Report - March, 2013

U12-118-8	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Mar	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.985	1.564	1.246	0.805
2	0.935	1.450	1.181	0.763
3	0.911	1.443	1.171	0.757
4	0.841	1.416	1.130	0.731
5	0.823	1.362	1.104	0.714
6	0.928	1.942	1.329	0.859
7	1.012	1.616	1.304	0.843
8	1.097	1.554	1.329	0.859
9	0.959	1.508	1.181	0.763
10	0.845	1.343	1.103	0.713
11	0.808	1.274	1.062	0.687
12	0.748	1.216	1.001	0.647
13	0.712	1.207	0.956	0.618
14	0.660	1.169	0.908	0.587
15	0.614	1.172	0.869	0.562
16	0.591	1.252	0.902	0.583
17	0.702	1.284	0.975	0.630
18	0.657	1.145	0.914	0.591
19	0.615	1.295	0.970	0.627
20	1.026	1.950	1.618	1.046
21	1.417	1.871	1.660	1.073
22	1.276	2.045	1.558	1.007
23	1.139	1.749	1.408	0.910
24	1.044	1.598	1.313	0.848
25	0.956	1.536	1.236	0.799
26	0.893	1.613	1.146	0.741
27	0.841	1.465	1.091	0.705
28	0.781	1.330	1.024	0.662
29	0.711	1.292	0.961	0.621
30	0.660	1.228	0.919	0.594
31	0.631	1.191	0.892	0.577
Mean	**0.865**	**1.454**	**1.144**	**0.739**
Maximum	**1.417**	**2.045**	**1.660**	**1.073**
Minimum	**0.591**	**1.145**	**0.869**	**0.562**
Total Flow (mg)		**22.919**		

U12-118-8

Levels with Flow

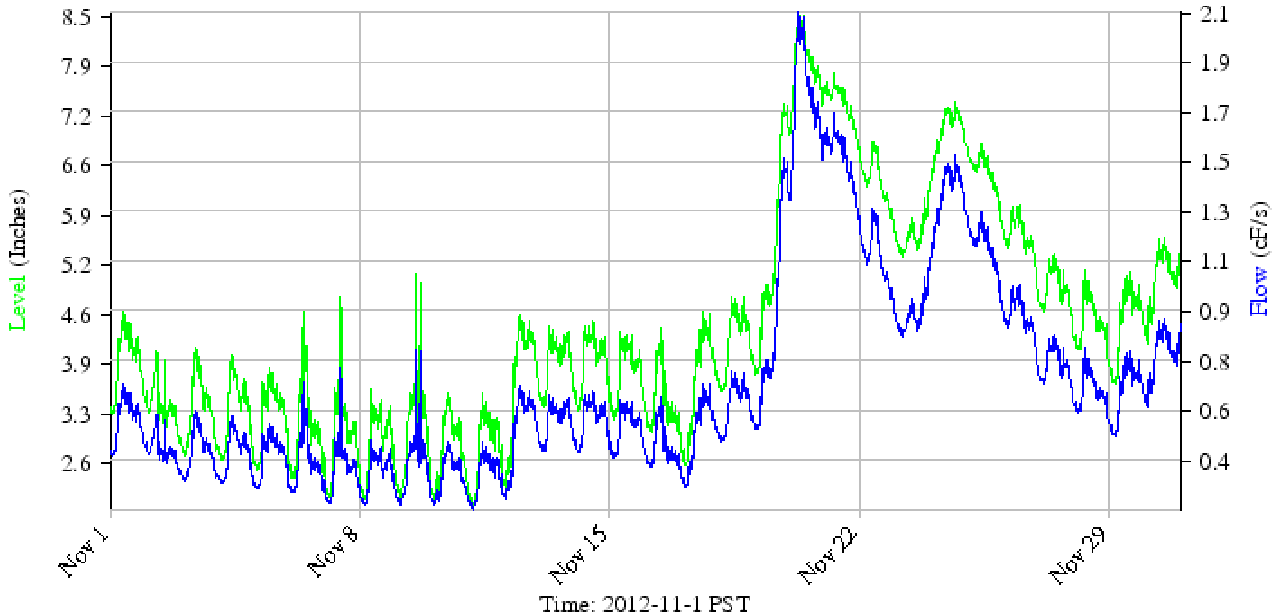


Site U12-118-8A

Summary Report - November, 2012

U12-118-8A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Nov	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.391	0.690	0.529	0.342
2	0.320	0.608	0.416	0.269
3	0.295	0.589	0.412	0.266
4	0.288	0.584	0.410	0.265
5	0.264	0.555	0.391	0.253
6	0.250	0.814	0.369	0.239
7	0.210	0.917	0.347	0.224
8	0.208	0.474	0.333	0.216
9	0.209	1.032	0.356	0.230
10	0.207	0.460	0.322	0.208
11	0.188	0.466	0.317	0.205
12	0.228	0.675	0.485	0.313
13	0.405	0.637	0.525	0.339
14	0.401	0.663	0.530	0.342
15	0.375	0.628	0.498	0.322
16	0.320	0.664	0.423	0.274
17	0.277	0.694	0.478	0.309
18	0.446	0.726	0.584	0.377
19	0.489	1.530	0.866	0.560
20	1.346	2.133	1.738	1.123
21	1.278	1.714	1.522	0.984
22	0.924	1.352	1.143	0.739
23	0.838	1.144	0.931	0.602
24	1.093	1.545	1.378	0.891
25	0.953	1.352	1.187	0.767
26	0.732	1.078	0.921	0.595
27	0.600	0.874	0.737	0.477
28	0.511	0.810	0.635	0.411
29	0.464	0.817	0.638	0.412
30	0.572	0.947	0.770	0.498
Mean	**0.503**	**0.906**	**0.673**	**0.435**
Maximum	**1.346**	**2.133**	**1.738**	**1.123**
Minimum	**0.188**	**0.460**	**0.317**	**0.205**
Total Flow (mg)		**13.051**		

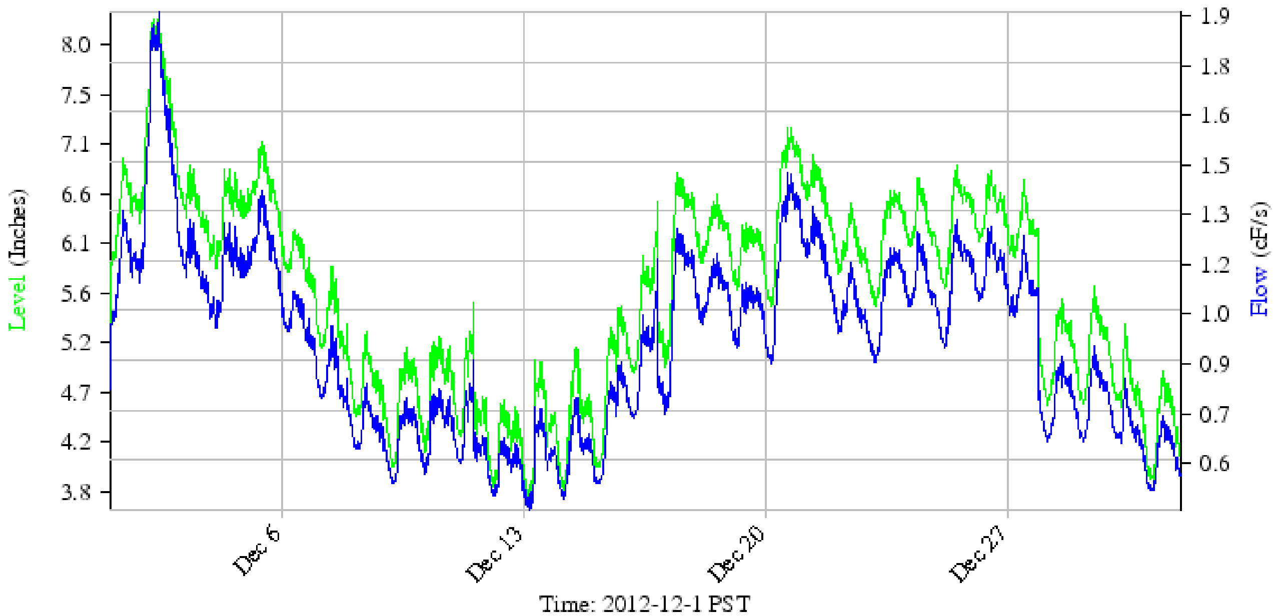
U12-118-8A
Levels with Flow



Summary Report - December, 2012

U12-118-8A	Flow (cF/s)			
	Min	Max	Avg	Total
Dec	cF/s	cF/s	cF/s	mg/d
1	0.791	1.358	1.167	0.754
2	1.235	2.005	1.629	1.053
3	1.008	1.345	1.171	0.757
4	0.974	1.327	1.167	0.754
5	1.022	1.447	1.243	0.803
6	0.845	1.130	1.009	0.652
7	0.700	1.093	0.833	0.539
8	0.607	0.843	0.698	0.451
9	0.523	0.800	0.663	0.428
10	0.557	0.837	0.703	0.454
11	0.535	0.914	0.667	0.431
12	0.491	0.687	0.581	0.376
13	0.453	0.776	0.600	0.388
14	0.485	0.804	0.631	0.408
15	0.527	0.922	0.719	0.465
16	0.727	1.245	0.886	0.573
17	0.736	1.335	1.081	0.699
18	0.998	1.246	1.104	0.714
19	0.922	1.177	1.043	0.674
20	0.877	1.491	1.216	0.786
21	1.034	1.375	1.224	0.791
22	0.939	1.211	1.051	0.679
23	0.886	1.260	1.088	0.703
24	1.020	1.311	1.129	0.730
25	0.933	1.328	1.141	0.737
26	1.070	1.309	1.168	0.755
27	0.729	1.274	1.073	0.694
28	0.652	0.929	0.778	0.503
29	0.656	0.947	0.787	0.509
30	0.568	0.859	0.693	0.448
31	0.506	0.750	0.612	0.395
Mean	**0.774**	**1.140**	**0.953**	**0.616**
Maximum	**1.235**	**2.005**	**1.629**	**1.053**
Minimum	**0.453**	**0.687**	**0.581**	**0.376**
Total Flow (mg)		**19.103**		

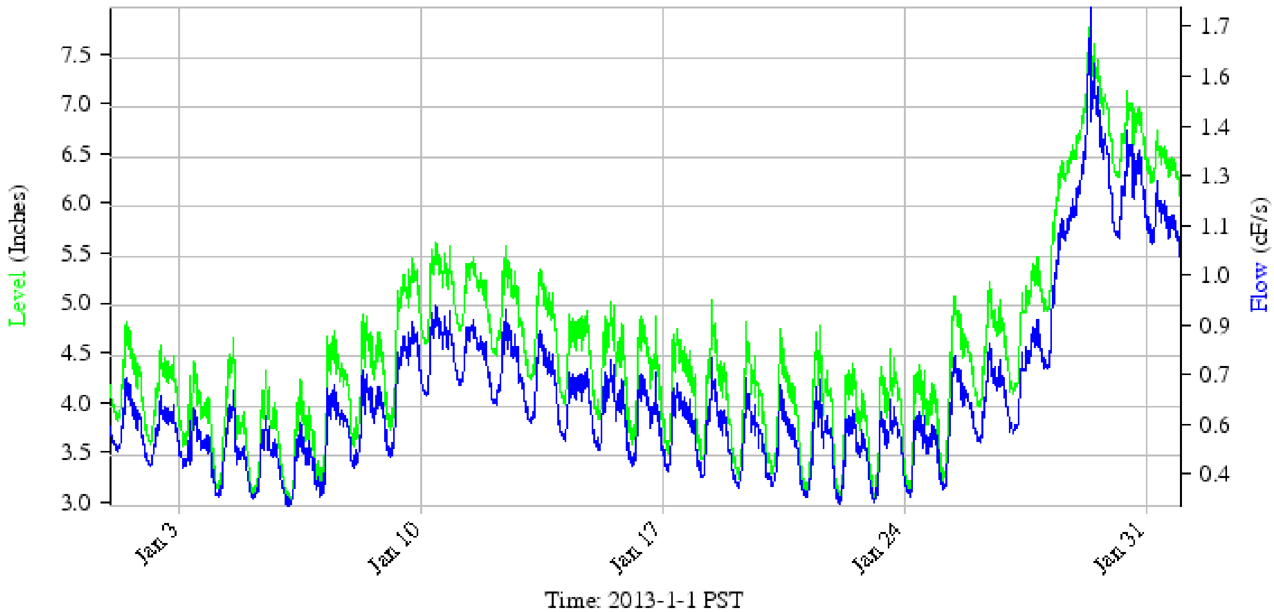
U12-118-8A
Levels with Flow



Summary Report - January, 2013

U12-118-8A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Jan	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.488	0.714	0.579	0.374
2	0.446	0.666	0.550	0.355
3	0.410	0.649	0.510	0.330
4	0.359	0.672	0.485	0.313
5	0.355	0.605	0.457	0.295
6	0.336	0.596	0.441	0.285
7	0.360	0.742	0.545	0.352
8	0.443	0.762	0.591	0.382
9	0.473	0.869	0.706	0.456
10	0.650	0.931	0.793	0.513
11	0.681	0.894	0.778	0.503
12	0.626	0.924	0.739	0.478
13	0.566	0.854	0.695	0.449
14	0.520	0.752	0.639	0.413
15	0.494	0.777	0.615	0.398
16	0.447	0.750	0.584	0.377
17	0.433	0.716	0.569	0.367
18	0.418	0.883	0.552	0.357
19	0.385	0.726	0.521	0.337
20	0.391	0.717	0.512	0.331
21	0.357	0.739	0.506	0.327
22	0.341	0.631	0.487	0.314
23	0.341	0.661	0.500	0.323
24	0.361	0.624	0.501	0.324
25	0.379	0.797	0.596	0.385
26	0.516	0.825	0.651	0.421
27	0.544	0.897	0.714	0.461
28	0.724	1.279	1.012	0.654
29	1.198	1.782	1.406	0.909
30	1.093	1.484	1.247	0.806
31	1.039	1.278	1.151	0.744
Mean	**0.522**	**0.845**	**0.666**	**0.430**
Maximum	**1.198**	**1.782**	**1.406**	**0.909**
Minimum	**0.336**	**0.596**	**0.441**	**0.285**
Total Flow (mg)		**13.336**		

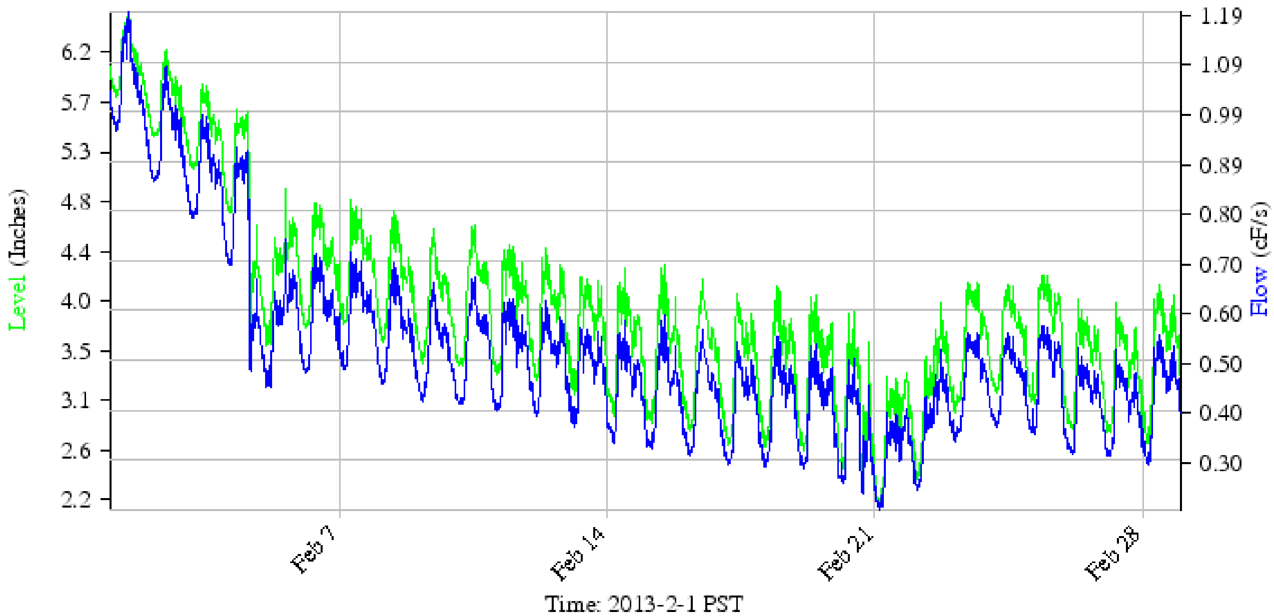
U12-118-8A
Levels with Flow



Summary Report - February, 2013

U12-118-8A	Flow (cF/s)			
	Min	Max	Avg	Total
Feb	cF/s	cF/s	cF/s	mg/d
1	0.920	1.250	1.049	0.678
2	0.851	1.102	0.951	0.615
3	0.760	1.011	0.871	0.563
4	0.475	0.952	0.727	0.470
5	0.443	0.758	0.579	0.374
6	0.475	0.746	0.593	0.383
7	0.480	0.733	0.599	0.387
8	0.455	0.754	0.564	0.365
9	0.418	0.692	0.522	0.338
10	0.413	0.679	0.533	0.344
11	0.396	0.674	0.521	0.336
12	0.389	0.643	0.505	0.326
13	0.367	0.594	0.483	0.312
14	0.335	0.608	0.454	0.294
15	0.324	0.632	0.444	0.287
16	0.310	0.583	0.428	0.277
17	0.294	0.553	0.412	0.266
18	0.287	0.575	0.418	0.270
19	0.282	0.556	0.411	0.266
20	0.228	0.520	0.349	0.226
21	0.203	0.423	0.320	0.207
22	0.242	0.539	0.380	0.246
23	0.340	0.576	0.461	0.298
24	0.369	0.569	0.463	0.299
25	0.349	0.597	0.470	0.303
26	0.312	0.559	0.419	0.271
27	0.308	0.555	0.412	0.266
28	0.294	0.564	0.437	0.282
Mean	**0.404**	**0.678**	**0.528**	**0.341**
Maximum	**0.920**	**1.250**	**1.049**	**0.678**
Minimum	**0.203**	**0.423**	**0.320**	**0.207**
Total Flow (mg)		**9.548**		

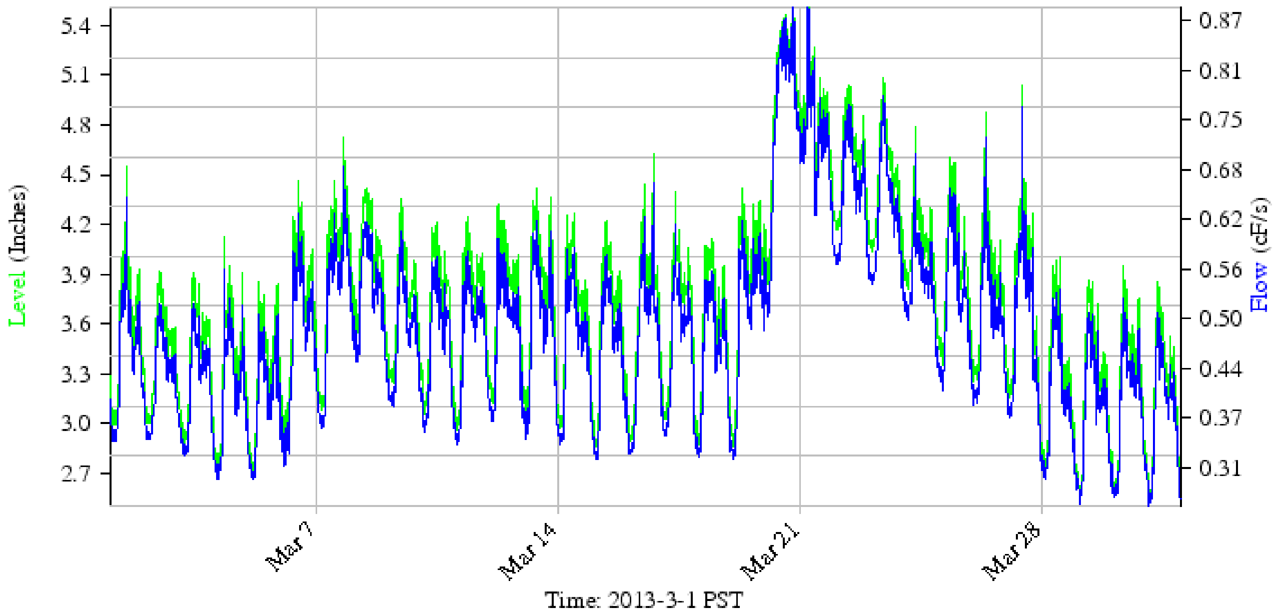
U12-118-8A
Levels with Flow



Summary Report - March, 2013

U12-118-8A	Flow (cF/s)			
	Min	**Max**	**Avg**	**Total**
Mar	**cF/s**	**cF/s**	**cF/s**	**mg/d**
1	0.343	0.730	0.450	0.291
2	0.344	0.540	0.423	0.273
3	0.322	0.523	0.415	0.268
4	0.294	0.570	0.406	0.263
5	0.294	0.509	0.396	0.256
6	0.310	0.649	0.474	0.306
7	0.358	0.690	0.518	0.335
8	0.434	0.661	0.532	0.344
9	0.386	0.625	0.475	0.307
10	0.353	0.591	0.460	0.297
11	0.337	0.602	0.473	0.306
12	0.359	0.628	0.481	0.311
13	0.344	0.630	0.487	0.315
14	0.338	0.617	0.463	0.299
15	0.316	0.602	0.455	0.294
16	0.325	0.703	0.467	0.302
17	0.332	0.635	0.459	0.297
18	0.321	0.585	0.455	0.294
19	0.320	0.641	0.509	0.329
20	0.485	0.902	0.750	0.485
21	0.571	0.923	0.720	0.465
22	0.545	0.786	0.655	0.423
23	0.516	0.796	0.630	0.407
24	0.421	0.718	0.544	0.352
25	0.394	0.753	0.521	0.336
26	0.385	0.738	0.498	0.322
27	0.316	0.789	0.479	0.310
28	0.283	0.556	0.396	0.256
29	0.264	0.628	0.385	0.249
30	0.273	0.537	0.393	0.254
31	0.263	0.514	0.377	0.244
Mean	**0.360**	**0.657**	**0.489**	**0.316**
Maximum	**0.571**	**0.923**	**0.750**	**0.485**
Minimum	**0.263**	**0.509**	**0.377**	**0.244**
Total Flow (mg)		**9.789**		

U12-118-8A
Levels with Flow



Appendix D: DEQ Letter - February 18, 2016

Oregon
Kate Brown, Governor

Department of Environmental Quality
Western Region Salem Office
4026 Fairview Industrial Dr SE
Salem, OR 97302
(503) 378-8240
FAX (503) 373-7944
TTY 711

February 18, 2016

RECEIVED
FEB 2 2 2016
BY:

Mr. Michael Adams
Public Works Director
City of Sweet Home
1140 12th Avenue
Sweet Home, OR 97386

RE: Facility Plan Meeting Summation Questions
WQ - Linn County / File No. 86840

Dear Mr. Adams:

As a result of the facility planning meeting on January 21, 2016, for the City of Sweet Home's wastewater treatment system, DEQ was asked to provide clarification and answers to a number of questions raised during that meeting. Per your email, dated January 26, 2016, you summarized those questions as follows:

1. Review and clarify process to incorporate prior MAO language on daily mass load limits, i.e., daily mass limits would not apply on days when the plant flow exceeds twice the average dry weather design flow, into the NPDES permit. As discussed, without this provision, the City is effectively required to meet effluent limits of 8 mg/L $CBOD_5$ and 8 mg/L TSS currently, dropping to 7/7 in future.
2. Review and clarify process to extend the wet season effluent limits in the NPDES permit through May (rather than April), as per the prior MAO. As discussed, if May is not included, the maximum month dry weather flow and the dry season daily mass limit combine to effectively require $CBOD_5$ and TSS effluent concentrations of 5 mg/L.
3. Review and clarify process to remove the ammonia limit from the NPDES permit upon approval of the new ammonia standard by EPA, as described in the current permit.
4. Given the December 17, 2015, SSO event at less than a 5-year recurrence flow, review DEQ willingness to negotiate a new MAO to cover time required for the City to complete Phases I and 2 of the proposed WWTP improvements.

DEQ hereby submits the following responses to the questions listed above:

1. Per OAR 340-041-0061(9)(a)(I), Paragraphs (A) through (G) of this subsection do not apply to the Cities of Athena, Elgin, Adair Village, Halsey, Harrisburg, Independence, Carlton, and Sweet Home. The City of Sweet Home and these other cities are considered "new facilities" that have individually assigned mass loads per OAR 340-041-0061(9)(b). Therefore, in accordance with the OARs, the City of Sweet Home is not eligible for the exemption in OAR 340-041-0061(9)(a)(C) in their NPDES permit. Per OAR 340-041-0061(9)(b), for new sewage treatment facilities or facilities expanding the average dry weather treatment capacity and receiving engineering plans and specifications approval from DEQ after June 30, 1992, the mass load limits must be calculated by DEQ based on the proposed treatment facility capabilities and the highest and best practicable treatment to minimize the discharge of pollutants.

2. In order for DEQ to approve and permit the current wet weather mass loads in the month of May, the City of Sweet Home would have to provide the data and information to justify a mass load increase under the Anti-degradation rule, OAR 340-041-0004(9)(a)-(c).
3. In accordance with Schedule A, Note 5 of the NPDES permit, upon approval of the new ammonia standard by the EPA, the following limits will automatically be applied to the discharge without a permit modification: Ammonia-N, No limit. The EPA approved, DEQs ammonia standard on August 4, 2015. Therefore, the City does not have to meet an ammonia limit in the current permit. However, when DEQ takes action on the City of Sweet Home NPDES permit renewal application, a Reasonable Potential Analysis (RPA) for ammonia toxicity in the effluent will be performed using the new ammonia standard criteria. If the RPA indicates toxicity above the standard, ammonia limits may be required in the renewal permit.
4. While the December 2015, SSO occurred due to flows in the collection system as a result of a storm event that was recorded at less than the 5-year, 24-hour event, DEQ considers many factors in determining the permittees responsibility for compliance and whether the SSO was beyond reasonable control of the permittee. E.g., was the SSO caused by conditions equivalent to the 5-year, 24-hour storm event; was the SSO caused by a storm event larger than what the system was designed to handle; or, did the SSO occur despite the fact that the permittee is implementing a good Capacity Management, Operation and Maintenance (CMOM) program. DEQ has determined that the latest event was beyond the reasonable control of the permittee and therefore is not taking enforcement action. DEQ understands that the City is still working to increase the capacity of the treatment facility to assure that sewage overflows do not occur in the future. However, DEQ does not negotiate or enter into MAOs for possible future violations. Should the City experience a raw sewage overflow in the future, DEQ will examine the evidence for the cause of the overflow and may use enforcement discretion if the overflow was beyond the City's reasonable control. Additional known permit violations may be a basis to negotiate a new MAO.

Thank you for your continued efforts in improving our environment and we look forward to working with the City of Sweet Home during Facility Planning and implementing the proposed upgrades to the wastewater treatment facility. Should you have any questions, please contact me at 503-378-5039.

Sincerely,

Robert Dicksa

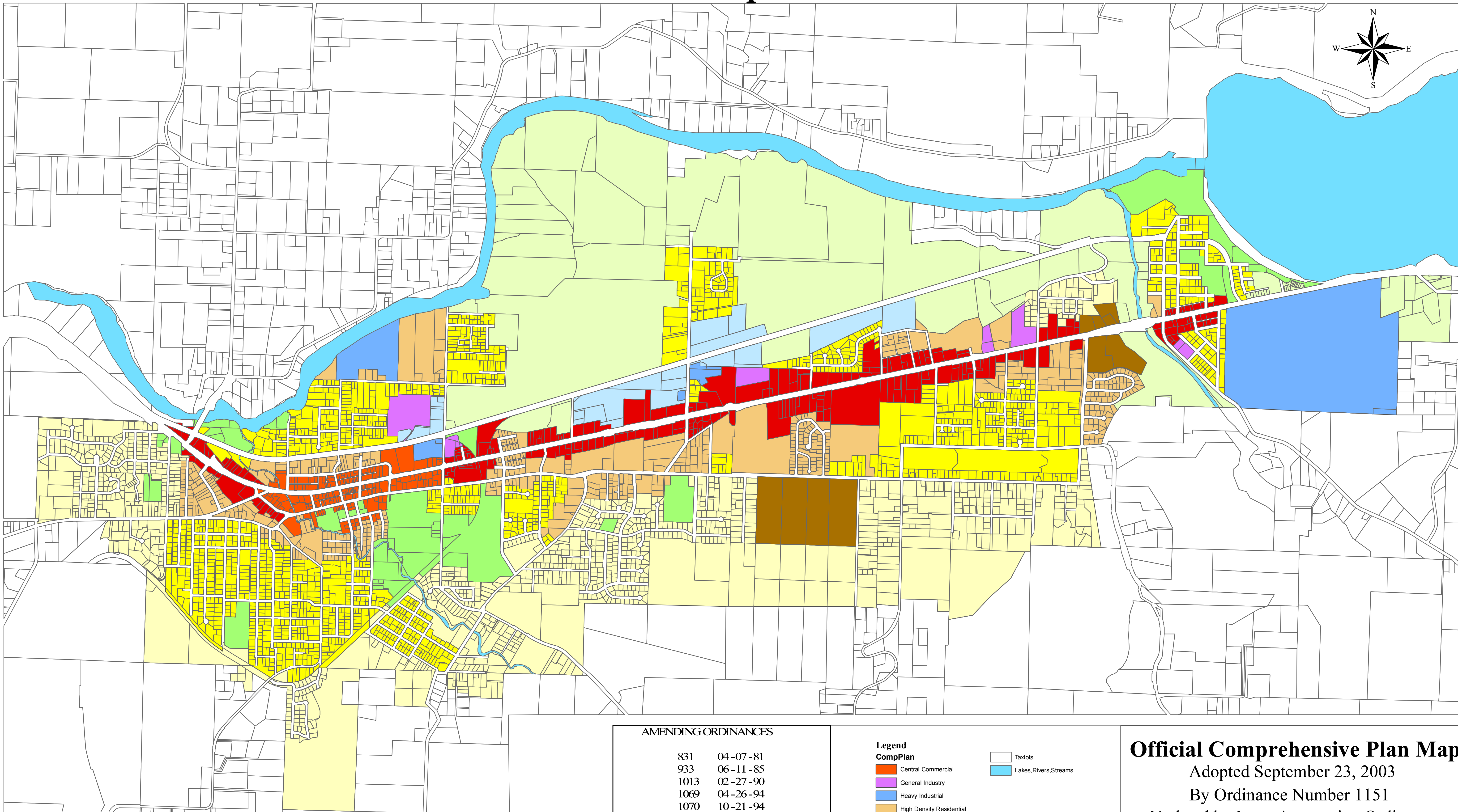
Robert Dicksa
Senior Water Quality Specialist
DEQ WR-Salem Office

Cc: Water Quality File-Salem

Jon Holland, P.E.
Brown and Caldwell
6500 SW macadam Avenue, Suite 200
Portland, OR 97239

Appendix E: Comprehensive Plan Map

Sweet Home Comprehensive Plan



AMENDING ORDINANCES

831	04-07-81
933	06-11-85
1013	02-27-90
1069	04-26-94
1070	10-21-94
1081	12-13-94
1083	06-19-95
1102	04-22-97
1105	05-27-97
1114	12-09-97
1125	11-09-09
1151	09-23-03
1197	12-11-07

Legend

CompPlan

- Central Commercial
- General Industry
- Heavy Industrial
- High Density Residential
- Highway Commercial
- Light Industrial
- Low Density Residential
- Medium Density Residential
- Mixed Use Residential
- Planned Recreation Commercial
- Public
- Taxlots
- Lakes,Rivers,Streams

Official Comprehensive Plan Map

Adopted September 23, 2003

By Ordinance Number 1151

Updated by Later Annexation Ordinances

Miles

0 0.5 1

1 inch equals 0.32 miles

This map was created for display purposes only and is subject to errors and/or omissions. The City of Sweet Home and Linn County disclaim any liability as to the accuracy of the data

Created By:

City of Sweet Home
Community Development

541-367-8113

Revised by SV 1/09/09

Appendix F: Cost Estimate

Memorandum

Date: November 10, 2015

To: Harry Ritter, Portland

From: Catherine Dummer, Portland

Internal ESG Review By: Butch Matthews

Project No.: 144602.004.001

Subject: Sweet Home Facilities Plan
Planning Level Design Completion
Basis of Estimate of Probable Construction Cost

The Basis of Estimate Report and supporting estimate reports for the subject project are attached. Please call me if you have questions or need additional information.

CD:cd

Enclosures (3):

1. Basis of Estimate Report
2. Summary Estimate

Basis of Estimate Report

Sweet Home Facilities Plan

Introduction

Brown and Caldwell (BC) is pleased to present this opinion of probable construction cost (estimate) prepared for the Sweet Home Facilities Plan in Sweet Home, Oregon.

Summary

This Basis of Estimate contains the following information:

- Scope of work
- Background of this estimate
- Class of estimate
- Estimating methodology
- Direct cost development
- Indirect cost development
- Bidding assumptions
- Estimating assumptions
- Estimating exclusions
- Allowances for known but undefined work
- Contractor and other estimate markups

Scope of Work

The scope includes estimates for a variety of alternatives and costs will be used both to compare with other alternatives as well as for budgeting purposes. Alternatives detailed in the estimate will be packaged appropriately by the project team into complete alternatives for presentation to the client. See the estimate for a breakdown of the alternatives.

Background of this Estimate of Probable Construction Cost

The attached estimate of probable construction cost is based on documents dated June 9, 2014 through July 3, 2014, received by the ESG. These documents are described as planning level documents. Further information can be found in the detailed estimate reports. Additional information was also provided for updates in early December 2014, early February 2015, and March 2 2015. Additional modifications were made in October and November 2015, primarily comprising modification to the work breakdown structure.

AACEI Estimate Classification

In accordance with the Association for the Advancement of Cost Engineering International (AACE) criteria, this is a Class 5 estimate. A Class 5 estimate is defined as a Conceptual Level or Project Viability Estimate. Typically, engineering is from 0 to 2 percent complete. Class 5 estimates are used to prepare planning level cost scopes or evaluation of alternative schemes, long range capital outlay planning and can also form the base work for the Class 4 Planning Level or Design Technical Feasibility Estimate.

Expected accuracy for Class 5 estimates typically ranges from -50 to +100 percent, depending on the technological complexity of the project, appropriate reference information and the inclusion of an appropriate contingency determination. In unusual circumstances, ranges could exceed those shown.

Estimating Methodology

This estimate was prepared using quantity take-offs, vendor quotes and equipment pricing furnished either by the project team or by the estimator. The estimate includes direct labor costs and anticipated productivity adjustments to labor, and equipment. Where possible, estimates for work anticipated to be performed by specialty subcontractors have been identified.

Construction labor crew and equipment hours were calculated from production rates contained in documents and electronic databases published by R.S. Means, Mechanical Contractors Association (MCA), National Electrical Contractors Association (NECA), and Rental Rate Blue Book for Construction Equipment (Blue Book).

This estimate was prepared using BC's estimating system, which consists of a Windows-based commercial estimating software engine using BC's material and labor database, historical project data, the latest vendor and material cost information, and other costs specific to the project locale.

Direct Cost Development

Costs associated with the General Provisions and the Special Provisions of the construction documents, which are collectively referred to as Contractor General Conditions (CGC), were based on the estimator's interpretation of the contract documents. The estimates for CGCs are divided into two groups: a time-related group (e.g., field personnel), and non-time-related group (e.g., bonds and insurance). Labor burdens such as health and welfare, vacation, union benefits, payroll taxes, and workers compensation insurance are included in the labor rates. No trade discounts were considered.

Indirect Cost Development

A percentage allowance for contractor's home office expense has been included in the overall rate markups. The rate is standard for this type of heavy construction and is based on typical percentages outlined in Means Heavy Construction Cost Data.

The contractor's cost for builder's risk, general liability and vehicle insurance has been included in this estimate. Based on historical data, this is typically two to four percent of the overall construction contract amount. These indirect costs have been included in this estimate as a percentage of the gross cost, and are added after the net markups have been applied to the appropriate items.

Bidding Assumptions

The following bidding assumptions were considered in the development of this estimate.

1. Bidders must hold a valid, current Contractor's credentials, applicable to the type of project.
2. Bidders will develop estimates with a competitive approach to material pricing and labor productivity, and will not include allowances for changes, extra work, unforeseen conditions or any other unplanned costs.
3. Estimated costs are based on a minimum of four bidders. Actual bid prices may increase for fewer bidders or decrease for a greater number of bidders.

4. Bidders will account for General Provisions and Special Provisions of the contract documents and will perform all work except that which will be performed by traditional specialty subcontractors as identified here:

- Electrical and Instrumentation
- HVAC systems
- Painting and Coatings
- Pre-engineered metal building construction

Estimating Assumptions

As the design progresses through different completion stages, it is customary for the estimator to make assumptions to account for details that may not be evident from the documents. The following assumptions were used in the development of this estimate.

1. Contractor performs the work during normal daylight hours, nominally 7 a.m. to 5 p.m., Monday through Friday, in an 8-hour shift. No allowance has been made for additional shift work or weekend work.
2. Contractor has complete access for lay-down areas and mobile equipment.
3. Equipment rental rates are based on verifiable pricing from the local project area rental yards, Blue Book rates and/or rates contained in the estimating database.
4. Contractor markup is based on conventionally accepted values that have been adjusted for project-area economic factors.
5. Major equipment costs are based on both vendor supplied price quotes obtained by the project design team and/or estimators, and on historical pricing of like equipment.
6. Process equipment vendor training using vendors' standard Operations and Maintenance (O&M) material, is included in the purchase price of major equipment items where so stated in that quotation.
7. Bulk material quantities are based on manual quantity take-offs.
8. There is sufficient electrical power to feed the specified equipment. The local power company will supply power and transformers suitable for this facility.
9. Soils are of adequate nature to support the structures. No piles have been included in this estimate.

Estimating Exclusions

The following estimating exclusions were assumed in the development of this estimate.

1. Hazardous materials remediation and/or disposal.
2. O&M costs for the project with the exception of the vendor supplied O&M manuals.
3. Utility agency costs for incoming power modifications.
4. Permits beyond those normally needed for the type of project and project conditions.

Allowances for Known but Undefined Work

The following allowances were made in the development of this estimate.

1. Bypass pumping
2. Stairs and platforms
3. Headworks piping for screens, washer/compactor, and sluiceway
4. Headworks grit piping

5. FRP baffle walls
6. Existing aeration basin bar screen and channel demo, concrete repair
7. Replace existing aeration basin outlet structures with adjustable weirs
8. Aeration basin air piping
9. Coating
10. Existing aeration basin concrete joint repair
11. Chemical feed pump duplex system
12. Pipe supports
13. Existing filter pump station retrofit
14. Rapid mixer
15. Disinfection analyzers
16. Repair existing chlorine contact chamber
17. Modifications to existing chlorine contact for dry and wet weather UV
18. Improve inlet and outlet piping at existing chlorine contact
19. Outfall diffuser assembly
20. Existing dewatering building piping modifications
21. Biosolids storage or digester air supply piping
22. Aerobic digester feed piping modifications
23. Site piping
24. Civil site work including paving, grading, and landscaping
25. Small bore piping
26. Pipe support
27. Valves and fittings

Contractor and Other Estimate Markups

Contractor markup is based on conventionally accepted values which have been adjusted for project-area economic factors. Estimate markups are shown in Table 1.

Table 1. Estimate Markups	
Item	Rate (%)
Net Cost Markups	
Labor (employer payroll burden)	10
Materials and process equipment	8
Equipment (construction-related)	8
Subcontractor	5
Material Shipping and Handling	2
Gross Cost Markups	
Contractors General Conditions	12
Start-up, Training and O&M	2
Undesigned/Undeveloped Detail Construction Contingency	35
Builders Risk, Liability and Auto Insurance	2
Performance and Payment Bonds	1.5
Escalation from 2014 (database costs) to Nov 2015 plus allowance for lower competition due to rural location	8

Labor Markup

The labor rates used in the estimate were derived chiefly from the latest published State Prevailing Wage Rates. These include base rate paid to the laborer plus fringes. A labor burden factor is applied to these such that the final rates include all employer paid taxes. These taxes are FICA (which covers social security plus Medicare), Workers Comp (which varies based on state, employer experience and history) and unemployment insurance. The result is fully loaded labor rates. In addition to the fully loaded labor rate, an overhead and profit markup is applied at the back end of the estimate. This covers payroll and accounting, estimator's wages, home office rent, advertising and owner profit.

Materials and Process Equipment Markup

This markup consists of the additional cost to the contractor beyond the raw dollar amount for material and process equipment. This includes shop drawing preparation, submittal and/or re-submittal cost, purchasing and scheduling materials and equipment, accounting charges including invoicing and payment, inspection of received goods, receiving, storage, overhead and profit.

Equipment (Construction) Markup

This markup consists of the costs associated with operating the construction equipment used in the project. Most GCs will rent rather than own the equipment and then charge each project for its equipment cost. The equipment rental cost does not include fuel, delivery and pick-up charges, additional insurance requirements on rental equipment, accounting costs related to home office receiving invoices and payment. However, the crew rates used in the estimate do account for the equipment rental cost. Occasionally, larger contractors will have some or all of the equipment needed for the job, but in order to recoup their initial purchasing cost they will charge the project an internal rate for equipment use which is similar to the rental cost of equipment. The GC will apply an overhead and profit percentage to each individual piece of equipment whether rented or owned.

Subcontractor Markup

This markup consists of the GC's costs for subcontractors who perform work on the site. This includes costs associated with shop drawings, review of subcontractor's submittals, scheduling of subcontractor work, inspections, processing of payment requests, home office accounting, and overhead and profit on subcontracts.

Contractor Startup, Training, and O&M Manuals

This cost markup is often confused with either vendor startup or owner startup. It is the cost the GC incurs on the project beyond the vendor startup and owner startup costs. The GC generally will have project personnel assigned to facilitate the installation, testing, startup and O&M Manual preparation for equipment that is put into operation by either the vendor or owner. These project personnel often include an electrician, pipe fitter or millwright, and/or I&E technician. These personnel are not included in the basic crew makeup to install the equipment but are there to assist and trouble shoot the startup and proper running of the equipment. The GC also incurs a cost for startup for such things as consumables (oil, fuel, filters, etc.), startup drawings and schedules, startup meetings and coordination with the plant personnel in other areas of the plant operation.

Builders Risk, Liability, and Vehicle Insurance

This percentage comprises all three items. There are many factors which make up this percentage, including the contractor's track record for claims in each of the categories. Another factor affecting insurance rates has been a dramatic price increase across the country over the past several years due to domestic and foreign influences. Consequently, in the construction industry we have observed a range of 0.5 to 1 percent for Builders Risk Insurance, 1 to 1.25 percent for General Liability Insurance, and 0.85 to 1 percent for Vehicle Insurance. Many factors affect each area of insurance, including project complexity and contractor's requirements and history. Instead of using numbers from a select few contractors, we believe it is more prudent to use a combined 2 percent to better reflect the general costs across the country. Consequently, the actual cost could be higher or lower based on the bidder, region, insurance climate, and on the contractor's insurability at the time the project is bid.

Material Shipping and Handling

This can range from 2 to 6 percent, and is based on the type of project, material makeup of the project, and the region and location of the project. Material shipping and handling covers delivery costs from vendors, unloading costs (and in some instances loading and shipment back to vendors for rebuilt equipment), site paper work, and inspection of materials prior to unloading at the project site. BC typically adjusts this percentage by the amount of materials and whether vendors have included shipping costs in the quotes that were used to prepare the estimate. This cost also includes the GC's cost to obtain local supplies; e.g., oil, gaskets and bolts that may be missing from the equipment or materials shipped.

Escalation to Midpoint of Construction for All Project Cost

In addition to contingency, it is customary for projects that will be built over several years to include an escalation to midpoint of anticipated construction to account for the future escalation of labor, material and equipment costs beyond values at the time the estimate is prepared. For this project, the anticipated rate of escalation is 3 percent per annum.

No escalation to midpoint of construction is applied to the alternative costs presented here. It is assumed that the project design team will apply escalation to the packaged and phased alternatives.

Because the material and labor costs for this estimate are based on information from early 2014, an escalation factor of 8% has been applied to bring costs to current conditions. The 8% also includes a small allowance for the less competitive bidding environment we are currently seeing in rural areas.

Undesigned/Undeveloped Detail Construction Contingency

The contingency factor covers unforeseen conditions, area economic factors, and general project complexity. This contingency is used to account for those factors that cannot be addressed in each of the labor and/or material installation costs. Based on industry standards, completeness of the project documents, project complexity, the current design stage and area factors, construction contingency can range from 10 to 50 percent. Contingency is applied at the estimators discretion based on the amount of Undesigned/undeveloped detail for the particular project

Performance and Payment Bonds

Based on historical and industry data, this can range from 0.75 to 3 percent of the project total. There are several contributing factors including such items as size of the project, regional costs, and contractor's historical record on similar projects, complexity and current bonding limits. BC uses 1.5 percent for bonds, which we have determined to be reasonable for most heavy construction projects.

SUMMARY ESTIMATE REPORT
WITH MARK-UPS ALLOCATED

Sweet Home Wastewater Facilities
Plan - Planning Level Costs

Project Number:	144605.004.001
BC Project Manager:	Harry Ritter
BC Office:	Portland
Estimate Issue Number:	1
Estimate Original Issue Date:	July 29, 2014
Estimate Revision Number:	7
Estimate Revision Date:	November 10, 2015
Lead Estimator:	Catherine Dummer
Estimate QA/QC Reviewer:	Butch Matthews
Estimate QA/QC Date:	July 27, 2104

Sweet Home Wastewater Facilities Plan - Planning Level Costs

Description	Total w/ Markups Allocated
Preliminary Treatment and Pumping	**6,607,976**
IP-1 - New Influent Pumping Station	
02 - Site Construction	140,510
03 - Concrete	60,237
09 - Finishes	10,025
11 - Equipment	1,998,576
15 - Mechanical	187,075
16 - Electrical	897,165
IP-1 - New Influent Pumping Station Total	3,293,588
IP-2 - Influent Pumping Capacity Expansion	
11 - Equipment	252,207
16 - Electrical	177,513
IP-2 - Influent Pumping Capacity Expansion Total	429,720
PT-1 - Mechanical Bar Screen Facility (one screen)	
01 - General Requirements	309
02 - Site Construction	11,163
03 - Concrete	62,974
05 - Metals	50,628
09 - Finishes	254
11 - Equipment	229,588
14 - Conveying Systems	948
15 - Mechanical	13,695
16 - Electrical	122,484
PT-1 - Mechanical Bar Screen Facility (one screen) Total	492,043
PT-2 - Additional Mechanical Bar Screen	
02 - Site Construction	892
11 - Equipment	198,841
15 - Mechanical	14,989
16 - Electrical	70,473
PT-2 - Additional Mechanical Bar Screen Total	285,195
PT-3 - Flow Diversion Pipe and Structure	
01 - General Requirements	154
02 - Site Construction	23,082
05 - Metals	1,311
11 - Equipment	44,694
15 - Mechanical	44,033
PT-3 - Flow Diversion Pipe and Structure Total	113,274

Description	Total w/ Markups Allocated
PT-4 - Grit Removal Facility	
01 - General Requirements	1,014
02 - Site Construction	78,671
03 - Concrete	302,018
05 - Metals	85,055
11 - Equipment	846,466
15 - Mechanical	184,604
16 - Electrical	496,327
PT-4 - Grit Removal Facility Total	1,994,156
Secondary and Tertiary Treatment	10,153,351
ST-1 - Aeration Basin Improvements	
02 - Site Construction	25,900
07 - Thermal & Moisture Protection	11,107
11 - Equipment	521,051
13 - Special Construction	29,955
15 - Mechanical	37,565
ST-1 - Aeration Basin Improvements Total	625,579
ST-2 - Secondary Clarifier Improvements	
02 - Site Construction	23,894
03 - Concrete	8,447
06 - Wood & Plastics	15,417
09 - Finishes	17,341
11 - Equipment	381,649
ST-2 - Secondary Clarifier Improvements Total	446,748
ST-3 - New Aeration Basin	
01 - General Requirements	1,543
02 - Site Construction	235,157
03 - Concrete	459,209
05 - Metals	203,404
09 - Finishes	6,274
11 - Equipment	443,155
13 - Special Construction	31,952
15 - Mechanical	249,013
16 - Electrical	598,909
ST-3 - New Aeration Basin Total	2,228,615
TT-1 - Tertiary Filtration	
02 - Site Construction	187,540
03 - Concrete	32,603

Description	Total w/ Markups Allocated
09 - Finishes	8,267
11 - Equipment	1,620,791
13 - Special Construction	53,254
15 - Mechanical	391,807
16 - Electrical	1,038,306
TT-1 - Tertiary Filtration Total	3,332,567
WWT-1 - Wet Weather Treatment Facility	
01 - General Requirements	1,543
02 - Site Construction	84,676
03 - Concrete	226,316
05 - Metals	73,846
09 - Finishes	3,674
11 - Equipment	2,051,182
13 - Special Construction	68,165
15 - Mechanical	137,474
16 - Electrical	872,966
WWT-1 - Wet Weather Treatment Facility Total	3,519,841
Disinfection and Outfall	869,537
D-1 - Existing Contact Tank and Disinfection Improvements	
02 - Site Construction	83,009
11 - Equipment	8,787
17 - Instrumentation	7,337
D-1 - Existing Contact Tank and Disinfection Improvements Total	99,133
D-2 - Wet Weather Disinfection Facility	
01 - General Requirements	2,314
02 - Site Construction	86,514
03 - Concrete	235,522
07 - Thermal & Moisture Protection	1,366
11 - Equipment	16,541
15 - Mechanical	58,624
17 - Instrumentation	7,337
D-2 - Wet Weather Disinfection Facility Total	408,218
OI-1 - Outfall Improvements	
01 - General Requirements	771
02 - Site Construction	255,145
03 - Concrete	17,434
07 - Thermal & Moisture Protection	12,290
08 - Doors & Windows	2,559

Sweet Home Wastewater Facilities Plan - Planning Level Costs

Description		Total w/ Markups Allocated
11 - Equipment		65,646
15 - Mechanical		8,340
	OI-1 - Outfall Improvements Total	362,186
Biosolids Treatment and Disposal		1,187,240
BS-1 - Biosolids Handling Improvements		
02 - Site Construction		55,484
09 - Finishes		4,593
11 - Equipment		733,950
15 - Mechanical		99,306
16 - Electrical		293,907
	BS-1 - Biosolids Handling Improvements Total	1,187,240
Site Civil		532,451
CS-1 - Civil Site Work		
02 - Site Construction		532,451
	CS-1 - Civil Site Work Total	532,451
Miscellaneous Improvements		739,183
M-1 - Miscellaneous Improvements		
11 - Equipment		103,361
15 - Mechanical		375,652
	M-1 - Miscellaneous Improvements Total	479,013
SG-1 - New Standby Generator		
02 - Site Construction		3,236
03 - Concrete		9,114
16 - Electrical		247,820
	SG-1 - New Standby Generator Total	260,170
	Grand Total	20,089,737